# DYNAMIC PROGRAMMING MODEL AND QUANTITATIVE ANALYSIS, ROSWELL BASIN, NEW MEXICO

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and

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#### ABSTRACT

Aquifers enclosed or confined by sediments that impede or retard the vertical movement of groundwater generally leak. This report is concerned with the optimal utilization of a composite system of interacting aquifers in which one aquifer leaks into another, and with a surface-water subsystem from the viewpoint of maximizing the value added to the system by operation of the system over a long period of time. The problem is solved through dynamic programming, a sequential decision-making approach. Stochastic recharges, base flow, and natural discharge from the system are considered in addition to the interaquifer leakage.

The model thus developed is applied to the coupled leaky aquifer and surface-water system of the Roswell Basin which forms part of the Pecos River Basin in New Mexico, and which is believed to be one of the largest naturally recharging multiaquifer systems in the world. A hydrologic analysis of the basin is also performed and the results are fed into the dynamic-programming model as inputs.

The optimal operating policies for the two aquifers of the Roswell Basin are derived by taking into consideration the physical characteristics of the system and the extent of the areas to be irrigated with the water drawn from the system. The optimal operating policies are strongly influenced by the interaquifer leakage. As a result, coupled leaky aquifers should be considered for conjunctive utilization only.

A mathematical model for the prediction of drawdowns due to the operation of well fields in coupled leaky aquifers is also presented as an appendix.

#### ACKNOWLEDGMENTS

Many individuals and groups were helpful in this study. Special appreciation is expressed to many private, local, state, and federal agencies for providing the data without which this investigation would have been very difficult. They include the offices of the State Engineer in Roswell and in Santa Fe, the U. S. Geological Survey, U. S. Bureau of Reclamation, Smith Machinery Company of Roswell, Pecos Valley Artesian Conservancy District, the State Climatologist, and many individual farmers in the Roswell Basin. I especially thank Zane Spiegel, Fred Hennighausen and Don Akin of the office of the State Engineer, William Hale and Ed Welder of the U. S. Geological Survey, and Cecil Schrimsher of Smith Machinery Company of Roswell.

C. E. Jacob reviewed the preliminary draft of the report before his death on January 30, 1970. I am grateful to M. S. Hantush for many useful discussions and suggestions and for reviewing part of this report, and to Gerardo W. Gross who devoted considerable time to reviewing the manuscript; both are at New Mexico Institute of Mining and Technology, Socorro.

The study is part of an interdisciplinary-interuniversity project entitled "A Comprehensive Water Resources Analysis of a Typical Overdrawn Basin in an Irrigated Semiarid Area-Pecos River Basin, New Mexico." The principal investigators for the whole project were the late C. E. Jacob and Z. A. Saleem, hydrologists, and W. K. Summers, geologist, New Mexico Institute of Mining and Technology, and George L. Dawson and Robert R. Lansford, agricultural economists; John W. Hernandez, civil engineer; Harold Dregne, soil scientist; Arden Baltensperger, agronomist, all of New Mexico State University, Las Cruces; and, Ralph d'Arge, economist; W. H. Ellis, law, University of New Mexico, Albuquerque. I am also grateful to H. R. Stucky, Director of the New Mexico Water Resources Research Institute, who served as the coordinator of all phases of the project and was cooperative and helpful, especially in obtaining data.

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#### GLOSSARY

Most of the terms used in this publication are defined when they first occur in the appropriate sections. The terms used to discuss hydrologic concepts are those that are in common use by groundwater hydrologists. However, some geohydrologic terms not in common usage are defined below.

- Hydraulic conductivity (L/T): The ease with which water will pass through a medium, stated as the number of cubic feet per day per square foot of cross-sectional area of the material when subjected to a hydraulic gradient of one foot per foot length of flow (or simply ft/day). It is used in place of the term coefficient of permeability.
- Leakage factor (L): The square root of the ratio of transmissivity of the leaky aquifer to leakance of the aquitard through which leakage is taking place, usually expressed in feet.
- Leakance or leakage coefficient (1/T): The ratio of vertical hydraulic conductivity of an aquitard to its thickness.
- Potentiometric surface: Replaces the term piezometric surface and is the imaginary surface representing the heights to which water would rise in tubes or observation wells penetrating the confined aquifer at various points.
- Storativity: The volume of water of an aquifer that a vertical column of the aquifer of unit cross-sectional area releases from storage as the average head within this column declines a unit distance. The storativity of an unconfined aquifer, for all practical purposes, corresponds to its specific yield (see Hantush, 1964). Storativity is used in place of the term storage coefficient.
- Transmissivity or coefficient of transmissivity (L2/T): The flow capacity of an aquifer in cubic feet per day per foot width of the aquifer (or in ft³/day); equal to the product of hydraulic conductivity times the saturated thickness of the aquifer. It replaces the term transmissibility or coefficient of transmissibility.

# DYNAMIC PROGRAMMING MODEL AND QUANTITATIVE ANALYSIS,

# ROSWELL BASIN, NEW MEXICO

Z. A. Saleem and C. E. Jacob

# Chapter 1 INTRODUCTION

# Background and Purpose of the Study

The optimal utilization of surface reservoirs and of aquifers, both separately and conjunctively, has been studied in the past by several authors [Little, 1955; Moran, 1959; Maass, et al., 1962; Buras, 1963; Burt, 1964; Young, 1967; and Hall et al., 1968]. Most of the studies were about reservoirs or aquifers. The investigations of Buras and Burt were related to conjunctive utilization of single aquifers and single surface reservoirs. They were hypothetical and the results were not applied to any specific basin, nor did they consider the pumping costs arising from the drawdowns at the pumping wells.

The investigation covered in this report is concerned with the optimal operation of two coupled leaky aquifers and a surface-water system as a function of time. Preliminary results of part of this study have been presented elsewhere [Saleem and Jacob, 1968].

The study is specifically based on a complex hydrologic system and the results are applied to the Roswell Basin in southeastern New Mexico, which forms part of the Pecos River Basin (see figure 1). The Roswell Basin consists of two coupled leaky aquifers and associated sources of surface water, mainly the Pecos River system. Water levels in both aquifers have been declining since the agricultural development of the basin due to heavy artificial draft.

The reason for applying the study to the Roswell Basin was to derive optimal operating policies for the basin so that the value added to the basin would be maximized over a long period of time. The optimal operating policies take into consideration the physical characteristics of the basin.

# Approach and Presentation

The problem of optimal operation of two coupled leaky aquifers and a surface-water system is formulated mathematically, using the technique of dynamic programming. Two dynamic programming derivations are described, the first for optimization of the system as a

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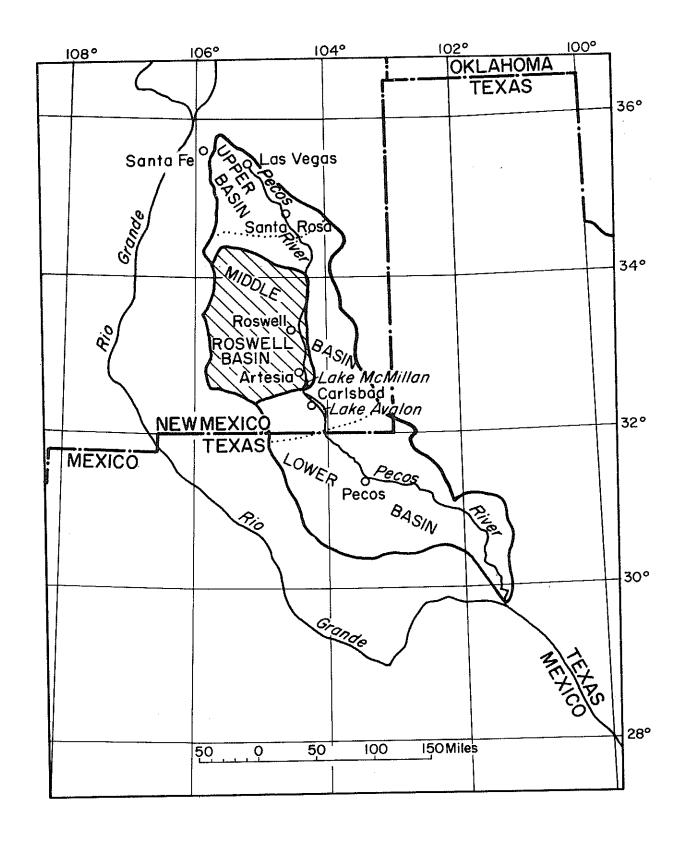


Figure 1. Location of Roswell Basin, New Mexico.

function of time and the second for optimization in space. The results obtained are discussed and some conclusions are drawn.

A hydrologic description of the Roswell Basin is given and some of the hydrologic analyses performed are discussed. Mathematical models were derived for predicting the response of the system to the operation of well fields in coupled leaky aquifers and are presented in Appendix A.

# Sources of Data

The basic data, most of them unpublished, were provided by the U. S. Geological Survey, the offices of the State Engineer in Roswell and Santa Fe, U. S. Bureau of Reclamation, Smith Machinery Company of Roswell, Pecos Valley Artesian Conservancy District, Hagerman Canal Company, and some individual farmers in the basin.

Most of the water-level data from the U.S. Geological Survey were based on preliminary land surface elevations. The data from Fiedler and Nye [1933], shown on the water table and the potentiometric surface maps for January 1926, were not all restricted to the month of January and some observations were made during later months.

#### Chapter 2

# THE ROSWELL BASIN, DESCRIPTION AND ANALYSIS

A description and analysis of the components of the hydrologic cycle of the Roswell Basin are presented in this chapter. Some of the derivations and data are presented in the appendices.

# Previous Studies

The geology and hydrology of the Roswell Basin have been studied frequently during the last three or more decades. The information gathered in those reports is fairly valuable for a qualitative understanding of the basin and it is helpful in predicting future responses of the system to various stresses using modern techniques.

One of the earliest reports [Means and Gardner, 1900], is primarily concerned with the soil conditions and includes a water-table map of the Roswell area. Fisher [1906] published a reconnaissance study of the area. One of the most thorough investigations [Fiedler and Nye, 1933] was conducted in 1925-1928 and contains many valuable data relating to groundwater conditions at that time, but unfortunately many of the modern techniques of hydrologic analysis were then unavailable. Morgan [1938] published a report about the shallow-water resources in the basin.

An extensive investigation of the surface-water resources was published by the National Resources Planning Theis's work on the origin of water Board [1942]. in Major Johnson Springs, at the southern end of the basin, was published in 1938. Theis [1951] also investigated the relation of the Hondo reservoir to the artesian aquifer. Bean's report on the geology of the Roswell Basin and its relation to the Hondo reservoir appeared in 1949. Hendrickson and Jones [1952] published a report on the groundwater resources of Eddy County, which includes the southern part of the basin. Hantush [1955] carried out a quantitative study of the basin during 1954-55, applying the theory of leaky aquifers to the basin and estimating the characteristics of the aquifers at several locations. From recharge and discharge studies, Hantush concluded that the artificial discharge from the aquifers exceeded the natural recharge to the aquifers.

Reports of the investigations of Thomas [1963], Hood [1963], Motts and Cushman [1964], and Mower et al. [1964] have been published as Water Supply Papers of the U. S. Geological Survey. Hood et al. [1960] studied saline water occurrence in the basin. Among the several unpublished reports about the basin, one by Mower [1958] is significant, dealing with the pumpage in the basin.

The U. S. Geological Survey has been working on an analog model of the basin since 1963. Four recent publications relating to the basin are by Spiegel [1967], Havenor [1968], Kinney et al. [1968], and Maddox [1969]. A bibliography pertaining to the Pecos River Basin was compiled by Hernandez and Eaton [1968].

# Location, Topography and Climate

The Pecos River Basin is conveniently subdivided [Thomas, 1963] into an upper basin, above Alamogordo Reservoir in New Mexico; a middle basin, above Red Bluff Reservoir also in New Mexico; and a lower basin in Texas (see figure 1).

The Roswell Basin forms most of the middle Pecos River Basin and is the most important area of the basin with respect to water-resource utilization and economic productivity. The basin is bounded on the east by the high plains--about 25 miles east of the Pecos River— on the south by the Seven Rivers Cuesta, on the west by the high Sacramento Mountains, and extends northward nearly to Mesa.

The basin has a semiarid climate. The winters are usually cold enough but too dry for appreciable snow accumulation, and summers are dry with frequent thundershowers. The normal frost-free period at Roswell extends from April 11 through October 10 [Blaney and Hanson, 1965]. More than 75 percent of the total annual precipitation occurs during this period, which is also the growing season. Because the average rainfall (1905 through 1968) at Roswell and Artesia is 11.7 and 11.2 inches respectively, and the deviations from the average are large, irrigation is essential in all parts of the basin for crop production.

Yearly averages of the precipitation totals at Roswell and Artesia are shown in table 1.

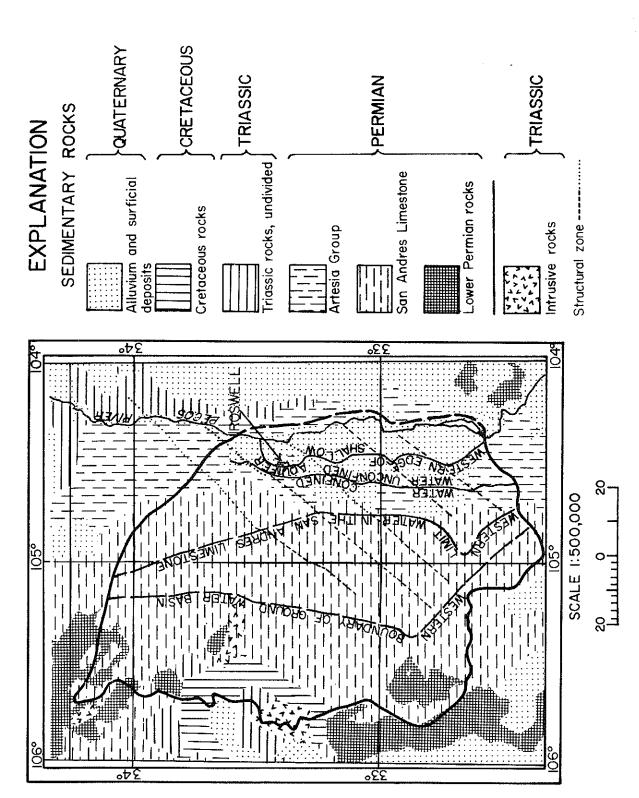
# Hydrogeology

A generalized geologic map of the Roswell Basin, and a geologic section at the latitude of Roswell, are shown in figures 2 and 3 respectively. The main aquifers in the

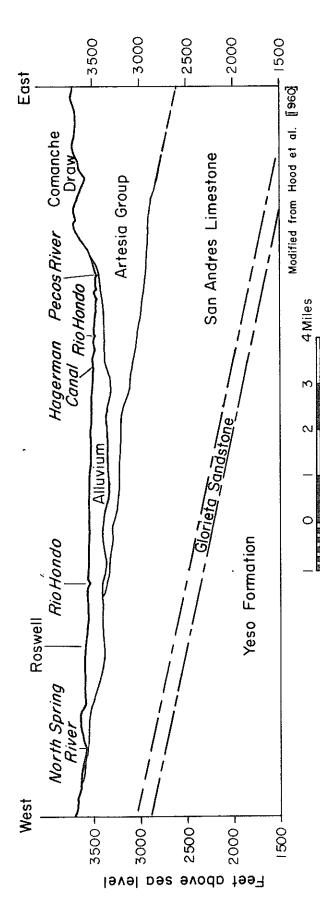
Table 1. Average annual rainfalls (in ascending order) at Roswell and Artesia, New Mexico, 1901-1968.

Order (m)	Year	Rainfal (R) (inches	111		Order (m)	Year	Rainfa (R) (inche	1II
1	1917	5.09	0.0145		35	1930	11.49	0.5072
2	1910	5.16	.0290		36	19 36	11.54	.5217
3	1924	5.63	.0435	ı	37	1946	11.55	.5362
4	1956	5.68	.0580		38	1962	11.56	.5507
5	1927	5.71	.0725	l	39	1937	12,29	.5652
5 6	1964	6.06	.0870	ı	40	1929	12.43	.5797
7	1963	6.12	.1014	- [	41	1940	12.49	.5942
8	1934	6.61	.1159	Ì	42	1944	12.55	.6087
9	1945	6.64	.1304	١	43	1913	12.87	.6232
10	1947	6.66	.1449		44	1960	13.04	.6377
11	1922	6.78	.1594	1	45	1921	13.30	.6522
12	1951	7.12	.1739		46	1920	13.61	.6667
13	1965	7.19	.1884		47	1942	13.81	.6812
14	1953	7.33	.2029		48	1950	13.91	.6957
15	1961	7.42	.2174		49	1907	14.03	.7101
16	1933	7.61	.2319		50	1906	14.12	.7246
17	1957	7.64	.2464		51	1904	14.35	.7391
18	1952	7.69	.2609		52	1949	14.58	. 75 36
19	1959	7.81	.2754		53	1968	14.82	.7681
20	1909	8.22	.2899		54	1915	14.86	.7826
21	1967	8.24	.3043		55	1928	15.08	.7971
22	1918	8.34	.3188		56	1914	15.36	.8116
23	1903	8.40	.3333		57	1916	15.37	.8261
24	1955	9.05	.3478		58	1902	15.70	.8406
25	1943	9.55	.3623		59	1931	16.12	.8551
26	1954	9.74	.3768		60	1926	16.17	.8696
27	1948	10.20	.3913		61	1901	16.33	.8841
28	1925	10.28	.4058		62	1958	16.63	. 8986
29	1935	10.57	.4203		63	1923	17.02	.9130
30	1966	10.57	.4348		64	1911	17.88	.9275
31	1908	10.62	.4493		65	1932	19.88	.9420
32	1938	10.81	.4638		66	1919	20.42	.9565
33	1939	10.96	.4783		67	1905	21.37	.9710
34	1912	11.21	0.4928		68	1941	34.61	0.9855

<sup>(\*)</sup> n = 68.



Generalized geologic map of the Roswell Basin, New Mexico. Figure 2.



Geologic section at the latitude of Roswell, Roswell Basin, New Mexico. Figure 3.

basin occur in the three geological formations, the alluvium, the Artesia Group, and the San Andres Limestone. Figure 4 shows most of the boundaries of the various aquifers in the Roswell Basin.

#### Alluvium (Shallow Aquifer)

Thick valley-fill materials of Tertiary (?) and Quaternary age occur almost wholly west of the Pecos River in an area 12 to 20 miles wide and 60 to 70 miles long. The sediments are mostly conglomerates, gravel, sand, silt, and clay, and are quite heterogenous. The alluvium and, near Roswell, the top part of the underlying Artesia Group, constitute the shallow aquifer of the Roswell Basin. The thickness of the shallow aquifer ranges from more than 400 feet just east of the river to zero along its western boundary. An average north-south cross section and an average east-west cross section based on the three isopach maps of the principal aquifers [Hale, personal communication; Kinney, et al., 1968] are shown in figures 5 and 6.

During 1967 and 1968 the pumpage from the shallow aquifer was about 30 percent of the total amount of ground-water pumped.

Artesia Group (Shallow Confined Aquifer)

The Artesia Group embraces five formations of Permian age occurring in eastern New Mexico and West Texas [Tait et al., 1962]. It separates the underlying San Andres Limestone from the overlying alluvium and acts mostly as an "aquitard" (that is, it transmits appreciable quantities of water but has a low storativity).

The shallow confined aquifer is in the Artesia Group and has a maximum thickness of more than 500 feet in the southern part of the basin near Artesia and is like a wedge, becoming thinner to the north near Roswell. The groundwater extracted from the shallow confined aquifer is less than 8 percent of the total annual groundwater pumpage in the Roswell Basin.

In this report, the shallow confined aquifer is not treated as a distinct aquifer. One-half of the pumpage from each well in this aquifer is incorporated into the shallow aquifer and the other half of the pumpage is added to the pumpage from the principal confined aquifer.

San Andres Limestone (Principal Confined Aquifer)

The San Andres Limestone, of Permian age, conformably overlies the Glorieta Sandstone, also of Permian age, and is overlain by the Artesia Group. It is composed almost

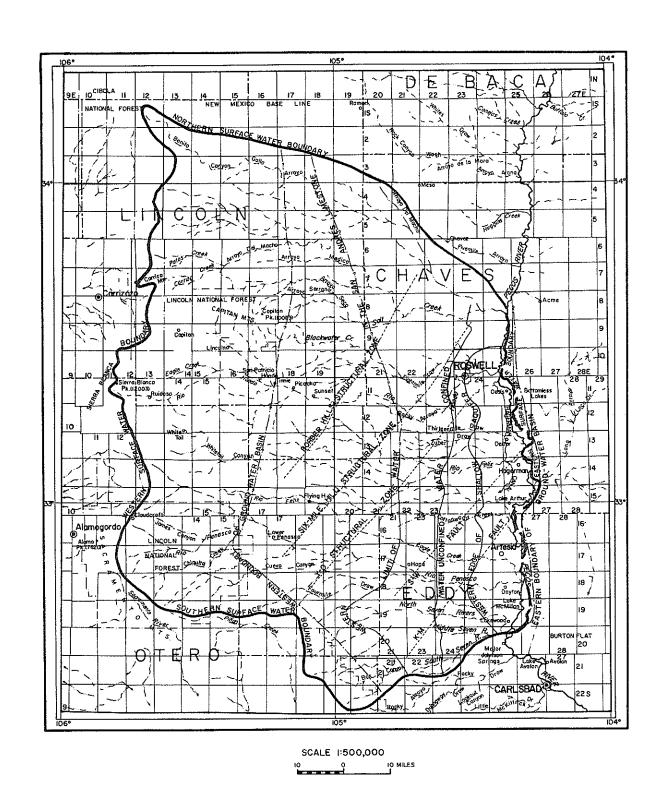
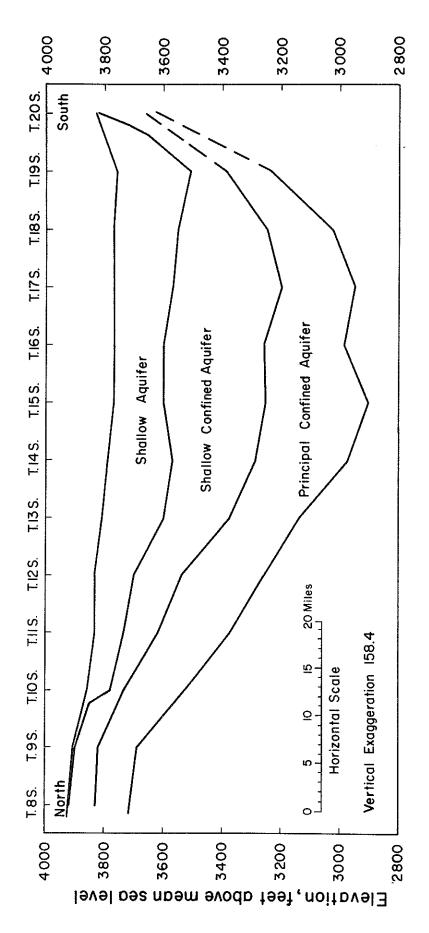
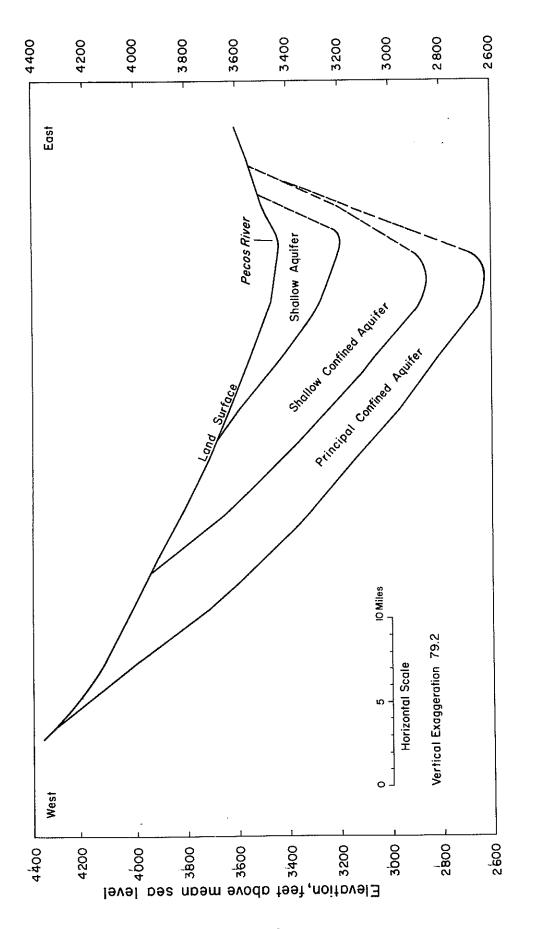


Figure 4. Boundaries of the various aquifers in the Roswell Basin, New Mexico. (Structure after Kinney, et al., [1968]).



Average north-south cross section of the Roswell Basin, New Mexico. Figure 5.



Average east-west cross section of the Roswell Basin, New Mexico. Figure 6.

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entirely of limestone, dolomite, anhydrite, and anhydritic limestone. The San Andres Limestone is widespread, correlative units having been recognized in New Mexico, West Texas, and parts of eastern Arizona [see Havenor, 1968, and Maddox, 1969, for details].

The principal confined aquifer occurs in San Andres Limestone except in the southern part of the basin, where it also includes the bottom portion of the Artesia Group [Maddox, 1969]. The maximum thickness of the principal confined aquifer is greater than 500 feet near Artesia, and varies on the average from about 150 feet to about 300 feet.

The principal confined aquifer contributes more than 60 percent of the total groundwater pumped in the basin.

# Surface Hydrology

The Pecos River and its tributaries drain the Roswell Basin. The Pecos River gains water from the Roswell Basin as groundwater outflow and loses water through evaporation, and, to some extent, by diversion for irrigation. Salt cedars along the banks of the river consume considerable amounts of water. Table 2 shows the base flow from the Roswell Basin, between the gaging stations at Acme and at Artesia, to the Pecos River on a monthly basis. The annual base flow has been decreasing from more than 60,000 acrefeet per year in the 1930s to less than 25,000 acrefeet per year during the late 1960s. The causes for depletion of the Pecos River flow are discussed in a separate section later in this chapter. Figure 16 shows the yearly base flows corrected for depletion because of the surface—water diversions.

The writer estimates that the Pecos River system loses about 10,000 acre-feet of water per year through evaporation in the Roswell Basin, not including losses from the McMillan Reservoir (figure 1).

#### Surface Water

Before the large-scale utilization of groundwater, surface water was used for irrigation along the Pecos River and along the lower reaches of some of its tributaries in the Roswell Basin. Some of these tributaries near Roswell were fed by several springs, described elsewhere in this report.

In the early 1900s several artesian wells were drilled and the artesian heads were lowered as a consequence. Some of the springs started drying up, and the lands formerly irrigated from the streams were either abandoned or forced to rely upon groundwater partially or completely.

Estimated Pecos River base flows (in thousands of acre-feet) from Acme to Artesia, New Mexico. Table 2.

,CJ						
Annual	86.7 78.6	71.0 64.3 58.4 73.2 61.8	77.9 66.8 67.0 78.6 67.9	77.1 92.6 71.0 50.2 64.5	62.5 79.2 55.4 46.1 49.6	101.3 110.9 61.3 65.1 54.4
Annual	84.4	67.5 60.3 57.7 70.0 60.3	74.9 61.1 64.7 72.4 63.1	71.8 88.3 66.7 46.1 59.7	57.9 78.9 54.4 46.1	101.3 110.9 59.8 61.5 48.2
Dec.	13.8 9.0	8.3 6.1 7.7 9.0	8.7 9.3 7.3	9.7 10.6 6.8 5.4 7.6	6.8 5.5 6.2 6.2	18.9 10.9 7.7 7.3 5.6
Nov.	12.0	7.7 5.4 8.6 8.6 8.6	7.3 7.7 7.3 8.7	8 9 9 9 8 9 8 9 8 9 8 9 8 9 8 9 9 8 9	4.6 6.6 6.4 5.1	16.5 9.4 5.0 6.6 4.8
Oct.	8.5	7 4 5 2 . 8 4	6.4 4.8 5.3 6.4	6.1 7.4 6.3 2.7 4.8	3.6 5.0 3.1	13.8 9.1 3.6 6.0 4.2
Sept.	4.0	4.0 2.8 3.5 4.8	4.0 4.0 5.6	4.3 4.2 4.0 1.1 3.9	2.9 5.3 3.0 1.7	10.7 7.2 3.0 3.7 1.8
Aug.	3.9	3.7 3.4 1.5 4.0	3.1 3.0 5.0 4.1	3.0 2.7 3.0	3.7 4.9 2.0 1.4	8.5 5.1 2.9 1.7
July	5.2	3.6 4.5 5.0 8.0 8.0	2.7 2.5 3.9 2.6	3.9 4.7 2.7 2.6	2.5 4.2 1.2 3.2	5.00 2.00 3.00 3.00 3.00 3.00
June	4.6	3.4 3.6 2.4 3.7 2.3	4.4 1.6 2.4 3.9	2.5 2.6 5.7	2.9 2.9 1.8 3.1	4.1 5.7 2.6 2.3
May	5.2	3.1 4.1 6.2 2.6	7.6 3.7 3.9 5.1	4.5 4.5 4.4	3.7 6.3 3.4 3.1	4.1 7.4 3.3 3.3
Apr.	5.6	4.8 4.2 6.3 7.4	7.8 5.9 4.7 5.3	4.3 8.1 4.7 6.0 5.0	4.7 4.2 3.2	4.2 4.3 4.2 3.5
Mar.	5.8	6.7 8.1 5.5 7.0	7.4 7.3 7.1 8.6 5.4	7.0 8.1 6.5 8.5	7.6 8.7 5.7 5.2 4.4	4.6 11.7 6.7 5.7 5.8
Feb.	7.4	7.8 7.5 8.2 5.8	6.8 7.8 7.6 7.9	8 7 8 7 8 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5	7.0 8.2 5.9 6.7	4.9 13.0 7.2 6.9 5.5
Jan.	8.4	9.8 8.6 6.4 10.3	8.7 7.7 9.6 6.8	9.8 10.2 10.3 6.8	8 8 6 8 8 6 8 6 8	5.2 17.2 10.6 9.8 6.6
Year	1919	1921 1922 1923 1924 1924	1926 1927 1928 1929 1930	1931 1932 1933 1934 1935	1936 1937 1938 1939 1940	1941 1942 1943 1944 1945

Estimated Pecos River base flows (in thousands of acre-feet) from Acme to Artesia, New Mexicol (continued). ⟨ Table

a12																										
Annua	57.6	6	6	ij	2		•		•	51.8	•	6	ė	34.6	'n	Ġ		•	30.8			4	21.9	5.		55.5
Annual	51.7		•				•			43.5	•	2	φ.	29.8	4.	r-i	-	o,	22.0	7	ŝ	7.	17.6	ċ		51.0
Dec.	8.8	•	•	•	•		٠	٠	•	6.5	•		•	4.3	•	•	•	٠	2.9	•	•	•	1°0	•		6.5
Nov.	7.3									8.4	•			3.6			•		2.2	•	6		7.3	•		5.5
Oct.	5.2		•	•	•		٠	•	•	6.2				3.6	•	•	6		1.5		۲.		1.0	•		4.2
Sept.	1.8	•	•	٠	•		•	•	•	2.1	•	•	•	2.1	•		•	2.0	1,3	9.	9.	•	1,3	•		2.9
Aug.	1.5			•	•		•	•	•	1.4	•	•	•	1.3		•	•	1,5	0	4.	9.		1.6	6.		2.5
July	2.3	•	•	•			٠	•	•	1.6	•	•	•	1.2	٥.	4.		•	0	6.	9.	5.	٥.	4.	No	2.3
June	1.8	٠	•	•	•		•	٠	•	2.3	•	•	•	1.5	•	•	1,4	•	0		o.	•	1.1	•		2.5
May	3.2		•		•		•	•	•	2.3	•	•		1.5	•	•	•	•	2,3	•	•	•	0.5	•		3.4
Apr.	3.5		•	•	•		•	•		2.3	•	•	•	1.8	•	•	•	•	2.9	•	•	•	I.1	•		4.0
Mar.	5.2	•	٠	•	•					3.5		•		2.8		•	•	•	2.8	•	•	3,4	2.1	2.7		5.2
Feb.	5.2	•			•		•		•	3.4	•		•	3.1	•		•	•	2.9		•	•	2,5	•		5.6
Jan.	5.9	•	•	•		L ~	ر. د	4.7	3.6	3,5	5,5		•	3.0				•	3.2	•		•	2.3	•	     	9*9
Year	1946	1947	1948	1949	1950	Ł	<b>?</b> ∣	S	S	1954	ĽΩ	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	Mean	1968

From records in the office of the New Mexico State Engineer, Santa Fe. ---

 $\alpha$ 

Corrected for depletion by the river pumps.

Almost all of the surface water used in the Roswell Basin is for irrigation. During 1967 and 1968 only 3.8 percent and 3.1 percent, respectively, of the total water used for irrigation was surface water. Surface water is used mostly in the Roswell-East Grand Plains area and in the Dexter-Hagerman area of the Roswell Basin. The sources of surface water are: (1) Hagerman Canal, which in turn gets water through diversions from the Rio Hondo and some groundwater, (2) private drains which get their water mostly from the Pecos River, and (3) some direct pumpage from the river. Table 3 shows the monthly and yearly totals of surface water usage in the basin for several years.

# Depletion of River Flow in the Middle Basin

The streamflow generated by the middle basin (figure 7) has been decreasing for the last four decades [see Thomas, 1963]. Major factors for the depletion are: (1) gradual increase of and continued effects of groundwater exploitation in the middle basin; (2) repeated droughts in the area since 1943; and, (3) increased consumptive-waste use by salt cedars.

Figure 7 shows a cumulative departure from the 60-year mean precipitation (1901-1960) at the two stations in the upper basin (Las Vegas and Santa Rosa), and at the three stations in the middle basin (Roswell, Artesia, and Carlsbad). Figure 7 also shows the outflow from the upper basin, as recorded at Puerto de Luna, and the difference between the inflow from the upper basin and outflow from the middle basin as given by the sum of the runoffs of the Pecos River at Red Bluff and the Delaware River near Red Bluff.

In the 52 years 1916-1967, the cumulative departure in precipitation from the mean in the middle basin is about four and a half times the mean and this departure is also reflected in the runoff curve for the middle basin. During the same 52-year period, the cumulative departure from mean precipitation in the upper basin is only 100 percent. The annual outflow from the upper basin also does not show any progressive decreasing trend.

Figure 8 shows the cumulative streamflow/precipitation relationship in the Pecos River middle and upper basins. There is a gradual decrease in the slope (runoff per unit precipitation) of the curve for the middle basin after 1925, and a negative slope from 1943 to 1955 and from 1966 to 1968. These are caused mostly by the three factors listed earlier as causes for depletion of the Pecos River flow in the middle basin. The graph for the upper basin has a uniform slope, indicating no significant increase in depletion of the river flow in the upper basin.

Monthly surface-water diversions (in acre-feet) in the Roswell Basin, New Mexico, 1955-1968. Table 3.

į			· ron	ADFLL	May	n nue	Λτη r	Aug.	sept.	Oct.	Nov.	Dec.	Total
TAPP	99	87	0.470	969	754	979	2,584	1,652	1,126	237	329	147	9,114
95		$\sim$	$\circ$	,29	.03	34	76,	92	) Te	H		$\infty$	1,33
957	31	99	07	49	.93	90	986	,26	, 91	$Q_{J}$	ì—		0,47
958	,23	4.2	,64	16°	5.54	<b>,</b> 92	,04	43	43	77	Ц	$\sim$	7,70
1959 1959	•	7,00,7	4,905 705	5,164	4,5337	2000 8000 1000	6,078	6,307	4,309	2,703	2,538	1,450	48,054
V .		<b>,</b>	ر - و	7	0	ς Σ	ر د د	C7 (	a A	נ ל			7 T C
1961		354		<u>~</u>	0	57	60	78	<u>,</u> 47	34	9	38	3,50
96	-17 '	Ω Ω	, 80	62	95	4,176	•	اكر	4,388	1,176	276	45	34,838
963	92	12	9	, 20	12	,58	62	5	96,	<sup>4</sup> 48	$\mathcal{O}$		0,23
954	9	3,00	80	96	42	500	დე ლ	60,	, 43	7.0	Q,		7,36
965	35	, 8	25	95	, 5 8	986	,56	æ ∞ 1	90	8 7 6	59	Н	5,32
		804	,18	,24	,63	2 ty *	39	88	,82	986	$\vdash$	ĽΩ	9,20
	787	φ	2,359	2,347	4,003	3,148	3,925	3,697	$(\lambda)$				25,528
96	Ŋ		93	77	• 79	,29	,2 <sup>4</sup>	<b>,</b> 42	51	33	S		2,42
e a													
1955-	771	1,524	3,623	3,888	3,387	3,690	4,410	4,249	3,155	1,493	658	383	31,231
96	<u></u>	,52	62	χ χ	33	69	₹ † €	,24	77	, 49	Ŋ	$\infty$	1,23
ø													
cent	2.47	4.88	11.60	12.45	10.84	11.81	14.12	13.61	10.10	4.78	2.11	1.23	100.00

Total flow of Hagerman Canal, which consists of surface-water diversions and some ground-water, was used in the computations.

Source: Based on the Watermaster Reports, Pecos Valley Surface Water Listrict.

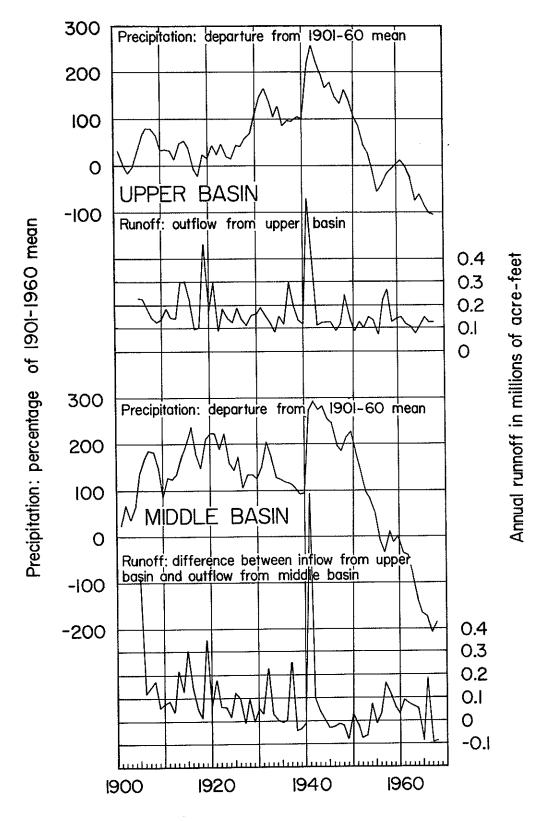


Figure 7. Cumulative percentage departure from mean precipitation and streamflow, 1905-1968, Middle and Upper Pecos River Basins, New Mexico.

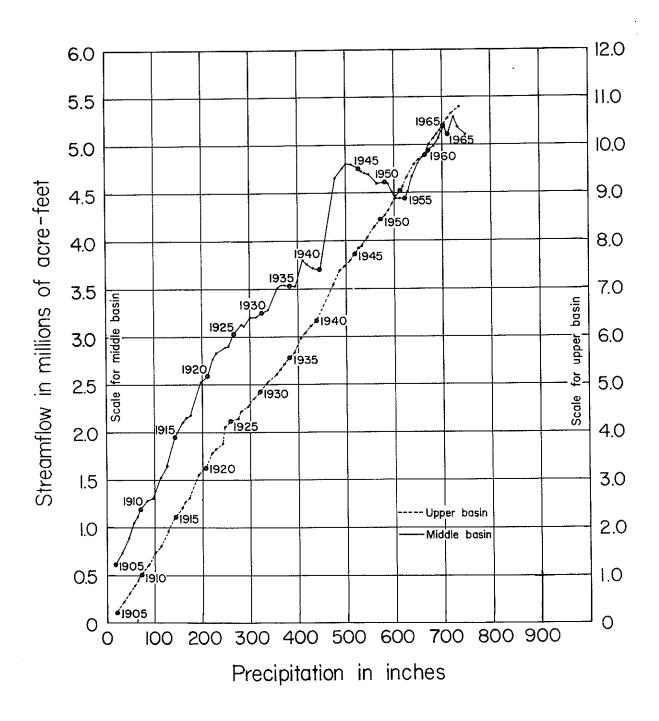


Figure 8. Cumulative mass curves of streamflow vs. precipitation, 1905-1968, Pecos River Middle and Upper Basins, New Mexico.

The other two causes of depletion are discussed elsewhere in this report. It may be mentioned here that groundwater exploitation in the middle basin has increased not only in the Roswell Basin but also in the Carlsbad area (figure 1).

# Geohydrology

The principal confined aquifer is coupled to the shallow aquifer through the shallow confined aquifer, and there is water leakage from one aquifer to the other, varying with the differences in head between the two aquifers. The rate of leakage from one aquifer to the other is a function of time as well as space and depends on the hydraulic conditions. This portion of the report discusses evaluation of the various components of the subsurface hydrology of the Roswell Basin and how they interact.

# Aquifer Characteristics

# Previous Studies

The hydraulic characteristics of the aquifers, namely, transmissivity, storativity, and leakance, had not been determined until Hantush [1955] made his analysis, except for estimates by Hale [reported in Hantush, 1955] and by Theis [1951], of the transmissivities of the valley fill and of the San Andres Limestone near Hondo reservoir. [1951] also estimated the storativity of the San Andres Limestone near Hondo reservoir. Hantush [1955] ran four pumping tests in the principal confined aquifer and six pumping tests in the shallow aquifer in the basin and applied the theory of leaky aquifers developed by him and Jacob [1955]. From his analysis he estimated the transmissivities and storativities of the two main aquifers and leakances for the shallow confined aquifer at different locations in the basin. Hantush's average values for the hydraulic characteristics at the aquifers and the aquitard are shown in table 4.

Summers [personal communication, 1968] ran some step-drawdown tests in the flowing wells in the northern half of the basin in early 1968 and obtained transmissivity values for the principal confined aquifer that were close to the values obtained by Hantush.

# Transmissivities of the Aquifers

Step-drawdown tests. The theory of step-drawdown tests was developed by Jacob [1947]. Briefly, according to Jacob, the drawdown,  $\mathbf{s}_{\mathbf{w}}$ , in a well can be expressed as

$$s_w = B(r_w, t)Q + CQ^2$$
 (1)

where  $B(s_w,t)$  is the head loss in the formation per unit discharge. For nonleaky confined aquifers,

$$B(r_w,t) = \frac{1}{4\pi T} W(u_w)$$
 (2)

Rorabaugh [1953] suggested that the head loss associated with turbulence should be expressed as  ${\sf CQ}^n$ , where  $\underline{n}$  is a constant, which is greater than 1 and which may exceed 2, and which is to be determined for individual wells.

If <u>B</u> is assumed to remain constant during the test period, then <u>B</u> and <u>C</u> can be evaluated from a step-drawdown test. In such a test a well is pumped during successive periods at uniform but differing rates which are fractions of the full capacity, and the drawdown is observed in the well. From these data, using (1), one gets a set of linear equations with <u>B</u> and <u>C</u> as unknowns which can readily be determined.

Step-Drawdown Tests in the Roswell Basin. Data for the step-drawdown tests in the Roswell Basin were taken from the files of Smith Machinery Company of Roswell who routinely run such tests to determine the optimum size of pump needed for a given well.

A well is pumped at a fraction of full capacity and then the discharge is increased in three or more steps, each step being of 30 to 40 minutes' duration. Sometimes the well is started at the maximum capacity and the discharge is then decreased in steps.

Since the duration of a typical routine test in the Roswell Basin is small (and reasonably constant from one test to another), and since the transmissivities are in general high, the following assumptions are made:

- 1) The effects of leakage are negligible and  $\underline{B}$  is given by equation (2).
- 2) Any changes in the values of  $\underline{B}$  from place to place in any particular aquifer are due to changes in the transmissivity values. This is based on the assumption that the parameter  $V(=r_W^2S/4t)$  is constant for each aquifer.

3) The effects of partial penetration of the wells are negligible.

More than 340 step-drawdown tests of wells tapping the three different aquifers in the basin were analyzed on the digital computer. These wells are scattered all over the basin. Values of  $\underline{B}$  and  $\underline{C}$  were calculated. Each well was classified according to depth and casing information as belonging to one of the three aquifers. Some wells tapped more than one aquifer and were classified as multipleaquifer wells.

Curves of  $\underline{B}$  versus  $\underline{T}$  were plotted on logarithmic paper using and treating  $\underline{\overline{V}}$  as a parameter (figure 9). Estimates of  $\underline{V}$  were made using  $\underline{S}$  values derived by Hantush [1955] and values of  $\underline{T}$  were obtained for each test, using calculated values of  $\underline{\overline{B}}$ . Estimated transmissivities were logarithmically averaged for each township and are shown in tables D-24, D-25, and D-26.

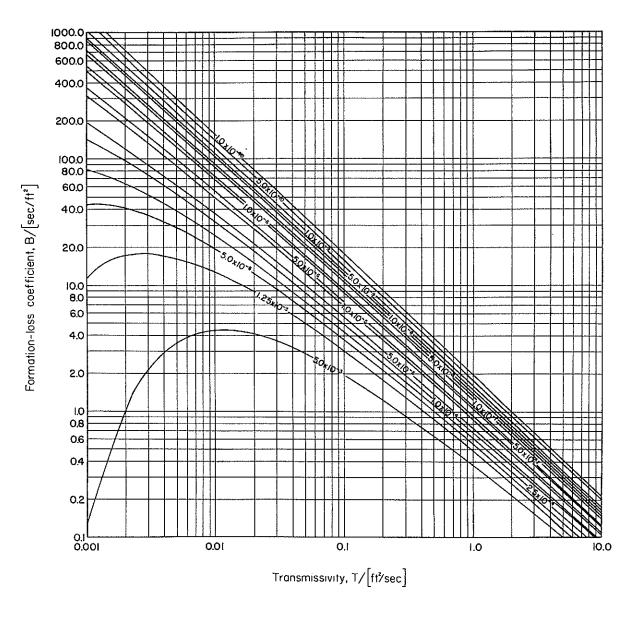
The accuracy of the calculated transmissivities depends upon the validity of the assumptions stated earlier. Since the tests were of short duration the effects of leakage are estimated to be negligible in both the aquifers except in the Roswell area of the principal confined aquifer.

When leakage is not negligible, the formation-loss coefficient  $\underline{B}$  is given by the following equation:

$$B(r_{W},t) = \frac{1}{4\pi t} W(u_{W},\beta)$$
 (2a)

where β is the well radius over the leakage factor and W(u,β) is the well function for leaky confined aquifers [see, for example, Walton, 1970]. For the average values of the hydraulic characteristics (table 4) of the principal confined aquifer in the Roswell area, the well function for nonleaky aquifers is about 5 percent higher than the corresponding value of the well function for leaky aquifers. This difference is about 25 percent for the extremely high transmissivities of the principal confined aquifer in the Roswell area (see table D-27). Because of these differences in the values of the two well functions, some of the calculated transmissivities for the principal confined aquifer in the Roswell area are too low.

The effects of partial penetration of the wells are negligible in all the aquifers. This conclusion is based on the criterion given by Hantush [1964] for the effect of partial penetration of wells on the drawdown at the face of a well. The criterion is that if the duration of the test  $\underline{t}$  is less than  $Sb(1-\ell/2b)^2/5K$ , then the effect of partial penetration is negligible. In this expression,  $\underline{S}$  is the storativity,  $\underline{b}$  is the thickness of the aquifer,  $\underline{K}_{Z}$  is the vertical hydraulic conductivity, and  $\ell$  is the



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Figure 9. Curves of formation-loss coefficient versus transmissivity for different values of parameter  $V = r_W^2 S/4t\,.$ 

Table  $\mu_{ullet}$  Average values of aquifer characteristics

		Confined Aquifer	uifer		Shallow Aquifer	Aquif	er
Area	T <sup>#</sup> (10 <sup>3</sup> ft <sup>2</sup> /day)	ν.	K'/b' (per day)	B (10 <sup>3</sup> ft)	K'/b' B T (per day) (10 <sup>3</sup> ft) (10 <sup>3</sup> ft <sup>2</sup> /day)	** **	B (10 <sup>3</sup> ft)
Roswell	187.0	1.0 x 10 <sup>-5</sup>		35.0	13.4	0.10	9.5
Dexter	10.0	5.0 x 10 <sup>-5</sup>		35.0	E	=	40.0
Artesia	20.1	$5.0 \times 10^{-5}$		25.0	E	E	20.0
Lakewood	8.8	1.0 x 10-4		26.0	E	=	36.0
Intake Area	10.0	5.0 x 10 <sup>-2</sup>					

L After Hantush [1955].

T, S, K' /b', and B denote transmissivity, storativity, leakance, and leakage factor, respectively. \*

<sup>\*\*</sup> The ultimate average specific yield is believed to be about 20 percent.

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length of penetration.

Using the average values of the quantities involved, the above expression is about three hours for the principal confined aquifer, and about one second for the shallow aquifer. Since the average duration for a step-drawdown test was about 1 to 2 hours, the shallow aquifer transmissivities calculated from such tests on partially penetrating wells are probably too low.

The errors resulting from assumption (2) are believed to be small because the drawdown varies as the logarithms of  $\underline{S}$ ,  $\underline{t}$ , and  $r_w$ .

The transmissivities calculated by this method are believed to be local transmissivities for the aquifer. The principal confined aquifer is known to be cavernous, thus giving rise to sudden changes in  $\underline{T}$  values from place to place.

## Storativities of the Aquifers

Storativities of the principal confined and the shallow aquifers were determined by Hantush [1955] from pumping tests at several locations in the basin and are shown in table 4. The average storativity of the shallow aquifer is about 10<sup>4</sup> times that of the principal confined aquifer. The storativity of the intake area is discussed in the next section.

# Hydraulic Characteristics of the Intake Area

Fiedler and Nye [1933] called the area lying just west of the pinch-out of the confining beds of the Roswell Basin and south of T. 9 S. the 'principal intake area'. The storativity of the principal confined aquifer is very small ( $\sim 10^{-5}$ ) in the confined part of the aquifer. If we assume that the aquifer should not be dewatered to the extent that the groundwater becomes unconfined in the valley area, then the storage in the valley part of the aquifer is negligible compared with the storage in the principal intake area where storativity is about 1,000 times higher and the aquifer is unconfined.

The water levels have been declining in the intake area since 1944 and all this water has been discharging across the western limit of the confined area. Hantush [1955] has given a solution for the intake-area boundary-value problem, treating it as a "slit"

$$s(x,t) = 814 \text{ q x/T } \left[\frac{e^{-u}x}{\sqrt{u}_x} - \sqrt{\pi} \operatorname{erfc}(\sqrt{u}_x)\right]$$
 (3)

where <u>s</u> is the drawdown, which is small compared with the original depth of flow; q = 0.045 gpd/ft, average discharge per unit length of the intake area; x = 2 miles, average distance of the observation wells from the slit;  $u_x = 1.87x^2S/tT$ ; erfc(x) = complementary error function  $= 1 - \frac{2}{\sqrt{\pi}} \int_{x}^{\infty} e^{-y^2} dy$ ; t = time in days.

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Method of Analysis. A method for the evaluation of aquifer characteristics is presented below and is based on the classical technique of least squares. The sum of the squares of differences between the observed drawdowns and the drawdowns calculated by using theoretical drawdown equations for the flow system under consideration is minimized, treating the aquifer parameters as decision variables. This becomes a problem of finding the minimum of a nonlinear objective function which is a function of several variables and, therefore, is a typical nonlinear programming problem. A brief discussion of nonlinear programming is given in chapter 3. The method can be applied to any flow system for which analytical expressions to calculate drawdowns corresponding to the observed drawdowns are known.

Expressing the problem in a mathematical form, one obtains  $_{\rm M}$ 

$$\theta(T,S,\beta) = \sum_{i=1}^{M} \sum_{j=1}^{N_i} \omega_{ij} [d_{ij} - s_{ij}(T,S,\beta)]^2$$
 (4)

where,

 $\theta(T,S,\beta)$  = the objective function to be minimized.

M, N = the number of observation wells, and the number of observations in each observation well, respectively. Note that N can be different for each well.

i, j = running indices for M and N, respectively.

d,  $s(T,S,\beta)$  = observed and calculated drawdowns, respectively.

T = transmissivity of the aquifer in the area.

S = storativity of the aquifer in the area.

 $\beta$  = K'/b', leakance or leakage coefficient; K' and b' are hydraulic conductivity and thickness of the aquitard. Leakance is zero for non-leaky flow systems.

Leakance is zero for non-leaky flow systems.

wij = weight assigned to the jth observation in the ith

Well. The values of adulter parameters whi

The values of aquifer parameters which minimize the objective function are the optimum values.

The objective function defined by (4) is for flow systems where  $\underline{T}$ ,  $\underline{S}$ , and K'/b' are to be determined. If the drawdown is dependent on parameters other than the above three, the other parameters can also be considered as components of the set comprising the decision variables, and these parameters can be appropriately expressed in the analytical expression for the drawdown.

The method was applied to analyze four pumping tests in nonleaky, leaky, and anisotropic nonleaky flow systems. The results are compared with those obtained by classical methods and are given in table 5. All observations were assigned equal weights. Contours of the objective function  $\theta$  (sum of the square of deviations), as a function of aquifer parameters, for the test at Arrowsmith, Illinois, are shown in figure 10. The aquifer parameters obtained both by the classical type-curve method and by the new computer method are marked on the figure.

Application to the Intake Area. The computer method described in the preceding section was applied to obtain the best estimates of average aquifer parameters, namely storativity and transmissivity. The objective function was defined as in equation (4), where  $\beta$  is zero and  $\underline{s}$  is given by (3). The observed drawdowns were used in the wells on which the Pecos Valley Artesian Conservancy District (PVACD) maintains recorders, and in the U. S. Geological Survey wells in the intake area. These observation wells are located at different distances from the "slit". The maximum value for time was 20 years and maximum value for the distance from the "slit" was 16 miles.

The average values calculated by the computer method for storativity and transmissivity in the intake area are 0.025 or 2.5 percent and 8,700 ft $^2$ /day, respectively. Theis [1951] calculated a value of 0.05 or 5 percent for the intake area near Hondo Reservoir. Hantush [1955] used a value of 10,000 ft $^2$ /day for transmissivity in the intake area.

Storativity and transmissivity values calculated by the above methods are based on the assumption that equation (3) describes the drawdown in the intake area. However, in the derivation of equation (3), the discharge was assumed to be constant, which may not be strictly true. Moreover, the average value for q of 0.045 gpd/ft may be off somewhat, and therefore the calculated values for the parameters of the intake area may also be off.

## Recharge to the Aquifers

Recharge to the principal confined aquifer in the Roswell Basin is derived from precipitation on the outcrop area of the San Andres Limestone and on the adjacent tributary drainage. Additional replenishment comes from upward leakage from the underlying formations, namely the Glorieta Sandstone and the Yeso Formation. The main sources of replenishment to the shallow aquifer are (1) local precipitation, (2) surface drainage and irrigation losses, and (3) upward leakage from the principal confined aquifer.

Pumping test analysis by graphical and computer methods. Table 5.

		Parameter Values	/alues		
Type of System	Aquifer Parameter	Graphical	Computer	Difference in %	Location of Test and Author
NonLeaky	I in ft <sup>2</sup> /sec	.02430	.02289	6,16	Arrowsmith, McLean Co.
Confined	S	2.54 x 10 <sup>-3</sup>	3.352 x 10 <sup>-3</sup>	24.22	Bruin and Hudson (1958)
Nonleaky	T in ft²/sec	.01563	.01534	1,89	Gridley, McLean Co.
Confined	ဖ	$2.0 \times 10^{-5}$	2.09 x 10 <sup>-5</sup>	4,30	Valton (1962)
Leakv	T in ft <sup>2</sup> /sec	,002337	.002786	16,12	Dieterich, Effingham Co.
Confined	S r/B*	2.0 × 10-4 0.220	1.8 x 10-4 0.155	11,11 41,94	Illinois. Walton (1962)
Anisotropic	Txx in ft <sup>2</sup> /sec	.02691	.02731	1,46	
Nonleaky	$T_{ m yy}$ in ft $^2/{ m sec}$	.02691	.02750	2,15	Papadopulos
Confined	T <sub>xy</sub> in ft <sup>2</sup> /sec S	01615 1.00 x 10 <sup>-4</sup>	01696 0.9778 x 10"4	4.82	(1965)

\* B is leakage factor =  $\sqrt{T/(K'/b^{\dagger})}$  and r is the distance from the pumped well,

<sup>\*\*</sup>  $T_{xx}$ ,  $T_{yy}$ , and  $T_{xy}$  are components of the transmissivity tensor.

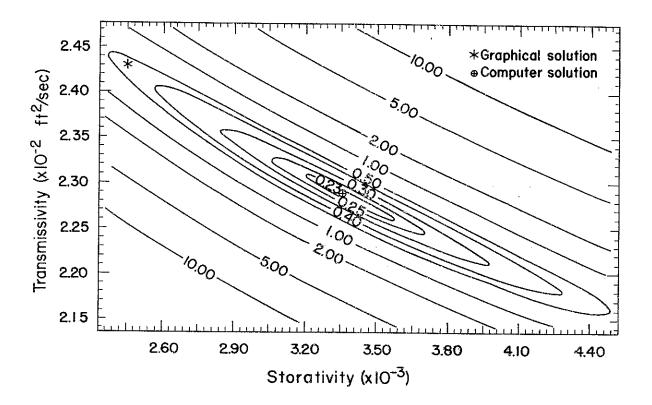


Figure 10. Objective function for pumping test at Arrowsmith, Illinois.

## Recharge to the Principal Confined Aquifer

An aquifer is in a steady state when its inflow equals outflow. The inflow or recharge to the principal confined aquifer when it is in a steady state can be estimated from its outflow, which is equal to pumpage plus leakage to the shallow aquifer. Hantush [1955] considered the flow in the aquifer to be in "dynamic equilibrium" during the three years 1928, 1936 and 1944. He estimated the leakages and then calculated the recharge during these years.

Jacob [1944] defined effective precipitation as "the rate of precipitation which, had it been maintained uninterruptedly throughout the past, would have produced the same water-table profile as actually existed at that particular time." Following Jacob, Hantush [1955] calculated the effective average rate of precipitation by

$$\overline{R}_{n} = \sum_{i=1}^{k} \frac{2(k+1-i)}{k(k+1)} R_{(n+1-i)}$$
 (5)

where

 $\overline{R}_n$  = the effective average rate of precipitation at the end of the  $n^{th}$  year, inches per year

R = annual precipitation during any year, inches per year

k = the number of years the rainfall of a given year
is effective.

Correlating the effective precipitation with the recharge during the three years mentioned above, he obtained the following for the recharge to the principal artesian aquifer:

Recharge = 21,000 (acre-feet/inch) 
$$\overline{R}_n$$
 (6)

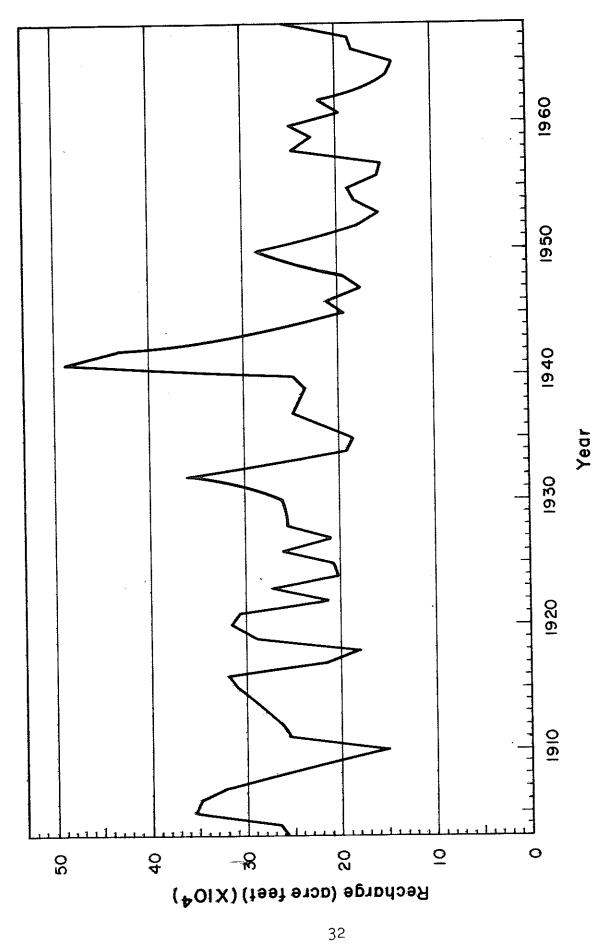
where the recharge during any year is in acre-feet, and  $\overline{\mathbb{R}}_n$  is the three-year effective average rate of precipitation in inches computed by (5), using the annual average rainfalls at Roswell and Artesia. Justification for using three-year effective average rainfall is given in Appendix B.

The values of annual recharge to the principal confined aquifer were calculated for the period 1903-1968 and are tabulated in ascending order in table 6, and graphed in chronological order in figure 11. The average rate of recharge for the Roswell Basin in this table is 240,000 acrefeet per year.

Table 6. Three-year effective average rainfall (inches) at Roswell and Artesia, New Mexico, with recharge to principal confined aquifer and recharge probabilities, in ascending order.

m	Year	Rainfall $\overline{R}_3$ (inches)	Recharge** (acre-ft)	m(*) 1+n	m	Year	Rainfall $\overline{R}_3$ (inches)	Recharge (acre-ft)	m 1+n
1	1965	6.638	139400	.0149	34	1937	11.753	246800	.5075
2	1964	7.001	147000	.0299	35	1944	11.763	247000	.5224
3	1910	7.093	149000	.0448	36	1949	11.803	247900	.5373
4	1957	7.222	151700	.0597	37	1958	11.811	248000	.5522
5	1953	7.418	155800	.0746	38	1960	11.898	249800	.5672
6	1956	7.481	157100	.0896	39	1968	11.917	250300	.5821
7	1963	8.153	171200	.1045	40	1911	12.034	252700	.5970
8	1947	8.288	174100	.1194	41	1928	12,141	255000	.6119
9	1918	8.432	177100	.1343	42	1903	12.158	255300	.6269
10	1952	8.538	179300	.1493	43	1929	12.195	256100	.6418
11	1954	8.599	180600	.1642	44	1908	12.345	259200	.6567
12	1966	8.693	182500	.1791	45	1930	12.403	260500	.6716
13	1935	8.759	183900	.1940	46	1912	12.428	261000	.6866
14	1967	8.839	185600	.2090	47	1926	12.452	261500	.7015
15	1955	8.996	188900	.2239	48	1904	12,595	264500	.7164
16	1945	9.096	191000	.2388	49	1923	12.989	272800	.7313
17	1934	9.159	192300	.2537	50	1933	13.121	275500	.7463
18	1948	9.246	194200	.2687	51	1913	13.155	276300	.7612
19	1961	9.362	196600	.2836	52	1950	13.517	283900	.7761
20	1924	9.619	202000	. 2985	53	1914	13.840	290600	.7910
21	1925	9.856	207000	.3134	54	1919	13.840	290600	.8060
22	1927	9.959	209100	.3284	55	1931	13.962	293200	.8209
23	1909	9.991	209800	.3433	56	1921	14.593	306400	.8358
24	1946	10.083	211700	. 3582	57	1915	14.698	308700	.8507
25	1922	10.096	212000	.3731	58	1920	15.003	315100	.8657
26	1917	10.148	213100	.3881	59	1943	15.152	318200	.8806
27	1936	10.396	218300	.4030	60	1916	15.203	319300	.8955
28	1962	10.428	219000	.4179	61	1907	15.288	321000	.9104
29	1951	10.630	223200	.4328	62	1906	16.578	348100	.9254
30	1959	10.726	225200	.4478	63	1905	16.870	354300	.9403
31	1939	11.133	233800	.4627	64	1932	17.228	361800	.9552
32	1938	11.428	240000	.4776	65	1942	20.528	431100	.9701
33	1940	11.701	245 700	.4925	66	1941	23.298	489200	.9851

<sup>(\*)</sup> n = 66.



Recharge to the principal confined aquifer, Foswell, New Mexico. Figure 11.

## Replenishment to the Shallow Aquifer

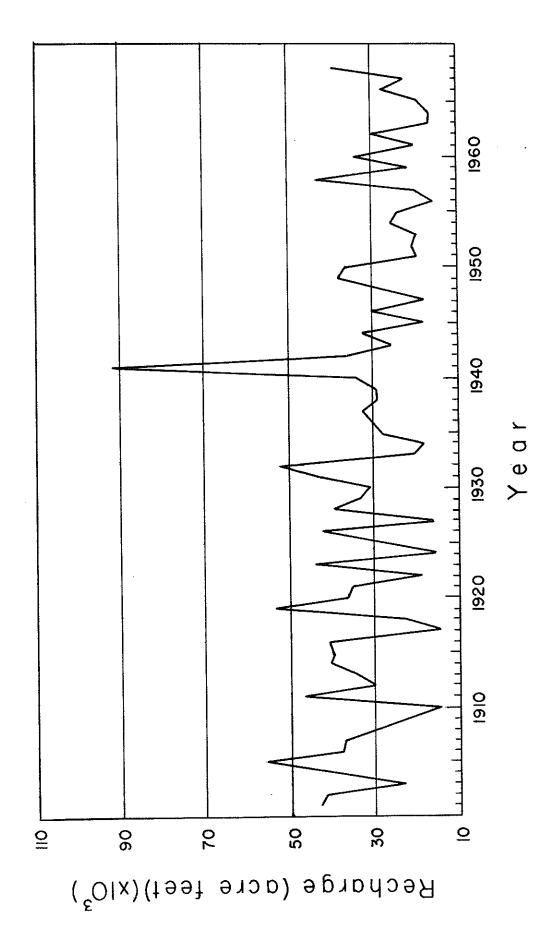
Local Precipitation. The average rate of precipitation at Roswell and Artesia for 1905-1968 is 11.40 inches per year. Of the total precipitation on the alluvium, part runs off through the surface drainage, part is lost by evapotranspiration, and the remainder percolates to the shallow aguifer. By comparison with the recharge estimate made by Theis [1937] for the High Plains, which lie just east of the Roswell Basin, Morgan [1938] deduced that the recharge averages somewhat less than one-half inch per year or about 530 acrefect for each one-mile strip across the basin. The average rate of recharge to the shallow aguifer from precipitation percolation is thus about 28,000 acre-feet per year. The average of yearly precipitations from the Roswell and Artesia stations of the U. S. Weather Eureau is shown in table 1 and the recharge is shown in figure 12.

Surface Drainage and Irrigation Losses. The Poswell Dasin is crossed by numerous ephemeral streams which head in the mountains to the west. During the summer months, when most of the rain falls, the streams flow and part of the flow percolates down to the groundwater reservoir.

The recharge from irrigation is made up of seepage from irrigated lands, from reservoirs, and from ditches. The amount of recharge from surface drainage and from irrigation losses has not been previously estimated. Yearly irrigation water requirements for different crops and yearly irrigation efficiencies were estimated for 26 years (see table 18) in the Roswell Basin. From this information and from the information given by Blaney and Hanson [1965] it is assumed that the replenishment is about 30 percent of the total amount of irrigation water pumped. The total amounts of water used for irrigation in the Roswell Basin during 1967 and 1968 were 388,000 and 339,000 acre-feet, respectively. We thus get estimates of replenishment of 116,000 and 102,000 acre-feet during 1967 and 1968, respectively.

#### Leakage

Leakage from one aquifer to the other in the coupled leaky-aquifer system of the Roswell Basin is a function of both time and space. The direction of leakage is determined by the difference in the hydraulic heads of the two aquifers, the direction of leakage being from the aquifer with higher head to the aquifer with lower head. Leakage is directly proportional to the product of the difference in the heads of the two main aquifers and the leakance of the aquitard in a given area.



Recharge (computed as proportional to the precipitation) from local precipitation to the shallow aquifer, Roswell Basin, New Mexico. Figure 12.

## $Q = K'A\Delta h/b'$ (7.)

where Q is the volumetric rate of leakage through a vertical column of the shallow confined aquifer with a basal area  $\underline{A}$ ,  $\underline{A}h$  is the difference between the average heads of the two main aquifers in area  $\underline{A}$ , and K'/b' is the leakance or leakage coefficient or the ratio of the hydraulic conductivity (K') of the aquitard to its thickness (b') in the area under consideration.

Equation (7) was used to calculate leakage for January of the six years for which water table and potentiometric surface maps were constructed. The January leakage and the leakage coefficients used to calculate leakage are shown in table 7. The leakage was positive in all cases. This is because normally the average January elevation of the potentiometric surface in the principal confined aquifer is higher than the average January elevation of the water table in the shallow aquifer. However, this is not necessarily true during the pumping season, and the direction of leakage is then often reversed.

Referring to table 7, the January leakage is highest for 1944 and lowest for 1964. The leakages for 1944 and 1969 are slightly higher than for 1926 and 1964, respectively. Any decrease or increase in leakage between two years is because the potentiometric surface declines or recovers, respectively, at a faster rate than the water table. Elevation of the potentiometric surface changes from one year to another at a higher rate than that of the water table, mainly owing to two reasons: (1) the yearly pumpage from the principal confined aquifer is greater than the yearly pumpage from the shallow aquifer, and (2) the hydraulic diffusivity of the principal confined aquifer is much larger (about 105 times) than the hydraulic diffusivity of the shallow aquifer.

Monthly values of water levels in the Roswell Basin are unavailable and therefore the yearly net leakage could not be calculated by equation (7). However, the yearly net leakage has been estimated by the water budget method in a later section. Yearly net leakage should not be computed from the January leakage simply by multiplying by 12.

#### Natural Discharge

The natural discharge from the coupled leaky aquifers and the Pecos River system of the Roswell Basin has three main components: (1) base flow, (2) flow from springs, and (3) consumptive use by phreatophytes. All components are discussed below.

Table 7. Leakage from principal confined aquifer to shallow aquifer during January of different years (thousands of acre-feet), Roswell Basin, New Mexico.

Area	Leakance <sup>1</sup> (per month)	1926		Year 1954	1964	1969
Roswell Townships 10, 11	4.5 x 10 <sup>-3</sup>	10.7	12.4	8.8	5.2	5.6
Dexter Twps. 12, 13, 14	2.5 x 10 <sup>-4</sup>	1.1	1.2	1.2	0.8	1.3
Artesia Twps. 15 and 16	9.0 x 10 <sup>-4</sup>	2.0	2.6	1.1	0.6	1.8
Lakewood Twps. 17, 18, 19, 20	3.9 x 10 <sup>-4</sup>	3.0	3.2	1.5	0.2	1.0
Total		16.8	19.4	12.6	6.8	9.7

<sup>1</sup> Hantush [1955].

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#### Base Flow

The Pecos River gains water as it flows through the Roswell Basin, as is apparent from the record of gaging stations at Acme, just north of the basin, and at Artesia in the southern part of the basin (see tables 8 and 9).

The base flow into the Pecos River between Acme and Artesia is estimated from the analysis of hydrographs (obtained by plotting daily stream flows versus day of the year on semi-log paper) at Artesia, minus those at Acme with a suitable time lag. The estimated monthly base flows and yearly totals since 1919 are shown in table 2.

The base flow has been decreasing steadily since the development of the shallow aquifer in 1938, except during 1941 to 1944 when it increased because of heavy rains. The average yearly base flow for 1919-1968 is 51,000 acre-feet. The rate of base flow is least during the pumping season and increases during the winter months. The monthly averages show that during July and January the Pecos River receives the smallest average amount (2,300 acre-feet) and the highest average amount of base flow (6,600 acre-feet), respectively.

## Spring Flow

Before the development of the basin there were a few artesian springs in the northern part of the basin near Roswell, and the Major Johnson Springs, which still flow, at the southern tip of the basin. Besides these there reportedly were some springs in the lower reaches of Penasco and Felix Creeks. Information on the springs has been gathered by the U. S. Geological Survey [Fiedler and Nye, 1933] and is described here.

Berrendo Springs. Berrendo Springs originally flowed 65 cubic feet per second, or 47,060 acre-feet per year from three springs. Each was estimated to have an equal flow of about 21.7 cfs (15,790 acre-feet per year)in 1900.

North Berrendo Springs. (NE $\frac{1}{1}$ , SW $\frac{1}{1}$ , SE $\frac{1}{1}$ , Section 5, T. 10 S., R. 24 E.) The flow of North Berrendo Spring decreased to 5 cfs (3,620 acre-feet per year) by 1926 and stopped by 1932.

Middle Berrendo Spring. (SW $\frac{1}{4}$ , SW $\frac{1}{4}$ , NE $\frac{1}{4}$ , Sec. 17, T. 10 S., R. 24 E.) The flow of Middle Berrendo Spring decreased to only 3 cfs (2,172 acre-feet per year) by 1926 and ceased by 1932.

South Berrendo Spring. (SW $_{\overline{4}}$ , SW $_{\overline{4}}$ , NE $_{\overline{4}}$ , Sec. 17, T. 10 S., R. 24 E.) South Berrendo Spring stopped flowing by 1926.

Discharge (in thousands of acre-feet) of Pecos River near Acme, New Mexico. Table 8.

Total	107.5 138.1 124.7	876.5 406.8 120.7 98.4 77.7	83.4 55.4 74.8 164.4 156.5	110.4 96.4 73.2 127.2 153.1	85.9 85.9 225.1 99.5 218.1	121.2 108.7 118.2 40.9 76.2	118.6 83.7 76.2
Dec.	1.5	14.5 7.8 2.7 1.4 .6	1.1 0 0 2.2	1.0 9.0 9.0	2 5.5 1.7 7.6	12.7 1.1  23	. 4.
Nov.	1.0	30.1 51.0 2.3 2.4 1.4	2.3 0 .3 3.9 12.4	1.1 .4 .3 3.2 1.5	2. 2. 6. 6. 6. 6. 6. 6. 6. 6. 6. 6. 6. 6. 6.	e. 1 8.1 8.2	1.0
Oct.	2.1	135.3 12.2 2.2 6.9 2.3	3.8 0 0 21.6 12.4	1.2 .9 .1 71.8 13.2	2.8 6.0 .8 .8	2.6 .9 .0 .0	∞. 4. ∞.
Sept.	14.3 21.3 17.0	209.9 138.2 17.5 8.9 14.1	40.8 0 0 48.4 9.2	1.3 6.1 69.6	1.7 12.8 33.6 .5	1.8 17.9 7.0 7.0 2.6	6.2
Aug.	15.1 11.4 17.6	44.6 16.4 29.1 2.8 5.4	.7 .1 16.2 13.2 11.6	5.5 33.5 27.6 14.0 2.0	33.9 29.3 31.6 31.3	21.6 7.4 25.9 .2	33.2 23.9 3.4
July	12.5 19.4 11.2	69.2 13.7 4.4 8.4	9.5 23.3 19.5 13.4 52.0	35.0 17.8 14.4 0	24.6 14.4 7.7 28.2 99.0	38.7 10.5 29.0 13.8	16.3 1.7 19.5
June	33.1 23.3 19.4	130.1 14.8 29.7 30.8 16.1	.3 0 3.8 26.5 37.7	19.5 14.9 .2 .7 18.9	14.6 4.0 24.6 4.5 16.2	19.2 30.4 24.9 .6	30.8 4.0 16.1
May	.2 24.1 6.8	164.8 51.8 1.3 .9	.1 4.5 7.0 5.7	13.5 .1 .3 16.0 5.0	4.4 4.5 65.6 19.5 23.2	7.2 7.1 7.1 3	.4 49.2 .9
Apr.	.7 11.0 19.4	40.7 72.4 12.1 1.4 35.1	23.2 18.5 33.7 27.2 12.8	4.8 24.3 21.0 19.0	.6 .4 25.2 1.5	4.8 1.8 1.2 21.4 2.9	1.6
Mar.	18.7 22.0 29.3	36.5 3.6 6.9 31.1	8.0 0 0.0 3.9	26.3 3.0 2.9 1.2	.8 11.3 19.0 2.2 18.1	4.4 27.1 24.8 .7 21.9	26.9 .5 24.3
Feb.	4.0.7.	.2 12.9 1.6 1.0	1. 4. 4. 4. 4. 4. 4. 4. 4. 4. 4. 4. 4. 4.	A00A	1.3	6.0 1.1 1.2 .7	4.6.2.
Jan.	1.1	.2 11.6 10.3 1.8	0.0 0.0 1.8	8.2.4.4.8.	1.0 .3 .7 1.3	7.6 1.9 5.1 1.	1. 6.
Year	1938 1939 1940	1941 1942 1943 1944 1945	1946 1947 1948 1949 1950	1951 1952 1953 1954	1956 1957 1958 1959	1961 1962 1963 1964	1966 1967 1968

281.4 159.6 243.3 182.4 261.6 149.7 123.2 289.5 204.9 250.7 265.0 136.5 180.8 754.4 262.9 215.2 360.1 176.0 93.7 190.5 275.0 198.0 215.7 374.0 493.2 281.1 270.3 239.7 137.2 193.5 Total acre-feet) of Peccs River near Artesia, New Mexico. 19.4 17.3 9.3 8.9 12.3 17.2 8.0 18.0 11.6 12.1 10.1 30.9 15.8 18.3 11.7 19.1 23.4 14.3 18.9 15.2 18.9 22.3 36.8 26.1 22.3 22.3 14.4 14.2 18.7 9.8 8.8 9.1 9.5 8.6 27.1 9.2 15.5 7.4 14.4 23.4 12.4 13.5 4.2 26.7 26.7 14.1 23.3 24.9 9.2 7.9 8.9 19.5 7.5 11.7 18.0 10.4 Nov. 16.5 6.6 56.6 16.0 5.9 3.8 97.1 8.8 17.0 18.5 62.6 8.2 4.2 7.1 111.7 27.4 2.8 6.8 6.8 17.2 5.3 33.2 55.7 5.7 22.0 9.8 13.3 18.5 12.5 109.9 21.3 12.6. 29.1 Sept. 13.1 4.8 16.8 4.3 47.2 34.2 19.3 14.8 24.6 6.3 21.9 21.1 10.6 186.6 12.0 12.3 6.9 7.3 13.3 7.0 16.1 16.1 28.3 8.4 6.8 41.8 28.6 22.3 19.8 34.5 9.4 39.0 10.1 67.6 2.5 18.1 9.4 74.1 48.3 26.1 31.3 36.3 14.8 21.8 9.7 5.3 29.3 34.8 16.6 20.6 95.9 10.9 87.9 Aug 10.4 22.2 24.5 .6 .6 2.7 2.3 4.6 84.6 18.5 70.7 13.5 9.9 28.0 43.5 32.2 13.9 17.8 9.2 July 39.5 28.5 29.4 10.1 3.9 90.4 9.8 18.0 81.2 42.3 16.2 17.8 16.6 3.5 19.8 .69.0 29.2 19.0 21.3 3.8 48.0 15.8 16.0 16.3 22.2 13.2 4.6 15.5 70.8 44.8 June 10.9 52.5 80.3 44.8 24.5 44.8 2.7 7.8 7.8 60.9 7.4 33.1 22.6 9.6 25.0 30.5 5.9 102.0 40.6 49.7 7.9 6.6 57.2 41.2 55.8 18.1 12.0 29.3 4.3 38.5 21.6 8.5 3.4 10.5 Discharge (in thousands of May 17.7 6.6 4.4 5.6 5.6 24.7 14.0 4.4 7.5 3.5 26.4 5.3 5.8 5.8 40.9 9.6 5.6 8.2 19.9 23.3 31.5 12.5 6.5 2.9 4.8 Apr. 9.7 10.0 8.6 9.6 9.6 10.8 8.0 6.7 11.6 9.8 11.7 9.8 9.4 6.9 11.9 8.5 4.8 6.7 6.7 9.9 11.7 12.2 12.7 5.7 11.1 10.3 8.6 9.8 20.3 14.9 9.0 7.4 139.2 13.6 Mar. 13.8 13.3 13.3 9.7 12.9 11.2 14.9 17.1 9.3 9.8 11.4 9.2 13.8 9.3 19.0 21.9 15.9 10.3 16.7 14.2 12.5 13.1 20.6 14.1 13.8 16.8 16.8 14.1 Feb. 16.6 17.5 16.8 10.8 9.0 13.7 16.4 11.2 14.4 13.6 27.0 28.8 22.9 21.2 18.3 19.2 16.6 21.1 16.8 24.3 20.3 21.5 19.7 23.2 26.7 17.5 14.4 11.6 11.6 25.8 16.8 Jan. . ص [able 1926 1927 1928 1929 1930 1931 1932 1933 1934 1935 1921 1922 1923 1924 1925 Year 1911 1912 1913 1914 1915 1916 1917 1918 1919 1920 1906 1907 1908 1909 1910

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Discharge (in thousands of acre-feet) of Pecos River near Artesia, New Mexico. (continued) Table 9.

202.9 581.6 175.4 189.6 179.8	1,351.0 511.7 183.9 155.8 114.1	146.0 90.6 127.7 248.2 191.5	128.1 106.6 77.8 239.6 191.9	96.4 93.5 244.8 105.1	131.2 123.5 116.8 44.1 87.9 141.0 83.4
17.1 8.3 7.0 7.2 6.4	37.7 20.0 11.8 9.2 6.2	დ ი ღ ი ღ ი ი ღ ი ღ ი ი ი ღ ი	2.4 9.5 6.8 6.6 7.6	3.6 4.1 8.7 4.1 12.1	6.3 4.1 2.1 2.7 2.5 2.5
11.2 8.9 8.4 5.0 5.7	73.7 62.2 7.5 9.3 6.6	10.0 4.4 5.1 10.7 5.6	5.8 3.5 3.3 12.4 6.0	4.2 4.3 8.9 3.5 10.6	3.5 2.1 2.1 2.3 1.8 2.2
15.8 8.7 17.1 5.3	258.4 23.8 6.1 14.8 7.2	17.2 2.0 2.6 25.8 14.2	2.4 2.7 1.3 152.7 32.1	1.1 5.4 12.4 1.5 26.0	2.1 3.9 1.3 2.7 2.7
29.8 25.8 24.5 20.6 17.8	339.4 134.3 17.9 12.2 17.1	47.6 .5 1.1 55.0 15.5	1.1 1.1 9.0 2.0 60.8	3.8 9.8 30.3 .7	1.9 20.7 9.4 2.6 12.5 1.0
7.5 12.8 13.6 19.9 23.3	54.1 18.3 27.8 4.2 2.5	2.6 .4 18.3 20.0 15.4	3.5 30.5 18.5 19.8 4.9	26.7 27.5 27.4 27.7 12.1	20.9 8.6 22.9 0 6.8 46.0 21.6
32.1 38.3 22.5 23.8 13.3	89.3 17.5 15.8 9.9 3.0	10.0 20.6 15.9 35.1 64.1	37.6 18.4 9.4 .1	24.0 10.6 11.6 28.4 89.3	31.2 10.8 21.7 10.0 32.9 17.4 1.2
26.6 208.0 35.5 24.3 22.9	150.2 19.1 31.8 30.5	5.0 .9 28.0 40.0 26.6	14.8 9.7 1.4 .9	9.4 5.4 23.2 4.0 20.0	18.7 29.9 20.8 3.4 8.5 24.1 5.8
21.5 182.7 3.6 27.5 20.6	235.8 67.1 5.1 4.5 3.7	2.8 10.5 4.3 11.4 4.5	16.6 2.4 2.1 19.1 6.8	5.2 6.6 70.7 15.9 18.7	7.3 2.7 2.7 1.1 4.0 1.4
3.5 30.2 4.9 15.9	54.4 76.9 19.1 5.4 34.6	23.9 23.1 31.6 26.5 13.5	5.7 22.5 19.7 15.6 3.2	2.5 1.9 22.3 2.4 7.1	5.4 2.8 17.8 5.3 3.2 8.5
9.3 20.2 23.3 25.1	47.2 13.6 9.1 34.4 4.9	4.3 3.3 6.6	24.1 3.5 3.0 2.4 4.4	3.7 10.9 20.7 3.8 15.5	7.4 25.3 22.1 1.8 17.3 23.3 1.4
2.7 19.4 6.6 6.7 6.5	5.1 27.9 9.7 8.4 6.0	5.3 4.7 5.6 8.7	4, 2, 2, 4, 2, 2, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3,	6.6 3.9 6.5 4.0	9.6 3.7 2.6 1.6 2.7 3.1
15.2 17.9 7.8 8.2 7.1	5.6 30.6 21.6 12.7 8.1	6.9 8.8 4.9 5.7 10.8	5.3 3.2 3.2 7.2	5.1 4.5 6.0 5.3	12.4 5.4 4.1 2.7 2.1 2.6 3.8
1936 1937 1938 1939	1941 1942 1943 1944 1945	1946 1947 1948 1949 1950	1951 1952 1953 1954 1955	1956 1957 1958 1959 1960	1961 1962 1963 1964 1965 1966

A Section of the sect

North Spring.  $(NW_{\overline{4}}^{1}, SE_{\overline{4}}^{1}, NE_{\overline{4}}^{1}, Sec. 36, T. 10 S., R. 23 E.)$  North Spring was originally flowing at the rate of 85 cfs (61,540 acre-feet per year) and the flow decreased to 77 cfs (55,748 acre-feet per year) in 1901 and completely stopped by 1926.

South Spring. (SE $\frac{1}{4}$ , SE $\frac{1}{4}$ , NE $\frac{1}{4}$ , Sec. 22, T. 11 S., R. 24 E.) South Spring had an original flow of 60 cfs (43,440 acre-feet per year) which diminished to 28 cfs (20,272 acrefeet per year) in 1902 and stopped completely by 1904.

Major Johnson Springs. (NE $\frac{1}{4}$ , NE $\frac{1}{4}$ , NW $\frac{1}{4}$ , Sec. 21, T. 20 S., R. 25 E.) Major Johnson Springs have been studied by Theis [1938] and Reeder [1963]. The springs discharged 40 cfs (28,960 acre-feet per year) until 1938, when the discharge started to diminish. The rate of flow in 1964 was 10 cfs (7,240 acre-feet per year) [Cox, 1967].

## Consumptive Use by Salt Cedars

Salt cedars were first observed in the basin near Lake McMillan in 1914. In the beginning they were welcome because they helped reduce the transport of sediments into the lake. Starting with about 500 acres in 1915, in the middle basin of the Pecos River, the area covered by salt cedar increased to 15,000 acres by 1939, 25,000 acres by 1946 and 40,000 acres by 1957 [Thompson, 1959].

Salt cedars are located mainly along the Pecos River channel and they consume mostly the shallow groundwater. The amounts of water consumed by salt cedars in the Roswell Basin during 1966, 1967, and 1968 are estimated to be 121,000, 132,000, and 107,000 acre-feet, respectively. The amount of water consumed during 1968 has decreased from the preceding two years partly because of increase in precipitation and partly because of the eradication program of the Bureau of Reclamation. Yearly estimates of consumptive use by salt cedars were based on data provided by the Bureau of Reclamation [Smith, personal communication, 1969, and U. S. Bureau of Reclamation, 1966], and are shown in table 10.

The estimates of salt cedar acreages given in table 10 are lower than the estimates of the National Planning Board [1942] and of Thompson [1959]. Therefore, it is believed that the estimates of salt cedar consumption in table 10 are probably low, and the discrepancy is believed to be greater for the earlier years.

#### Pumpage from the Aquifers

Groundwater is withdrawn in the basin mainly from the two principal aquifers: the shallow aquifer and the principal confined aquifer. Less than 8 percent of the groundwater

Table 10. Yearly estimates of consumptive use of water by salt cedars, Roswell Basin, New Mexico, 1937-1968.

<del></del>		. / \1		T. (21) <sup>2</sup>
Year	Salt Cedar		Consumptive	use (acre-reet)
icai	gross	K = 1.0	per acre	total
1937	9,100	7,400	3.86	28,500
1938	10,600	8,400	4.03	34,000
1939	11,800	9,400	4.00	37,400
1940	12,800	10,200	3.83	39,100
1941	13,800	11,100	1.46	16,200
1942	14,700	11,900	3.63	43,100
1943	15,600	12,700	4.18	53,100
1944	16,500	13,500	3.80	51,200
1945	17,300	14,300	4.51	64,300
1946	18,400	15,300	4.12	63,200
1947	19,000	15,800	4,49	70,900
1948	19,800	16,600	4.16	68,900
1949	20,600	17,300	3.68	63,700
1950	20,600	17,100	3.91	66,700
1951	22,100	18,800	4.55	85,400
1952	22,800	19,500	4.46	86,900
1953	23,600	20,200	4.57	92,400
1954	24,300	21,000	4.44	93,100
1955	25,000	21,700	4.34	94,000
1956	25,400	22,400	4.70	105,200
1957	26,400	23,100	4.52	104,200
1958	28,800	25,500	3.64	92,700
1959	27,800	24,500	4.48	109,500
1960	28,400	25,200	3.93	99,000
1961	29,100	25,800	4.43	114,400
1962	<b>29,8</b> 00	26,500	4.13	109,700
1963	30,400	27,200	4.67	127,100
1964	30,400	27,100	4.62	125,200
1965	31,700	28,600	4.56	130,300
1966	32,400	29,200	4.14	120,700
1967	33,000	29,900	4.42	132,000
1968	33,600	30,500	3.52	107,400

 $<sup>^{</sup>l}$  From the U. S. Bureau of Reclamation [1966].  $\underline{K}$  is the yearly consumptive-use coefficient.

 $<sup>^{2}</sup>$ Consumptive use minus effective precipitation.

pumped in the basin comes from the aquitard, also called in this report the shallow confined aquifer.

Data concerning the total number of wells tapping various aquifers and the amounts of water withdrawn from 1900 through 1968 are presented in tables 11 and 12, respectively. The rates of discharge for 748 wells were obtained from the well schedules which are available in the office of the State Engineer at Roswell. Tables 13 and 14 and figures 13 and 14 were prepared from this information, as well as from information about the metered pumpage for individual wells provided by the same office.

The total annual pumpage is distributed according to sources and periods as shown in table 15. These percentages are based on the history of the development of the basin [Welder, personal communication, 1968].

The yearly amounts of pumpage by source based on table 15 are shown in table 12. The pumpage estimates were derived by Mower [1960] and Welder [personal communication, 1968]. Table 12 is based on information in these sources as well as on metered jumpage for 1967 and 1968 provided by the office of the State Engineer in Roswell.

#### Use of Groundwater

Before large-scale development of the basin, ground-water in the Roswell Easin was used mainly from domestic and stock wells and the groundwater was not extensively exploited until large-scale irrigation was begun in the beginning of this century. The reader is referred to the publication of the National Resources Planning Board [1942] for the detailed distory of the basin's development.

During 1967 and 1968, only 4.3 percent and 4.6 percent, respectively, of the total groundwater pumped was used for municipal, commercial, and industrial purposes. More than 95 percent of the groundwater was, and still is being, utilized for farming.

## Irrigated Acreage

The Roswell Basin is located in Chaves County and in the northern part of Eddy County. Alfalfa and cotton are the major crops, followed by sorghum, and some small grains and commercial vegetables are also grown. During 1968, alfalfa, botton, and sorghum accounted for 95.4 percent of the total acreage in Chaves County. Tables 16 and 17 show yearly acreages of irrigated crops in Chaves and Eddy Counties, New Mexico, based on estimates by the U.S. Department of Agriculture and New Mexico Department of Agriculture.

Table 11. Average days of use and average discharge of wells tapping various aquifers, Roswell Basin, New Mexico, 1967.

Aquifer	Total Number of Wells	Number of Wells Analyzed	Average Days	Average Dis- charge(gpm)
1	814	473	82.6	1080
2	91	46	100.3	627
3	480	229	97.4	556
12	33	23	96.9	847
13	11	7	120.8	694
23	125	75	99.0	643
123	5	3	104.5	1007

Aquifer code numbers 1, 2, and 3 denote principal confined aquifer, shallow confined aquifer, and shallow aquifer, respectively. Two or three digits represent multiple aquifers.

Table 12. Annual pumpage by aquifer (in acre-feet), Roswell Basin, New Mexico, 1900-1968.

Year	Shallow Aquifer	Shallow Con- fined Aquifer	Principal Con- fined Aquifer	Total
1900-				-
1937	0	14,500	185,500	200,000
1938	93,960	23,300	185,740	303,000
39	98,920	24,530	195,550	319,000
40	97,990	24,300	193,710	316,000
.0	<b>,</b>	24,000	2, 3, 4, 2, 0	510,000
1941	58,610	14,530	115,860	189,000
42	105,430	26,150	208,420	340,000
43	114,740	28,450	226,810	370,000
44	106,670	26,450	210,870	344,000
45	125,900	31,220	248,880	406,000
1946	116,290	28,840	229,880	375,000
47	129,000	31,990	255,010	416,000
48	113,480	31,870	260,650	406,000
49	108,170	30,380		•
50	-		248,450	387,000
50	110,120	30,930	252,950	394,000
1951	133,600	37,520	306,880	478,000
52	124,940	35,090	286,970	447,000
53	129,690	36,420	297,890	464,000
54	131,920	37,050	303,020	472,000
55	125,780	35,330	288,900	450,000
1956	138,630	38,940	318,430	496,000
57	135,280	37,990	310,730	484,000
58	107,050	30,070	245,890	383,000
59	125,780	35,330	288,900	450,000
60	118,510	33,280	272,210	424,000
		-		
1961	125,500	35,250	288,260	449,000
62	137,730	36,500	290,760	465,000
63	150,770	39,960	318,280	509,000
64	158,760	42,080	335,160	536,000
65	132,400	35,090	279,510	447,000
1966	115,520	30,620	243,870	390,000
67	110,480	29,280	233,240	373,000
68	97,450	25,830	205,720	329,000
Mean 1938 <del>-</del> 1968	118,680	31,760	256,370	406,810

<sup>\*</sup> In acre-feet.

Table 13. Days of use of wells tapping various aquifers during 1967, Roswell Basin, New Mexico.

	Number	of Wells Analyzed	
Days	Principal Confined Aquifer	Shallow Confined Aquifer	Shallow Aquifer
0 to 20	20		4
21 to 40	62	5	26
41 to 60	70	9	36
61 to 80	101	5	36
81 to 100	83	4	36
101 to 120	52	6	28
more than 120	85	17	64
Total	473	46	229

Table 14. Discharge of wells tapping various aquifers, from the well schedules, Roswell Basin, New Mexico.

	Number	of Wells Ana	lyzed
Discharge (gpm)	Principal Confined Aquifer	Shallow Confined Aquifer	Shallow Aquifer
0 to 200	7	3	17
201 to 400	25	8	69
401 to 600	45	13	53
601 to 800	57	13	43
801 to 1,000	77	5	23
1,001 to 1,200	80	2	11
1,201 to 1,400	62	2	9
1,401 to 1,600	34		1
1,601 to 1,800	38		2
1,801 to 2,000	26		1
More than 2,000	22		
Total	473	46	229

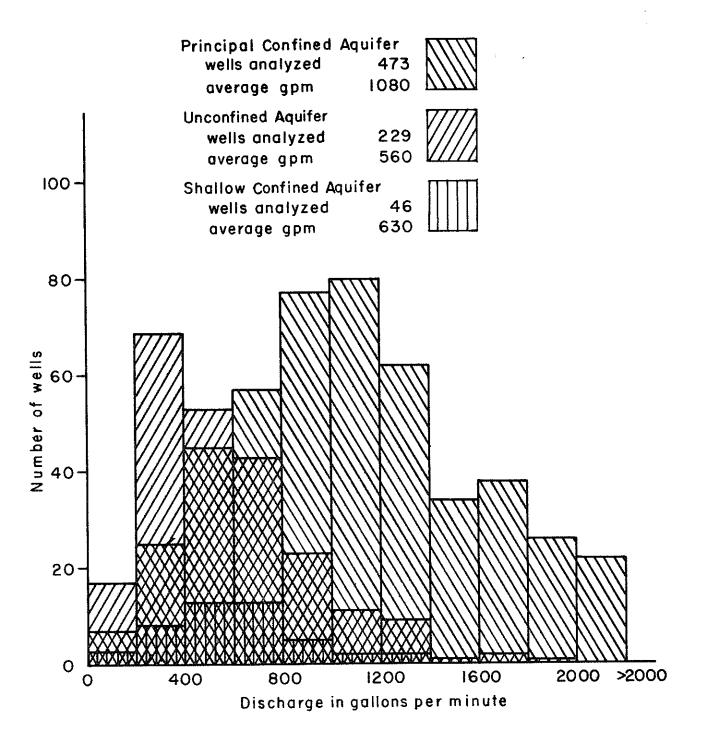


Figure 13. Discharge of wells tapping various aquifers, Roswell Basin, New Mexico.

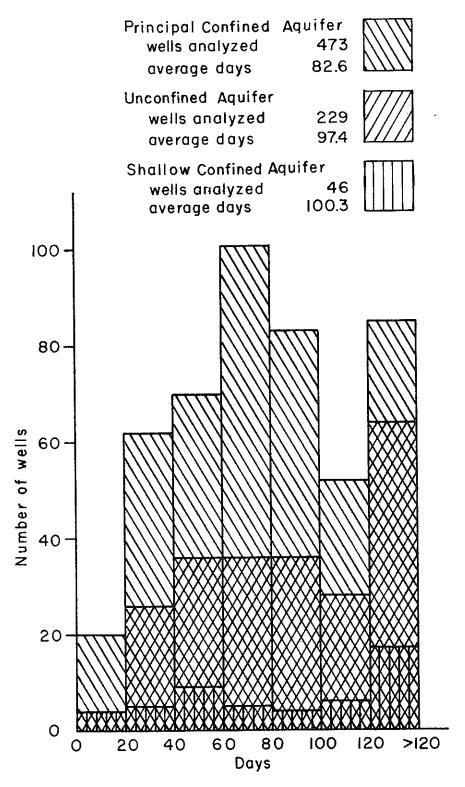


Figure 14. Days of use of wells tapping various aquifers during 1967, Roswell Basin, New Mexico.

Table 15. Percentage distribution of total annual pumpage according to source, Roswell Basin, New Mexico.

Period	Principal Confined Aquifer	Shallow Confined Aquifer	Shallow Aquifer
1900-1937	92.75	7.25	_
1938-1947	61.30	7.69	31.01
1948-1961	64.20	7.85	27.95
1962-1968	62.53	7.85	29.62

Table 16. Acreages of irrigated crops, Chaves County, New Mexico.1

Year	Alfalfa	Cotton	Sorghum	Small Grains	Total
1923	16,500	10,100	1,700	7,100	35,400
1924	14,900	17,000	1,400	5,500	38,900
1925	14,200	19,200	1,300	5,400	40,100
1926	15,300	18,000	1,300	7,500	42,100
1927	15,600	16,800	1,400	7,200	41,000
1928	13,800	20,800	1,700	5,600	41,900
1929	13,300	21,500	2,000	5,500	42,400
1937	15,000	38,000	13,000	10,000	76,000
1938	22,700	22,000	24,600	7,300	76,600
1939	19,000	23,000	18,500	12,100	72,700
1940	24,200	24,500	22,000	6,400	77,100
1941	24,000	24,300	28,300	7,500	84,100
1942	23,000	26,500	23,400	7,800	80,700
1943	26,000	26,600	29,300	7,900	89,800
1944	29,500	27,700	24,000	9,900	91,200
1945	30,000	33,000	21,000	8,200	92,200
1946	30,000	33,000	23,000	10,600	96,700
1947	24,500	46,400	18,500	8,000	97,400
1948	24,000	52,300	16,000	4,000	96,300
1949	24,000	55,000	23,000	2,500	104,500
1950	35,000	35,900	21,000	12,000	103,900
1951	32,000	58,000	15,000	4,000	109,000
1952	35,000	51,000	15,000	5 <b>,</b> 500	106,500
1953	35,000	59,100	12,000	6,000	112,100
1954	35,000	38,700	25,000	13,000	111,700
1955	35,000	32,300	25,000	16,000	108,300
1956	35,000	32,200	25,000	16,000	108,200
1957	35,000	31,600	27,000	17,200	110,800
1958	35,000	32,800	27,000	18,000	112,800
1959	27,600	32,400	5,600	9,300	75,000
1960	30,500	40,500	6,000	18,100	95,100
1961	34,000	35,000	9,100	19,200	97,300
1962	31,700	35,200	9,500	20,000	98,400
1963	33,200	31,400	11,500	19,000	95,100
1964	35,800	31,400	11,000	16,800	96,600
1965	41,000	30,800	8,000	9,000	88,800
1966	41,600	24,700	10,000	9,800	86,100
1967	38,400	22,200	9,000	8,600	78,200
1968	41,400	26,500	8,300	3,600	79,900

Compiled from records of the New Mexico Department of Agriculture [1962-1969], Roswell Chamber of Commerce, and the report by Mower [1960].

<sup>2</sup> Includes small grains plus miscellaneous crops.

Table 17. Acreages of irrigated crops, Eddy County, New Mexico. 1

Year	Alfalfa	Cotton	Sorghum	Small Gra	ins <sup>2</sup> Tota
1959	23,800	29,500	2,700	7,200	63,200
1960	25,000	32,100	2,800	8,500	68,400
1961	27,500	30,300	2,300	8,200	68,300
1962	27,000	29,900	2,200	7,800	66,900
1963	28,500	26,900	2,200	7,800	65,400
1964	26,400	24,400	1,000	1,900	53,700
1965	32,000	25,600	1,700	3,000	62,300
1966	31,700	20,800	1,700	4,100	58,200
1967	31,700	19,200	0	1,000	52,200
1968	35,000	22,900	1,300	1,000	60,200

<sup>1</sup> Compiled from records of the New Mexico Department of Agriculture [1962-1969].

<sup>&</sup>lt;sup>2</sup>Includes small grains and miscellaneous crops.

Consumptive Irrigation Requirement by Crops

"Consumptive irrigation requirement" as used here refers to the amount of water required for consumptive use by a crop per unit area, minus effective precipitation in that area during the period of time under consideration. Effective precipitation is that part of total precipitation which enters the soil and becomes available for plant use.

Consumptive use is defined by Blaney and Hanson [1965] as, "the unit amount of water used on a given area in transpiration, building of plant tissue, and evaporated from adjacent water surface, snow, or intercepted precipitation in any specified time. Consumptive use may be expressed in volume per unit area such as acre-inches or acre-feet per acre, or simply in depth such as in inches or millimeters or feet."

Values of monthly consumptive use were calculated by the Blaney-Criddle method [Blaney and Hanson, 1965] from 1905 to 1968, for the climatological data from the Roswell and Artesia stations. The Blaney-Criddle formula is expressed as follows:

$$u = kf, (8)$$

where

u = monthly consumptive use in inches

k = monthly empirical crop consumptive-use coefficient

 $f = t \times p/100 = monthly consumptive-use factor, t is mean monthly temperature in degrees Fahrenheit, and p is monthly percent of daytime hours of the year.$ 

Estimated values for  $\underline{k}$  are given by Blaney and Hanson [1965].

Monthly effective precipitations were calculated by using the following polynomial relation between the total monthly precipitation and the corresponding effective precipitation:

$$p_e = ap + bp^2 + cp^3 + dp^4$$
 (9)

where <u>p</u> = total amount of monthly precipitation; <u>a</u>, <u>b</u>, <u>c</u>, and <u>d</u> are constants where a=0.94574,  $b=0.27926 \times 10^{-1}$ ,  $c=-0.18451 \times 10^{-1}$ ,  $d=0.10224 \times 10^{-2}$ ; p<sub>e</sub> = monthly effective precipitation corresponding to p.

Relation (9) was derived from data used by the U.S. Bureau of Reclamation [Blaney and Hanson, 1965] for calculation of  $p_{\rm a}$  from  $p_{\rm c}$ .

A computer program was developed to calculate monthly consumptive irrigation requirements ( $u-p_e$ ) for alfalfa, cotton, sorghum, and small grains. Monthly irrigation requirements for these four crops in the Roswell and Artesia areas for the period 1905 to 1968 are given in Appendix D, along with other tables of related data.

The irrigated acreages for the years prior to 1959 in Eddy County are not available. As the crop pattern in the part of the basin located in Eddy County is more closely related to the pattern in Chaves County, yearly consumptive irrigation requirements (CIR) for crops in the Roswell Basin were estimated by multiplying CIR for crops in Chaves County by:

1.43 =  $\frac{2-\text{year}}{2-\text{year}}$  (1967-68) total groundwater pumped in Roswell Basin Chaves County

Table 18 shows yearly CIR values for the Roswell Basin. Yearly irrigation efficiency was obtained by dividing the yearly CIR value by the total water used for irrigation that year. The estimated yearly irrigation efficiencies are also shown in table 18. The average of the irrigation efficiencies from 1943 through 1968 in the Roswell Basin is 55 percent. These values may be off somewhat because of errors in estimating the total irrigation water applied and/or in estimating the yearly CIR values for the Roswell Basin.

#### Decline of Water Levels

Water-table maps of the shallow aquifer were drawn for 1926, 1938, 1944, 1954, 1964, and 1969 on a scale of one inch equals two miles. Potentiometric-surface maps of the principal confined aquifer were drawn for 1926, 1944, 1954, 1964, and 1969 on the same scale as the water-table maps. All of the maps were based on measurements made during January of the respective years.

Since 1938 the water table has been lowered conspicuously in three areas, forming large cones of depression near Artesia, Hagerman, and Dexter. The decline is more than 100 feet in some places.

The decline in the potentiometric surface since 1944 has also been large (as much as 100 feet at some places) but the cones of depression are flatter and extend out farther from the principal areas of withdrawal. The potentiometric surface is extremely flat in the Roswell area, mainly because the transmissivity of the principal confined aquifer is very high (~200,000 feet²/day in the Roswell area) compared with the transmissivity of the shallow aquifer (~13,000 feet²/day in the Roswell area). More than 40 percent of the pumpage from the principal confined

Table 18. Yearly irrigation water applied, consumptive irrigation requirement (CIR), and irrigation efficiencies, Roswell Basin, New Mexico.

	Water Applied <sup>l</sup>		Consumptiv	ent <sup>1</sup>	
Year	Groundwater	Total	Chaves County	Roswell Basin	Efficiency Percentage
1943	370.0	384.0	152.0	217.4	56.6
1944	344.0	358.0	139.1	198.9	57.8
1945	406.0	420.0	177.1	253.2	60.2
1946	375.0	389.0	149.4	213.6	54.9
1947	416.0	430.0	178.0	254.6	59.2
1948	406.0	420.0	176.5	252.4	60.1
1949	387.0	401.0	144.6	206.8	51.6
1950	394.0	408.0	136.7	195.5	47.9
1951	478.0	492.0	234.5	335.3	68.2
1952	447.0	461.0	212.1	303.4	65.8
1953	464.0	478.0	230.6	329.7	69.0
1954	472.0	486.0	195.8	280.0	57.6
1955	450.0	464.0	182.9	261.5	56.4
1956	496.0	510.0	222.7	318.5	62.5
1957	484.0	498.0	178.3	255.0	51.2
1958	383.0	397.0	154.2	220.6	55.6
1959	450.0	464.0	142.1	203.2	43.8
1960	424.0	438.0	138.3	197.8	45.2
1961	449.0	463.0	172.7	247.0	53.3
1962	465.0	479.0	143.0	204.4	42.7
1963	509.0	523.0	184.7	264.1	50.5
1964	536.0	550.0	184.1	263.3	47.9
1965	447.0	461.0	191.4	273.7	59.4
1966	390.0	404.0	165.5	236.7	58.6
1967	373.0	387.0	144.1	206.1	53.3
1968	329.0	343.0	114.8	164.2	47.9
Avera	ge 425.3	435.6	168.2	240.6	55.2

 $<sup>^{\</sup>mathrm{l}}$ In thousands of acre-feet.

aquifer is concentrated in the Roswell area. Figure 15 shows the average hydrograph for four recorder wells in the principal confined aquifer.

Average elevations of the water table in the shallow aquifer and average elevations of the potentiometric surface in the principal confined aquifer were estimated from the above-mentioned maps for different years and are shown in Table 19. The average elevation of the land surface in the same area is estimated to be about 3,465 feet above mean sea level.

Table 19. Estimated average January elevations of water table and potentiometric surface in the two aquifers of the Roswell Basin, New Mexico.

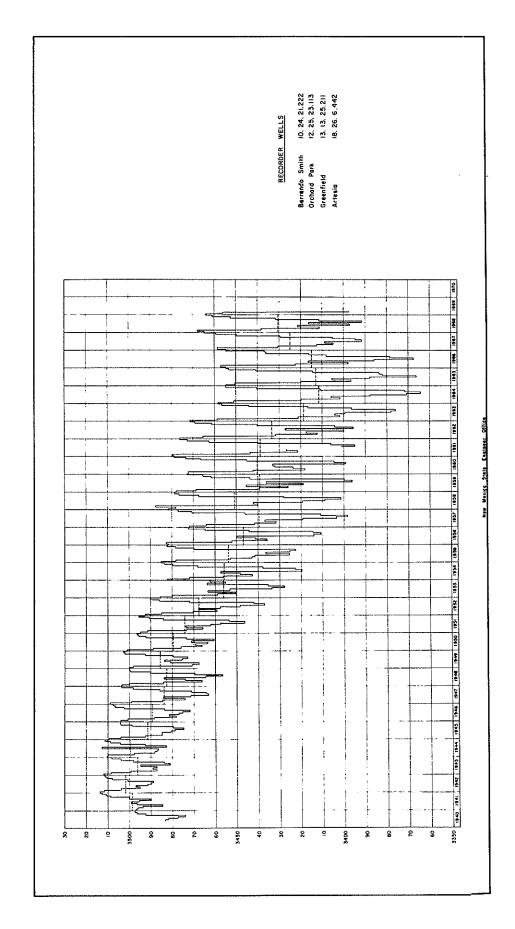
	Year				
Aquifer	1926	1944	1954	1964	1969
Shallow	(fee 3,407	t above 3,407	mean s 3,384	ea leve 3,374	1) .3,367
Principal Confined	3,451	3,461	3,417	3,397	3,403

The average water table declined 40 feet in the 25 years from 1944 to 1969. The average potentiometric surface sank 64 feet in the 20 years from 1944 to 1964, and then rose six feet in five years, from 1964 to 1969. This rise is attributed to an increase in precipitation in 1968 and a decrease in groundwater pumpage during 1967 and 1968.

The interpretation of changes in water level or changes in potentiometric surface in a coupled leaky aquifer system is complex. Pumping from one aquifer induces drawdown in the other. The amount of induced drawdown in a given time depends on the rate of discharge, on the hydraulic characteristics of both aquifers, and on the hydraulic characteristics of the aquitard that separates the two aquifers.

#### Water Budgets

This section contains a review of the analysis of water budgets of the Roswell Basin. The water budgets were estimated for four consecutive periods: 1926 through 1943; 1944 through 1953; 1954 through 1963; and 1964 through 1968. The



Eydrograph showing average mean monthly and mean annual hydraulic head in four observation wells penetrating the principal confined aquifer, Roswell Basin, Chaves and Eddy Counties, New Mexico. Figure 15.

water budgets were estimated using the hydrologic equations for the two aquifers which are simply water-inventory equations.

## Hydrologic Equations

The hydrologic equation for the principal confined aguifer for any period can be written as follows:

$$R + \Delta S_1 - P_1 = L + F$$
 (10)

where  $\underline{R}$  is recharge,  $\Delta S_1$  is change in storage during the period,  $P_1$  is pumpage from this aquifer,  $\underline{L}$  is leakage between the principal confined aquifer and the shallow aquifer, and  $\underline{F}$  is the groundwater flow to the east beyond the boundary of the basin (considered negligible compared to the other terms).

The hydrologic equation for the shallow aquifer can be written as:

$$R_{p} + R_{f} + L + \Delta S_{2} - P_{2} = D$$
 (11)

where R is recharge from local precipitation, R is replenishment from return flow,  $\Delta S_2$  is change in storage in the aquifer, P is pumpage from this aquifer, and D is the total natural discharge from this aquifer and includes the groundwater outflow to the Pecos River and the consumptive use of groundwater by salt cedars and other natural vegetation.

## Change in Aquifer Storage

The water table and potentiometric surface maps were drawn for January of 1926, 1944, 1954, 1964, and 1969 and the change in storage in the shallow aquifer was calculated from these maps. On all water-table maps, water-level elevations were interpolated at the section corners. The storage change was determined from map to map as follows: The elevation difference at each section corner was multiplied by the associated area, and the products were added algebraically and multiplied by the average ultimate specific yield.

The change in storage in the confined part of the principal confined aquifer is negligible because of very low storativity. The change in storage in the intake area of this aquifer was calculated by multiplying the weighted average change in the water levels in the intake-area wells by the area of the intake area, and then multiplying by the average storativity of the intake area.

## Leakage

Leakage between the principal confined aquifer and the shallow aquifer was estimated by applying equation (10). By this method, leakage could only be determined for those periods for which all the quantities in (10), except <u>L</u>, were known. Leakages were estimated for the four periods as shown in table 20. These values are net leakages during these four periods, and the yearly rates determined from them are not necessarily the actual rates of leakage.

The January leakage has always been positive as shown in table 7. The net leakages calculated with equation (10) have been negative since 1954. This is because the potentiometric surface during pumping seasons has been lowered much more than the water table and the direction of leakage is then reversed.

In conclusion, the leakage has been upward (positive) during January and probably during some other winter months too, and downward (negative) during the pumping season. The downward components of leakage during pumping seasons since 1954 have probably been higher, on the average, than the upward components during the winter seasons, which explains why the net leakage has been negative for the periods considered since 1954. It is, however, possible that the leakage may have been positive during some years since 1954.

## Natural Discharge

Natural discharge for the four periods was estimated using equations (10) and (11) and is shown in table 20. The change in storage in the intake area for 1926 through 1943 is not known. Moreover, the estimates for pumpages from both aquifers for this period are not reliable, and therefore the estimated natural discharge for 1926 through 1943 is uncertain. Natural discharge was slightly higher during 1944 through 1953 than during the subsequent two periods.

There could be some error in the estimation of any of the quantities in table 20, especially as regards changes in aquifer storage. This is especially true for the shallow aquifer because of lack of control in the western part of all the water-table maps.

## Summary of Water Budget, 1926-1968

Figure 16 summarizes the components of the hydrologic cycle of the Roswell Basin from 1926 to 1968. The upper portion shows yearly recharge, pumpage, leakage, and net gain or loss in storage in the principal confined aquifer. From 1926 through 1943, pumpage exceeded recharge during

Estimated water budgets (in thousands of acre-feet) for different periods in the Roswell Basin, New Mexico. Table 20.

	1926 . Total	- 1943 Per Year	1944 . Total	- 1953 Per Year	1954 . Total	- 1963 Fer Year	1964 Total	- 1968 Per Year
Principal Confined Aquifer	0 0 0 1	0 940	α α 	מ טוכ	0 880 1	8 80 5	С С С	α
ne charge	4,976.0	2.012	0.001,2	0.012	1,300.U	0.001	0.50k	161.0
Change in storage in the intake area	-240.03	-13.3?	725.0	72.5	938.0	93.8	405.0	81.0
Pumpage	-3,360.0	-186.6	-2,758.7	-275.9	-3,105.0	-310.5	-1,343.0	-268.6
Shallow Aquifer Leakage	1,372.0	76.2	74.3	η• Δ	-179.0	-17.9	-33.0	9.9-
Recharge from local pre-	634.1	35.2	262.0	26.2	252.6	25.3	125.0	25.0
Replenishment from return flow	1,560.6	7.98	1,359.6	136.0	1,468.0	146.8	633.5	126.7
Total replenishment	3,566.7	198.2	1,695.9	169.6	1,541.6	154.2	725.5	145.1
Change in storage	533.0	59.6	1,205.0	120.5	1,355.0	135.5	652.0	130.4
Pumpage	-1,150.3	-63.9	-1,358.0	-135.8	-1,477.0	-147.7	0.969-	-139.2
Natural discharge	2,949.4	163.9	1,542.9	154.3	1,419.6	142.0	681.5	136.3
Base flow	1,279.9	71.1	491.8	49.2	386.8	38.7	124.6	24.9
מתודמכם אמספו								

<sup>1</sup>Difference in yearly discharges of the Pecos River at Acme and at Artesia plus surface-water diversions.

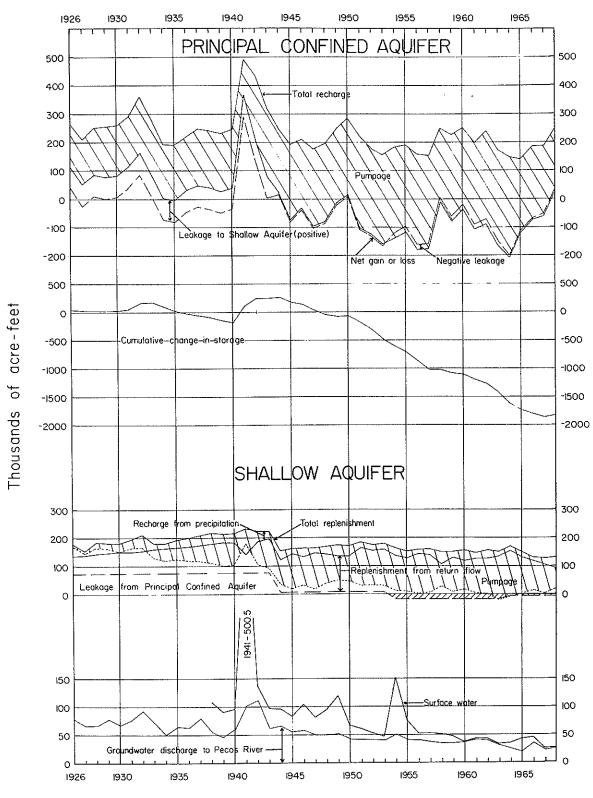


Figure 16. Estimated trends in inflows, outflows, and change in aquifer storage, Roswell Basin, New Mexico.

one year only (1935). From 1944 through 1968, pumpage exceeded recharge during all the years except 1944, 1950, and 1968. The net leakage was upward until 1953, when its direction was reversed for the subsequent two periods.

The cumulative change in storage in the principal confined aquifer is also shown in figure 16. The storage has been gradually decreasing since 1941 because of heavy withdrawals and in 1967 it reached its lowest value in history. In 1967, metering of water wells and limited pumpage went into effect as the result of a court decree fixing duty of water at 15 acre-feet per acre per five years. For this reason, and also because of higher precipitation, storage increased slightly during 1968.

Total replenishment to the shallow aquifer is shown in the lower part of figure 16. The pumpage from the shallow aquifer has always been less than the total replenishment except during 1963 and 1964. However, the pumpage plus the natural discharge has always been greater than the total replenishment to the shallow aquifer except perhaps during 1941. A gradual increase in pumpage has led to a progressive decrease in the groundwater discharge to the Pecos River from the basin. This decrease in discharge may also be partly due to a gradual increase in salt cedar acreage, a situation that has been aggravated by droughts. The precipitation at Roswell has been below average since 1950, except during the four years 1958, 1960, 1962, and 1968, and consequently the amount of Pecos River water available in the basin has gradually decreased (see figure 16 and table 20).

#### Saline Water Encroachment

The San Andres Limestone grades into salt beds in the northeastern-most part of the basin and beyond the basin boundary in that direction. In this section the groundwater in the San Andres Limestone has always been saline. Since the development of the basin, the saline water has been moving from the northeast toward the southwest, or toward Roswell, in the principal confined aquifer.

The chloride content of the water in the area varies from about 40 ppm to 40,000 ppm. Figures 17 and 18 [Hennighausen, personal communication] show isochlors and chloride content of selected wells tapping the principal confined aquifer. The location of isochlors changes with the time of year, ranging more to the southwest, toward the pumping centers, during the pumping season than during the shutoff period. Figure 19 [Hennighausen, personal communication] shows the encroachment of 500 parts per million isochlor from 1952 to 1965 and 1968. Hood [1963],

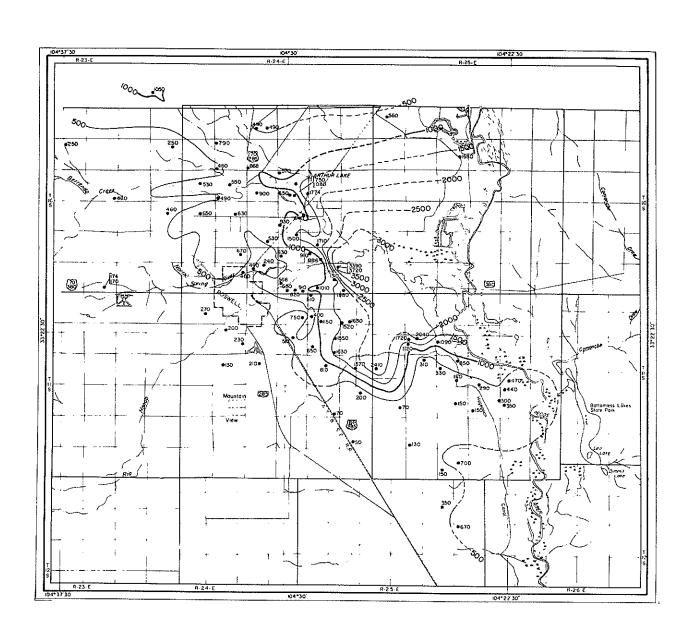


Figure 17. Isochlors in the principal confined aquifer near Roswell, New Mexico, March-April 1967.

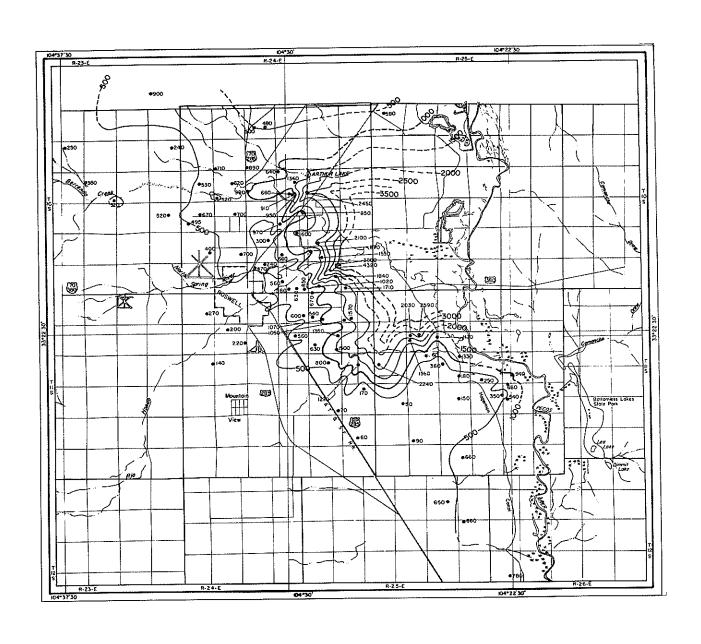


Figure 18. Isochlors in the principal confined aquifer near Roswell, New Mexico, August-September 1967.

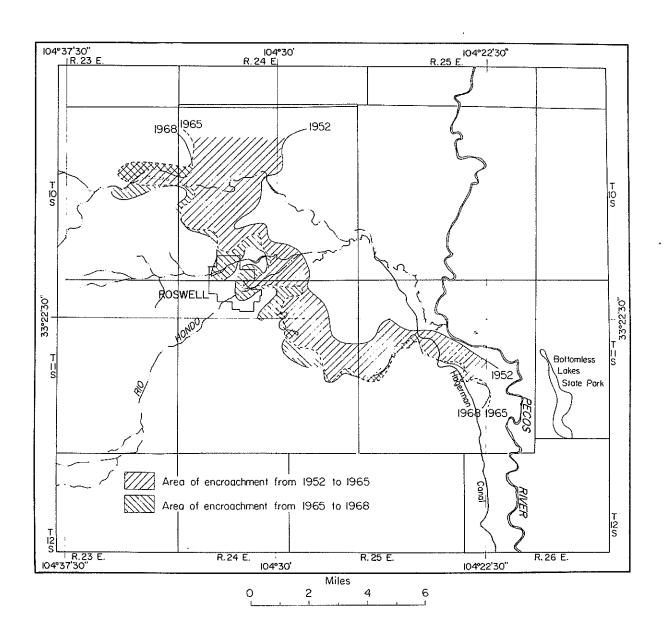


Figure 19. Encroachment of 500 parts per million isochlor from 1952 to 1965 and 1968 in the principal confined aquifer near Roswell, New Mexico.

Hood <u>et al</u>. [1960], Spiegel [1967], and Havenor [1968] have discussed in detail the saline-water encroachment in the Roswell Basin.

### Summary

The Roswell Basin in southeastern New Mexico is part of the Pecos River Basin and is comprised of two coupled leaky aquifers and a surface-water system. Groundwater occurs in unconfined as well as confined states in the basin. Groundwater is unconfined in the shallow aquifer and in the intake area of the principal confined aquifer. It is confined in the valley part of the principal confined aquifer and, in spite of the overdraft, there are still some wells in the northern half of the basin near the Pecos River that flow during winter months.

The recharge to the principal confined aquifer is from precipitation in the western and northwestern parts of the basin and averages about 240,000 acre-feet per year. The replenishment to the shallow aquifer is from: (1) gain or loss due to leakage between the principal confined aquifer and the shallow aquifer, (2) irrigation returns and losses from the surface drainage system, and (3) deep percolation from local precipitation.

Most of the pumpage from the principal confined aquifer is in the northern half of the basin, whereas the pumpage from the shallow aquifer is minimal in the northern part near Roswell. Water levels and potentiometric surface have been declining throughout the basin ever since its development.

The base flow to the Pecos River from the shallow aquifer has been decreasing continuously since the large-scale development of the aquifer started in 1938, except for a few years when, as a consequence of heavy rains in 1941, the aquifer almost recovered to its initial state.

The consumptive-waste use of water by salt cedars in the basin is still very great, amounting to about 107,000 acre-feet in 1968. The loss of water has decreased from 133,000 acre-feet during 1967 because of increased precipitation during 1968.

Saline-water encroachment still continues but it appears that the withdrawal of saline water from the principal confined aquifer just east of Roswell, by a saline water conversion plant, may retard the encroachment.

#### Chapter 3

#### THE DYNAMIC-PROGRAMMING APPROACH

#### Introduction

Most of the present-day water-resource systems are large and complex, and therefore difficult to analyze. best way to analyze such systems is to approach them syste-The Harvard Water Resources Group has developed matically. such a systematic approach [Maass, et al., 1962] to the methodologies of design and development of water-resource systems. They applied their studies to the Indus Plain of Pakistan which has one of the largest irrigation systems in the world [White House Report, 1964]. However, this report is concerned only with the optimal operation of existing water-resource systems, which is only one of the aspects of a complete system design. Optimal operation refers to the improvement of the operation of the system with optimum, or "the best", as a goal. Methods of achieving optima constitute the theory of optimization.

An important group of optimization methods comes under the mathematical-programming techniques. (The term "programming" here should not be confused with "setting up programs" for a digital computer.) Mathematical programming is defined by Sipple [1966] as the technique of finding an optimum value of a function of many variables when these variables are subject to restrictions in the form of equations or inequalities. The term is usually restricted to problems so complex that they require a digital computer for their solution.

Depending on the way in which a particular technique optimizes a problem, optimization techniques can be classified in two categories, namely, simultaneous and partial optimization [Meier and Beightler, 1968]. In simultaneous optimization techniques, of which linear programming and nonlinear programming are examples, all decision variables are directed toward their optimum values simultaneously by using some form of iterative technique. In partial optimization, of which dynamic programming is an example, the problem is decomposed into simpler subproblems that are analyzed sequentially by partially optimizing single variables or groups of variables while maintaining the effects of interactions among them.

Linear programming is suitable for solving problems with linear objective functions and linear constraints, whereas nonlinear programming is used for problems with

nonlinear objective functions and/or nonlinear constraints, making it much more complex. Details of these techniques are presented in Hadley [1962, 1964]. An application of nonlinear programming for the evaluation of aquifer parameters is given in the preceding chapter.

In problems that involve discrete input data or tabular data, or discontinuous objective functions, multistage optimization (dynamic programming) is often the only satisfactory method for optimization. Problems of this type are fairly common in water-resource systems. The main limitation of the usefulness of dynamic programming is the dimensionality of a problem—that is, the number of independent state variables involved in the problem. At the present, a two-dimensional, or even a three-dimensional, problem can be handled on a medium—to large-sized computer depending on the speed and storage capacity of the computer and the size of the problem. However, there are methods to overcome the problem of dimensionality to a limited extent [see Bellman, 1957, 1962].

A brief description of dynamic programming and some of the terms associated with it are presented in the following section.

#### Dynamic Programming

A process in which a single decision is to be made is called a single-stage process, whereas one which involves a sequence of decisions is known as a multistage decision process. Dynamic programming is a strategy for solving optimization problems involving multistage decision processes. Some problems can be formulated into such a process by introducing an artificial element of time. Richard E. Bellman coined the term "dynamic programming" and is responsible for developing the theory of dynamic programming, which he discusses in several books [Bellman, 1957, 1961, 1962; Bellman and Kalaba, 1965]. The original principle on which Bellman based his principle of optimality was formulated by Massé [1946].

Perhaps the earliest application of dynamic programming to water-resource studies was by Little [1955]. Bear et al. [1964] have surveyed the literature related to optimization of water-resource systems.

Some of the terminology applied in dynamic programming is described below:

State. A state of a process is a description of one of the conditions or situations in which the process may exist. A set of all possible states in which the process might

exist constitutes a state space and the set of variables defining this state of the process is called the  $\underline{\text{state}}$  variables.

Decision. A decision represents one of the choices available when the process is in a particular state. A decision set for a particular state is the set of all possible choices that might be made when the process is in that state.

Policy. An operating policy is an ordered collection of decisions containing one decision for each state in the state space. A policy which satisfies all the constraints on the system is known as an <u>admissible policy</u>, and an admissible policy that maximizes the objective function for a system is called optimal policy.

Transformation. Each stage of a process transforms the state of its input into an output state dependent on the decision that is made for the operation of that stage.

Constraints. In general, both the state and decision variables are limited by constraints. Typical constraints are physical, technical, budgetary, political, and legal. Generally they have the form of inequalities and/or equalities.

Objective Function. In an optimization problem there must be some criterion according to which the problem is to be optimized. This is usually called criterion function, return function, or objective function. Generally the objective is to maximize the net benefits or to minimize the costs. It is a scalar function (or functional) of decision and sometimes also of state variables (or functions of these variables). For further discussion of objective function, see Maass, et al. [1962] and Parker and Crutchfield [1968]. The maximized return for a given state is called optimal net return for that state.

Principal of Optimality. The first step in solving a mathematical model is to simplify it and/or to make certain changes or transformations so that it is easier to solve in the new form. Unlike simplifications, these changes completely preserve the properties of the model but the transformed model is easier to optimize. Such a transformation is accomplished for multistage-decision problems by the dynamic programming approach. A problem with N decisions can be transformed into N subproblems, each containing only one decision variable. This is accomplished through the application of Bellman's principle of optimality [Bellman, 1957]:

"An optimal policy has the property that whatever the initial state and initial decision are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision."

Or, to put it another way, "It is really saying that if you don't do the best you can with what you happen to have got, you'll never do the best you might have done with what you should have had" [Rutherford, 1964].

### Statement of the Problem

A schematic representation of two coupled leaky aquifers and an interrelated surface-water system is shown in figure 20. The combined system includes a shallow unconfined aquifer and a deeper confined aquifer. The two aquifers are separated by a semiconfining layer which permits leakage from one aquifer to the other, and the direction of leakage is locally toward the aquifer with lower head. The surface-water system is tied to the unconfined aquifer through base flow, which can be negative—that is, the aquifer can gain water from the river.

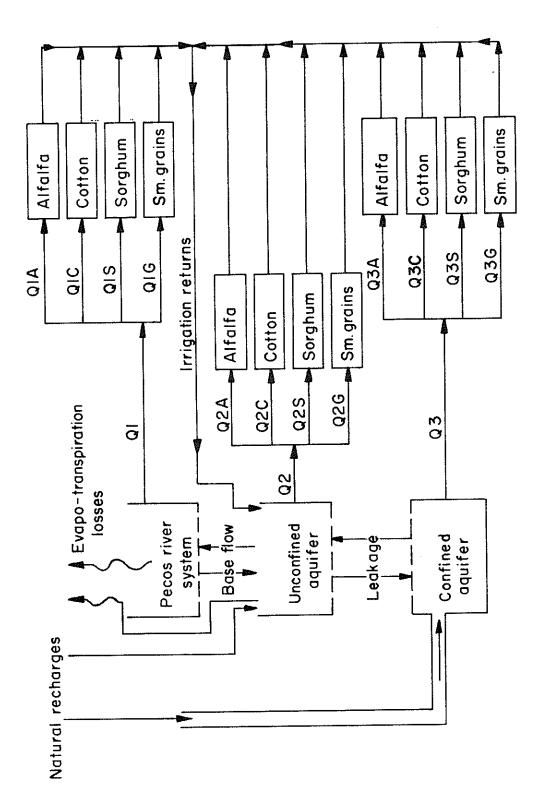
The system is assumed to be used only for irrigation of land. Only four crops are assumed to be grown: alfalfa, cotton, sorghum, and small grains (for example, barley). Areas of specific crops to be irrigated from each of the three sources (12 areas in all) are assumed to be known.

The shallow aquifer and the confined aquifer receive probabilistic recharges from precipitation. Besides leakage, which is assumed positive if it is from the confined aquifer to the unconfined aquifer, the latter gains water also in the form of return flows from irrigation and loses (or can gain) water in the form of natural discharge to the river. The river loses water through evaporation, and both the river and shallow aquifer lose water through consumptive—waste use by salt cedars.

The system is operated by pumping water from both aquifers and by diverting surface water for irrigation. The amounts of water to be withdrawn from both aquifers and the surface water to be used, in one period, are treated as decision variables. Each decision variable has four components, one for each of the crops. The purpose of the study is to find an optimal operating policy so that the value added to the basin is maximized over a long period of time.

# Dynamic Programming Formulation

The following notations are used for the mathematical formulation of the problem:



Suhematic representation of Roswell Basin, New Mexico. Figure 20.

- 1, 2, 3 = subscripts referring to the surface water system, shallow aquifer, and confined aquifer, respectively.
- $Q_{\rm ln}$ ,  $Q_{\rm 2n}$ ,  $Q_{\rm 3n}$  = volume of water diverted or pumped during the n<sup>th</sup> period from the end of the planning horizon.
- $v_1$ ,  $v_2$ ,  $v_3$  = amount of water in storage at the beginning of a period.
- r<sub>1</sub>, r<sub>2</sub>, r<sub>2k</sub> = probabilistic recharges to the surface water system, to the shallow aquifer, and to the confined aquifer, respectively, during the n period from the end of the planning horizon.
- $p_{1i}$ ,  $p_{2j}$ ,  $p_{3k}$  = probabilities for  $r_{1i}$ ,  $r_{2j}$ , and  $r_{3k}$ , respectively.
- $\rm E_n$  = natural losses from the surface-water system during the  $\rm n^{th}$  period from the end of the planning horizon.
- $\rm D$  = natural discharge from the shallow aquifer during the  $\rm n^{th^{}n}\rm period$  from the end of the planning horizon.
- $L_n$  = amount of leakage during the  $n^{th}$  period from the end of the planning horizon, assumed positive when to the shallow aquifer.
- u = discount rate to reduce future benefits to present values.
- $B_n(Q_{1n},Q_{2n},Q_{3n},V_1,V_2,V_3)=$  net value added to the basin during the nth period from the end of the planning horizon, obtained from the use of  $Q_{1n},Q_{2n}$ , and  $Q_{3n}$  amounts of water when the storages were  $V_1,V_2$ , and  $V_3$ , respectively, in the surface water system, the shallow aquifer, and the confined aquifer.
- $\begin{array}{c} f_n(V_1,\,V_2,\,V_3) = \text{the expected value added to the basin} \\ \text{from the n-stage process following an optimal policy starting} \\ \text{with } V_1,\,V_2,\,\text{and }V_3 \text{ as the amounts of water in the three} \\ \text{components of the system.} \end{array}$

The maximum of  $\underline{R}$  is to be obtained in the multi-dimensional space of the variables of which it is a function.

In order to obtain the maximum, the problem is approached through dynamic programming. The areas that are to be irrigated are treated as parameters of the system and the physical dimensions of the system are evaluated from hydrologic analysis.

Two distinct stages of optimization are involved. One is the optimization of the system as a function of time—that is, finding an optimum operating policy for a finite number of stages n. In order to perform optimization of the system as a function of time, one must first optimize the system in space. The results of optimization in space are incorporated in the dynamic model so that there is total optimization within the constraints of the system. The dynamic model is formulated first.

# Formulation as a Function of Time

In order to analyze the problem of two coupled leaky aquifers and a surface-water system, a set S and a set P are defined so that all admissible states and all admissible decisions are contained in S and P respectively--that is,  $(V_1, V_2, V_3)$   $\epsilon$  S and  $(Q_1, Q_2, Q_3)$   $\epsilon$  P.

Let  $f_0(V_1, V_2, V_3) = 0$  and consider the operation of the system for one stage; the optimal one-stage return is obtained as follows:

$$f_1(V_1, V_2, V_3) = \max_{(Q_1, Q_2, Q_3) \in P} \{B_1(Q_1, Q_2, Q_3, V_1, V_2, V_3)\}$$
 (12)

The state of the system is transformed by the transformation  $\underline{\mathbf{T}}$ , where  $\underline{\mathbf{T}} = \mathbf{T}(Q_1,Q_2,Q_3,r_1,r_2,r_3,D,E,L)$ .

Employing the principle of optimality, using the expected value criterion, and discounting the returns to current value, the functional equation of dynamic programming [Bellman, 1957] for the system can be written as:

max

$$\begin{split} f_n(v_1, v_2, v_3) &= & \{B_n(Q_{1n}, Q_{2n}, Q_{3n}, v_1, v_2, v_3) \\ &+ (1+u)^{-1} \int \int \int_{-\infty-\infty}^{\infty} f_{n-1}(v_1 + r_{1n} - E_n + D_n - Q_{1n}, v_2 + r_{2n} + c(Q_{1n} + Q_{2n} + Q_{3n}) \} \end{split}$$

+ 
$$L_n - D_n - Q_{2n}, V_3 + r_{3n} - L_n - Q_{2n} h_1 (r_1) h_2 (r_2) h_3 (r_3) dr_1 dr_2 dr_3$$
 (13)

where  $h_1(r_1)$ ,  $h_2(r_2)$  and  $h_3(r_3)$  are probability density functions for  $r_1$ ,  $r_2$ , and  $r_3$ , respectively, and  $\underline{c}$  is a constant for calculation of deep percolation from irrigation returns.

The above functional equation for an n-stage process can be interpreted as the maximization, with respect to water withdrawals during stage  $\underline{n}$ , of immediate net value added during stage  $\underline{n}$  plus the present value of future net value added during the (n-1) stages, following an optimal policy during the remaining (n-1) stages.

In order to solve equation (13), all of the variables are made discrete and the equation takes the following form:

max

$$f_{n}(v_{1},v_{2},v_{3}) = \{g_{n},Q_{2n},Q_{3n}\} \in P \{g_{n}(Q_{1n},Q_{2n},Q_{3n},v_{1},v_{2},v_{3})\}$$

+ 
$$(1+u)^{-1}$$
  $\sum_{i=1}^{L}$   $\sum_{j=1}^{M}$   $\sum_{k=1}^{N}$   $p_{1i}p_{2j}p_{3k}f_{n-1}(v_1+r_{1n}-E_n+D_n-Q_{1n}, v_1+v_2)$ 

$$V_{2}+r_{2n}+c(Q_{1n}+Q_{2n}+Q_{3n})+L_{n}-D_{n}-Q_{2n},V_{3}+r_{3n}-L_{n}-Q_{2n})$$
(14)

where

$$p_1, p_2, p_3 \ge 0, \qquad \sum_{i=1}^{L} p_{1i} = \sum_{j=1}^{M} p_{2j} = \sum_{k=1}^{N} p_{3k} = 1$$
 (15)

and  $r_{11}$ ,  $r_{12}$ ,  $r_{13}$ , ...  $r_{1L}$  are stochastic recharges to the surface water system with probabilities  $p_{11}$ ,  $p_{12}$ ,  $p_{13}$ , ...,  $p_{1L}$ ;  $r_{21}$ ,  $r_{22}$ ,  $r_{23}$ , ...,  $r_{2M}$  are stochastic recharges to the shallow aquifer with probabilities  $p_{21}$ ,  $p_{22}$ ,  $p_{23}$ , ...,  $p_{2M}$ ; and  $r_{31}$ ,  $r_{32}$ ,  $r_{33}$ , ...,  $r_{3N}$  are stochastic recharges to the confined aquifer with probabilities  $p_{31}$ ,  $p_{32}$ ,  $p_{33}$ , ...,  $p_{3N}$ , respectively.

It is assumed that the decision variables and the states are non-negative and that the decision variables cannot exceed the amounts of storage in the respective components of the system--that is,

$$0 \le Q_j \le V_j, \quad j = 1, 2, 3.$$
 (16)

The discretization of the problem reduces it to a finite Markovian decision process [Howard, 1960]. The

probabilities in equation (13) can be expressed in the form of a transition probability matrix, where the multidimensional matrix will define the probabilities of going from one state to the next.

Formulation as a Function of Space

This section is concerned with the optimal allocation of a given amount of water from one of the three sources to the four crops that are to be grown on given areas. This problem is an example of optimization in space, and it is approached in the same manner as the previous problem, which involved optimization over time. An artificial element of time is introduced and the stages of the preceding section are now replaced by the four crops. The problem is formulated as a functional equation of dynamic programming and solved through sequential analysis. Some additional symbols used in the formulation are described below:

- S = the initial state of the system, defined by the total amount of water in storage in the system component under consideration, and the total amount to be withdrawn from that component for allocation to different crops. The system has three components, namely, the surface-water system, the shallow aquifer, and the confined aquifer.
- $Q_{ik}$  = the amount of water allocated to the  $k^{th}$  activity and is a fraction of the total amount of water  $Q_i$  to be used from the  $i^{th}$  component of the system during one period.
- $G_k(Q_{ik},Q_i,V_\ell)$  = net value added to the basin resulting from the use of an amount  $Q_{ik}$  of water for the  $k^{th}$  activity with  $Q_i$  as the total amount of water to be withdrawn from one of the three components of the system, when storage in that component is  $V_\ell$ .
- $S_j$  = the state resulting from the transformation of state  $S_i^j$  . The transformation is due to allocation of an amount  $\overline{\mathbb{Q}}_{i\,k}$  of water to the  $k^{th}$  activity.

Using the principle of optimality, as in the previous section, the functional equation is derived for the optimal allocation of water from one of the components of the system to various crops:

$$f_{k}(s_{i}) = \max_{Q_{ik} \in P_{n}} \{G_{k}(Q_{ik}, Q_{i}, V_{i}) + f_{k-1}(s_{j})\}$$
(17)

where

$$S_i = (Q_i, V_\ell) \tag{18}$$

$$\mathbf{s_i} = (\mathbf{Q_i} - \mathbf{Q_{ik}}, \mathbf{v_l} - \mathbf{Q_{ik}}) \tag{19}$$

 $P_n$  = the set containing all the admissible decisions. There are three such sets, one for each of the components of the system.

$$f_0(S_i) = 0.$$
 (20)

There are three equations like (17), one for each of the three components of the system. Each equation can have a different state set, a different decision set, and a different set of  $G_{\bf k}$ 's.

The results obtained from the solution of (17) are incorporated into the dynamic model to obtain overall optimization.

### Discussion

Two types of maximization problems have been formulated in this chapter. One is the maximization of net value added due to the operation of the coupled leaky aquifersurface water system over a long period of time. The second problem, which is incorporated in the first one, is the maximization of net value added due to allocation of a given amount of water from one of the components of the system to various activities with independent objective functions.

If the system is to be operated for n stages (for the Roswell Basin a stage is of one-year duration), there will be 3n decisions to be performed simultaneously for a system consisting of three components. By the approach adopted in this report, the problem was reduced to a sequence of n different problems with only three decisions to be made in each. This approach of sequential decision-making has greatly simplified the original problem. Bellman [1957, 1962] has discussed the advantages from an analytic as well as from a computational point of view. The same approach is adopted for the problem of optimization in space.

The main disadvantage of the dynamic programming approach is the limitation of dimension. To illustrate: for a system with only one state variable and 10 possible states, the maximization will be performed only 10 times. For a system with two state variables, each having 10 possible states, the computational burden will increase 10 times—that is, the maximization will be carried out  $10^2$  times—and for a three-state variable,  $10^3$ , and so on.

At present, most of the large computers can only handle problems with three state variables or, at the most, four dimensional problems of limited size.

The formulation of the problem of optimization of the system in time in a dynamic-programming model is an example of stochastic dynamic programming (equation 13) because the transformation is not completely known and the resultant outcome of the transformation is in the form of a probability distribution. The second problem, of optimization in space, is an example of deterministic dynamic programming (equation 17) because the outcome of the transformation is uniquely determined by a decision.

For the stochastic dynamic model, the expected value criterion was used. It implies that if the system is operating for infinitely many stages, the system will yield the maximum average returns. In actual practice the computations are carried out only to a finite number of stages. This is a valid procedure, because for large  $\underline{n}$  the effect of all those stages which are far off in the future is negligible on the current decisions. Bellman [1957] and Howard [1960] have shown that when the objective function is invariant over the stages, the solution of a dynamic programming algorithm yields rapidly a long-run optimal operating policy -- that is, the policy remains the same no matter how many stages still remain in the process. Usually such a convergence is obtained in 7 to 15 stages. For the Roswell Basin the objective function was assumed to be invariant over the stages and a rapid convergence was obtained for n=8.

# Chapter 4

#### SOLUTION OF THE MODEL

### Introduction

The model developed in the preceding chapter is here applied to the Roswell Basin. The model is given by equations  $1^4$ , 15, and 17 of the preceding chapter, but as no general analytical solutions of these equations are available, they must be solved on digital computers for specific cases.

Models are usually simplified so that they can be handled on computers but oversimplification may lead away from the real situation. The problem for the Roswell Basin was solved on an IBM System 360 Model 44 computer. The inputs to the model, results, and limitations are discussed in this chapter.

# Inputs to the Models

There are two main types of inputs to the model—economic and hydrologic. The crop pattern used in the model is given under the heading of other inputs.

#### The Economic Inputs

#### Benefit Functions

The net returns per acre were derived for different crops for the Roswell Basin by Dregne, Garnett, and Lansford [1967]. The values added per acre to the basin were computed from the net returns to land and management by d'Arge [personal communication, 1968]. The benefit functions for all the crops are nonlinear. Second-degree polynomials were fitted through the values added as a function of the amount of water pumped in acre-feet per acre of land and are as follows:

$$v_a = -443.89 + 181.86x - 12.26x^2$$
 (21)

where  $v_a$  is the value added for alfalfa in dollars per acre and  $\underline{x}$  is the amount of water pumped for irrigation of one acre of land.

$$v_c = -90.91 + 242.54y - 32.32y^2$$
 (22)

where  $\mathbf{v}_{\mathbf{c}}$  is the value added for cotton in dollars per acre and  $\underline{\mathbf{y}}$  is the amount of water pumped for irrigation of one

acre of land. The pumping costs, which originally, in calculating the net returns, were assumed not to vary with dopth, were added to the values added. The pumping costs as functions of depth were then subtracted in our analysis to give net values added.

# Pumping Costs

The cost for pumping a unit amount of water depends on the depth to water in the wells, and varies in general from well to well and from aquifer to aquifer in the basin.

The fuel costs for 31 different wells in the basin were collected. Natural gas was used at four of the wells and the rest were equipped with electric motors. From this information and from information about the total amount of water pumped by each well during 1967, fuel costs per acrefoot of water were calculated. Dividing by weighted average lift, costs per acrefoot per foot of lift were calculated, and are summarized in table 21. An amount of \$0.002 per acrefoot per foot of lift was added for maintenance, repairs, lubrication, and labor. These costs are fairly close to those obtained by Long [1965] for the Roswell Basin.

Table 21. Pumping costs per acre-foot per foot of lift, Roswell Basin, New Mexico, 1967.

Aquifer		Fuel Cost lars)	Weighted Average Cost <sup>l</sup>
	Electricity	Natural Gas <sup>2</sup>	(dollars)
Shallow	0.0386	-	0.0355
Principal Confined	0.0303	0.0170	0.0283

lncludes maintenance, repair, lubrication, and labor costs.

The Hydrologic Inputs

# Storage-Depth Relations

The relations of average depth to water to the amount of water in storage in the shallow aquifer and in the principal confined aquifer were assumed to be linear on a yearly basis. It is implied that the pumpage from the aquifers is reasonably uniformly distributed.

 $<sup>^{2}</sup>$ Using the same ratio for the shallow aquifer.

The relation of average depth to the amount of water in storage in the shallow aquifer was estimated from the average east-west cross section of the basin and from the average elevations of the water table calculated from the water-table maps. A maximum permissible average depth of pumping of 145 feet, and an average minimum depth of pumping of 45 feet were assumed to preclude waterlogging and other drainage problems. An average storativity of 0.15 was used for the shallow aquifer. The relation is as follows:

$$D = 145 - 25.0 V_{1}$$
 (23)

where

D = the average January depth to the water table in feet,

V<sub>1</sub> = the volume of water in storage in the permissible depth range in the shallow aquifer, in millions of acre-feet.

The similar relationship was obtained for the principal confined adulfer. The storativity in the confined part of this aquifer is negligible compared with the storativity in the intake area. The January average water levels in the intake-area wells were correlated with the January average potentiometric surface as recorded by the U.S. Geological Survey in the confined part of the aguifer.

$$h = 150 - 27.273 V_2$$
 (24)

where

h = average January lift in the principal confined
 aquifer, in feet,

V<sub>2</sub>= the amount of water in storage in the permissible depth range in the principal confined aquifer, in millions of acre-feet.

A maximum average lift of 150 feet was allowed and average storativity of 0.03 was assumed in the intake area. The maximum storage capacities of the shallow aquifer and the principal confined aquifer were estimated within the permissible depth ranges to be 4.0 and 5.5 million acrefeet respectively.

#### Drawdowns at the Wells

Previous authors [Buras, 1963; Burt, 1964] did not deduct from the benefit function the additional pumping costs associated with the self-drawdowns of pumping wells. As a well pumps water, the amount of lift increases somewhat in proportion to the amount of water pumped from the

well. Drawdowns at both shallow and deep wells were calculated as functions of discharges from the wells, from the average values of B, the formation-loss coefficient, and of C, the well-loss coefficient. For the shallow aquifer, Togarithmic average values for B and C are 8.37 ft/cfs and 2.98 ft/(cfs)<sup>2</sup>, respectively. The logarithmic average values of B and C for the principal confined aquifer are 3.41 ft/cfs and 2.20 ft/(cfs)<sup>2</sup>, respectively. These values were obtained as described in Chapter 2 and were adjusted to take into consideration the pumping time in an average irrigation cycle, average days of use of wells, and the number of wells tapping the two aquifers.

The relation for self-drawdown is

$$s = BQ + CQ^2$$
 (25)

Pumping costs due to additional lifts at the wells were also subtracted from the value added, in addition to the costs associated with the lifts calculated from the storage-depth relations.

#### Leakage

The amount of leakage through the semi-confining layer was calculated as described in Chapter 2. Not only was the leakage calculated as a function of difference in average heads of the two aquifers, but also the area of the semi-confining bed, through which leakage takes place, was incorporated as a variable.

### Recharge to the Aquifers

The rates of recharge to the shallow aquifer and to the principal confined aquifer were calculated as described in Chapter 2 and tabulated in increasing order in tables 1 and 6. The probabilities were calculated by Kimball's method [1946]:

$$p(r_m) = \frac{m}{n+1} \tag{26}$$

where p(r<sub>m</sub>) is the probability of recharge of magnitude r<sub>m</sub> or less and having a rank  $\underline{m}$ , when the sample size is  $\underline{n}$ . For the numerical solution, the amounts of recharge and their probabilities are shown in table 22 and were used as input to the model.

#### Natural Discharge

For the numerical solution of the model a total value of 130,000 acre-feet per year for the natural discharge from the shallow aquifer was used, which includes base flow as well as other natural losses.

Table 22. Amounts of recharge (in acre-feet) and probabilities, based on records for 68 years, Roswell Basin, New Mexico.

Principal C	onfined Aquifer	Shallov	v Anuifer
Recharge	Probability	Recharge	Probability
150,000	0.10	20,260	0.30
184,000	0.20	35,000	0.40
246,000	0.40	49,600	0.30
291,000	0.20		
361,000	0.10		

#### Other Inputs

An interest rate of 5 percent was used to discount the future benefits to present values. The crop pattern assumed for the model is shown in table 23 and is based on the yearly records of crops grown in the basin.

Table 23. Crop pattern (in thousands of acres) for the dynamic programming model, Roswell Basin, New Mexico.

Crop	Shallow Aguifer	Principal Confined Aquifer	Total
Alfalfa	18	30	48
Cotton	14	22	36
Sorghum	Ħ	8	12
Small Grain	is 6	12	18
Total	42	72	114

# Solution of the Model

It was mentioned earlier that the surface water used for irrigation is less than 4 percent of the total amount used for irrigation in the Roswell Basin. Therefore surface water was not included in the dynamic programming model as an independent state and the model became simplified from a three-dimensional to a two-dimensional model, with a considerable saving of computer time, and the error introduced is small.

It was assumed that the amount of surface water used will continue to be the same as at the present time. It was further assumed that 3.00 acre-feet and 2.25 acre-feet of water per acre of land are pumped for irrigating sorghum and small grains, respectively. These amounts give the maximum gross value added of \$105.04 and \$66.15 per acre for sorghum and small grains, respectively, and these were added to the benefit function. This was done to save computer time and only a small error was introduced by this simplification because the water consumed by sorghum and small grains is slight compared with that by the two main crops, alfalfa and cotton.

Flow charts for the numerical solution of the dynamic programming functional equation are given in several publications, such as Bellman [1962] and Nemhauser [1966].

### Results

The optimal operating policies for the shallow aquifer and the principal confined aquifer are shown in figures 21 and 22, respectively. These policies are based on the physical and hydrological characteristics of the system as described in the preceding sections. The benefit function was assumed to be invariant with respect to time, and the solution converged to the optimal policies on the eighth stage.

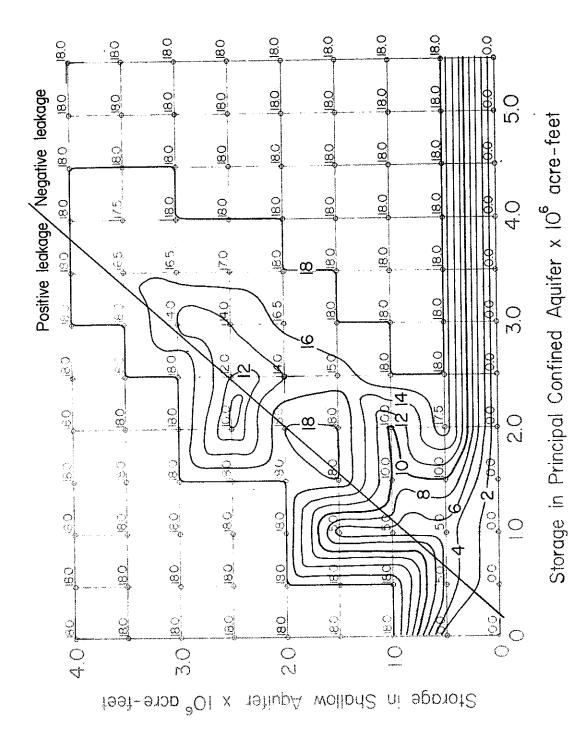
### Interpretation of Results

In order to interpret the optimal operating policies shown in figures 21 and 22, one must understand the economic factors affecting the optimal operating policies [Burt, 1966]. The two economic factors that tend to increase the operating policy for an aquifer are the discount factor, which tends to reduce the influence of future benefits and thus increases the influence of current benefits, and the increase in marginal returns at low levels of water use per period. The two economic factors that reduce the optimal operating policy are the decrease in marginal returns when the policy is already high, and the increase in expected pumping costs associated with the remainder of the planning horizon.

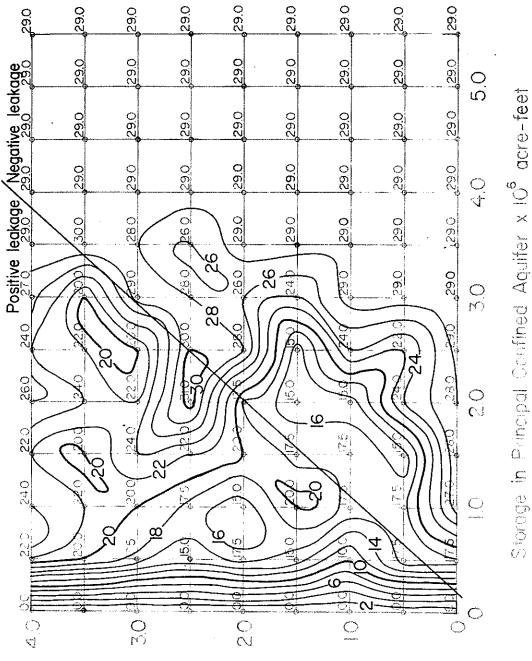
In addition to the above factors there is another factor that influences the operating policy of a coupled leaky aquifer system. This is the mutual interaction of the aquifers. The amount of leakage from one aquifer to the other makes the optimal operation of one aquifer strongly dependent on the storage in the other aquifer in addition to its own storage.

The final optimal operating rule is a function of all the five factors and is, as a matter of fact, the result of all these appropriately weighted positive and negative factors. This makes the interpretation of the optimal policy fairly complicated.

The results of the computer solution are shown as isoquants depicting the optimal operating policies for the shallow aquifer and for the principal confined aquifer in figures 21 and 22, respectively. A line of zero leakage is also drawn. The leakage is positive (from the principal confined to the shallow aquifer) to the right of this line, and leakage is negative to the left of the line.



aquifer which maximize the objective function for n>8, Roswell Basin, New Mexico. Yearly pumpages (in 104 acre-feet) from the shallow 21. Figure



Storage in Shallow Aquifer taat-ann aoi x

confined aquifer which maximize the objective function for n>8, Roswell Basin, New Mexico. Yearly pumpages (in  $10^4$  acre-feet) from the principal Figure 22.

Referring to figure 21, the optimal operating policy for the shallow aquifer is uniform for all storages in the aquifers except when storage is very low in the shallow aquifer, and except near the zero-leakage line. The optimal operating policy appears to be strongly influenced by mutual leakage of the aquifers. This is manifested by a broad low which trends parallel to the zero-leakage line. Corresponding to the two highs to the left of the zero-leakage line in figure 22, there are two lows in figure 21, probably because the shallow aquifer is losing water through leakage at those storages.

The optimal operating policy for the principal confined aquifer is also influenced by the interaquifer leakage. This is shown, in general, by the alignment of highs and lows on the opposite sides of the zero-leakage line. The two highs can be interpreted as indicating that the optimal policy is to pump more water from the principal confined aquifer at these states because the aquifer is gaining water through leakage. Similarly, lows indicate that the pumpage should be less because the aquifer is losing water. The low appearing in figure 22, when storage in the shallow aquifer is about 3.25 million acre-feet, is probably caused by other factors affecting the optimal operating policy.

In general, the results obtained agree closely with those expected intuitively—that is, the optimal operating policy for an aquifer is high when the storage in the aquifer is high. The operating policy is never higher than the amount of water at which the net output ceases to have a positive marginal value. The sum of the operating policies for both aquifers at given storage values also follows this relationship, and furthermore, the influence of leakage is not obvious as in the case of individual aquifers.

The operating policies shown in figures 21 and 22 are the amounts of water to be pumped for irrigating alfalfa and cotton only. In addition, water must be pumped from the shallow aquifer and the principal confined aquifer for growing sorghum and small grains. The acreages of these two crops are shown in table 23.

# Procedure for Use of Results

The average depths to water level and potentiometric surface in the shallow aquifer and in the principal confined aquifer, respectively, are first calculated from the January maps for the year for which optimal pumpages are required. Using the storage-depth relations, storage in each aquifer is calculated for the year under consideration. Knowing the storage in the two aquifers, optimal amounts of groundwater to be pumped in that year from the two aquifers

are read from figures 21 and 22, respectively. This pumpage is for growing alfalfa and cotton only; the acreages of these crops are listed in table 23. To these pumpages must be added the amounts of groundwater needed to grow sorghum and small grains.

### Limitation of Results

The optimal operating policies derived in this paper are based on the hydrologic, economic, and other inputs to the model. Reliability of the results depends upon the validity of the various assumptions explicitly stated or implied in the formulation and solution of the model. hydrologic inputs that may introduce errors are the storagedepth relations. These relations imply maximum and minimum permissible depths of pumping, average hydraulic parameters of the aquifers, and aquifer area. Optimal policies are strongly influenced by the objective function, which in turn is made up of values added for different crops for different amounts of water applied, and the pumping costs. The discount rate also influences the optimal policies. The error of discretization, which is introduced when the model is simplified from a continuous to a discrete form, can be further reduced by making the storage and pumpage intervals smaller and by increasing the number of recharges and associated probabilities used as input to the model. In conclusion, the optimal policies derived are optimal with respect to the limitations and constraints imposed on the model.

# Application of Results

The average depths to the water table in the shallow aquifer and to the potentiometric surface in the principal confined aquifer were estimated from the water-table maps and the potentiometric-surface maps, respectively, of the Roswell Basin for January of different years and are shown in table 24. Using these values and the storage-depth relations (equations 23 and 24), the storages in the two aquifers were estimated for different years and the optimal yearly pumpages from the two aquifers were then read from figures 21 and 22 and are shown in table 24. These pumpages are for growing alfalfa and cotton only; acreages are shown in table 23. To these pumpages, 80,000 acre-feet of water is added which is to be used for growing sorghum and small grains. Table 24 shows the total amount of groundwater and the average amount per acre to be pumped for growing all four crops.

The average optimal amount to be pumped varies between 4.4 and 4.8 acre-feet per acre. However, the court recently fixed the duty of water in the Roswell Basin at 3.0 acre-

Average elevation of water levels, storage, optimal pumpage from the two aquifers, and optimal duty of water in the Roswell Basin, New Mexico. Table 24.

Storage (106 acre-feet)         Pumpage (106 acre-feet)         Pumpage (103 acre-feet)         Pumpage (103 acre-feet)         Duty (106 acre-feet)           3.48         180         3,451         4.99         290         550         4.8           2.56         175         3,417         3.74         275         530         4.6           2.16         165         3,403         3.01         265         515         4.6           2.71         174         3,425         4.06         530         4.6		Sh	Shallow Aquifer	T;	Princip	Principal Confined Aquifer	Aquifer	Total Pumpage	
180       3,451       4.99       290       550         180       3,461       5.35       290       550         175       3,417       3.74       275       530         165       3,397       3.01       260       505         170       3,403       3.23       265       515         174       3,425       4.06       276       530	Average Elevation (feet)		Storage (10 <sup>6</sup> acre-feet)	Pumpage (10 <sup>3</sup> acre-feet)	Average Elevation (feet)	Storage (10 <sup>6</sup> acre-feet)	Pumpage (103 acre-feet)	for the Four Crops (103 acre-feet)	Duty (acre-feet per acre)
180       3,461       5.35       290       550         175       3,417       3.74       275       530         165       3,3397       3.01       260       505         170       3,403       3.23       265       515         174       3,425       4.06       276       530	3,407		3.48	180	3,451	4.99	290	550	4.8
175       3,417       3.74       275       530         165       3,3397       3.01       260       505         170       3,403       3.23       265       515         174       3,425       4,06       276       530	3,407		3.48	180	3,461	5.35	290	550	4.8
165     3,397     3.01     260     505       170     3,403     3.23     265     515       174     3,425     4,06     276     530	3,384	1	2.56	175	3,417	3.74	275	530	4.6
170 3,403 3.23 265 515 174 3,425 4.06 276 530	3,374		2.16	165	3,397	3.01	260	505	ħ•ħ
174 3,425 425 11.06 276 530	3,367		1.88	170	3,403	3.23	265	515	4.5
	Aver- age 3,388		2.71	47.	3,425	li .06	276	530	9° ħ

feet per water-right acre, plus 6.0 acre-inches per acre for carriage losses (State of New Mexico v. L. T. Lewis, et al., 1970; and State of New Mexico v. Hagerman Canal Company, et al., 1970). However, some parties reported that they historically have appropriated to beneficial use in excess of 4.0 acre-feet per acre. The average optimal pumpage per acre is higher than the actual pumpage recorded in the basin, owing to several reasons. Changes in the crop pattern will change the optimal pumpage. The values added for alfalfa and for cotton, the two main crops in the basin, used as input to the decision model, give maximum outputs at 7.33 and 3.75 acre-feet per acre, respectively. current practice in the Roswell Basin is to pump about 5.0 or 6.0 acre-feet per acre of alfalfa. The optimal average pumpage is per acre of harvested land, whereas the pumpage recorded by the office of the New Mexico State Engineer is for the water-right acreage, which includes the fallow land as well as the planted acreage. The water-right acreage is usually greater than the harvested land in the basin, therefore the duty calculated by the office of the State Engineer is less than the optimal duty.

In the Roswell Basin, saline water has been encroaching from the northeast and east of the City of Roswell toward the city in the principal confined aquifer [Hood et al., 1960; and Spiegel, 1967]. The rate of encroachment increases as the net withdrawal of water from the aquifer in the Roswell area increases. This constraint was not incorporated in the decision model because of lack of information. This constraint, if imposed on the model, would probably decrease the operating policies for the aquifer.

### Chapter 5

#### CONCLUSION AND RECOMMENDATIONS

# Conclusion

The problem of optimal operation of two coupled leaky aquifers and a surface-water subsystem is approached through stochastic dynamic programming. The three-dimensional dynamic programming model is simplified to a two-dimensional model by not incorporating the surface-water system as an independent state variable. This is a fairly valid simplification for the Roswell Basin because the surface water used in the basin is less than 4 percent of the total amount of water used for irrigation in the basin. Only two crops, alfalfa and cotton, which comprise about 80 percent of the total irrigated acreage in the basin, are explicitly considered in the model.

The optimal operating policies for the shallow aquifer and the principal confined aquifer are derived from the solution of the model while taking into consideration not only the mutual leakage of the aquifers, stochastic recharges to the aquifers, and natural discharge from the shallow aquifer, but also the physical characteristics of the system, the extent of the areas to be irrigated, and the additional pumping costs associated with the self-drawdown of the wells. The optimal operating policies are influenced by the values added to the basin by different crops, by the amounts of water in storage in the two aquifers, by the discount rate, and by mutual leakage of the aquifers. of these factors tend to increase while others tend to decrease the optimal operating policies. Leakage tends to change its effect -- that is, to increase or decrease the optimal policy--depending on the storages in the aquifers. The result of all these factors determines the final optimal operating policies. The policies are optimal with respect to the limitations and constraints imposed on the model.

The leakage from one aquifer to the other strongly influences optimal operating policies for the two aquifers. It is concluded here that the optimal operating policies derived for a coupled leaky-aquifer system, without incorporating leakage from one aquifer to the other, will not maximize the value added to the basin. The aquifers must be utilized conjunctively.

## Recommendations

The first requirement for optimal utilization of any system is the identification of the system. Identification of a system means that all of its dimensions, and all components of the system and their characteristics and mutual interactions are explicitly known. Under these conditions it should be possible to predict the response of the system to any expected stresses on it.

The following measures, some of which were recommended by Hantush [1955], would be helpful in optimal management of the water resources of the Roswell Basin:

- 1. Detailed geologic investigations are needed. Specifically, the relation of the geologic structures to the potentiometric surface in the southern half of the basin needs to be studied.
- 2. The potentiometric-surface and water-table maps lack control in the undeveloped, and especially the south-western, parts of the basin. Since wells for water-level measurements are not available in these areas, a suitable number of wells should be drilled there.
- 3. Recharge to the principal confined aquifer should be determined periodically by independent hydrogeologic methods. This calls for a study of the infiltration rates in the intake area and for a detailed study of the intake area itself.
- 4. Replenishment to the shallow aquifer from return flows should be measured on some typical farms in different parts of the basin.
- 5. Hydraulic gradients in the intake area and in the shallow aquifer near the Pecos River should be measured periodically.
- 6. Recorders should be installed on selected wells penetrating the shallow aquifer in different parts of the basin.
- 7. Water-table and potentiometric-surface maps should be constructed for periods of low water levels near the end of the pumping season.
- 8. Amounts of groundwater and surface water consumed by the salt cedars in the basin should be estimated periodically.
- 9. Cooperation in planning and working among all interested organizations and agencies should be increased.

An independent technical expert should help in guiding, planning, and coordinating activities of the various bodies concerned.

- 10. Digital and/or analog model studies of the basin should be conducted for the refinement and verification of the hydraulic characteristics of the aquifers and aquitard.
- ll. Estimates of crop yield as a function of water applied should be made in more detail and verified experimentally. Data are lacking on crop yield when excessive water is applied. Optimal withdrawals of water from the aquifers, determined by a model like the one developed in this report, depend on these relations.
- 12. Optimal operating policies for the Roswell Basin should be derived again after a few years when more refined inputs to the model become available.

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APPENDICES

### Appendix A

DRAWDOWN DISTRIBUTION RESULTING FROM WELL FIELDS
IN COUPLED LEAKY AQUIFER SYSTEMS

### Introduction

The drawdown distribution caused by several pumping wells in an area can be more conveniently calculated by assuming that the pumping of the well field is uniformly distributed over the area. Analytical expressions for the response to pumping of well fields in single aquifers are given for a limited number of cases by Hantush [1964]. Analytical expressions for the formation of groundwater mounds in flow systems caused by uniform percolation can also be used for predicting the effect of operation of well fields by changing the sign of the quantities representing recharge rates. Baumann [1952], Bittinger and Trelease [1960], Glover [1961], and Hantush [1963, 1967], among others, have obtained solutions for buildup of mounds caused by uniform percolation.

This paper discusses the prediction of response to simultaneous pumping of well fields in two coupled leaky aquifers, one confined and the other unconfined. Pumping from one or both of the aquifers can be negative or zero, or one aquifer can be pumped while the other is being replenished.

### Analysis

A schematic representation of well fields in a coupled leaky aquifer system is shown in figure A-1. Unless otherwise stated, the following assumptions are implied in the derivations [see Hantush, 1967]: (1) The aquifers are effectively infinite in areal extent and are homogeneous, isotropic, and resting on a horizontal impermeable bed; (2) all the wells are completely penetrating; (3) the aquifer parameters remain constant with time and in space; (4) contrasts between the hydraulic conductivities of the two aquifers and the semipervious layer are so great that the flow is essentially vertical in this layer and horizontal in the two aquifers; (5) the storativity of the semipervious layer is negligible; (6) the well fields are operated at constant rates and the distribution of pumpage from the two aquifers is uniform over the two concentric circular well field areas of radii  $R_1$  and  $R_2$ .

Two related cases of the problem are analyzed. In the first case, the induced drawdown in the upper aquifer, resulting from the operation of well fields, is small relative to its saturated thickness. This actually represents the case

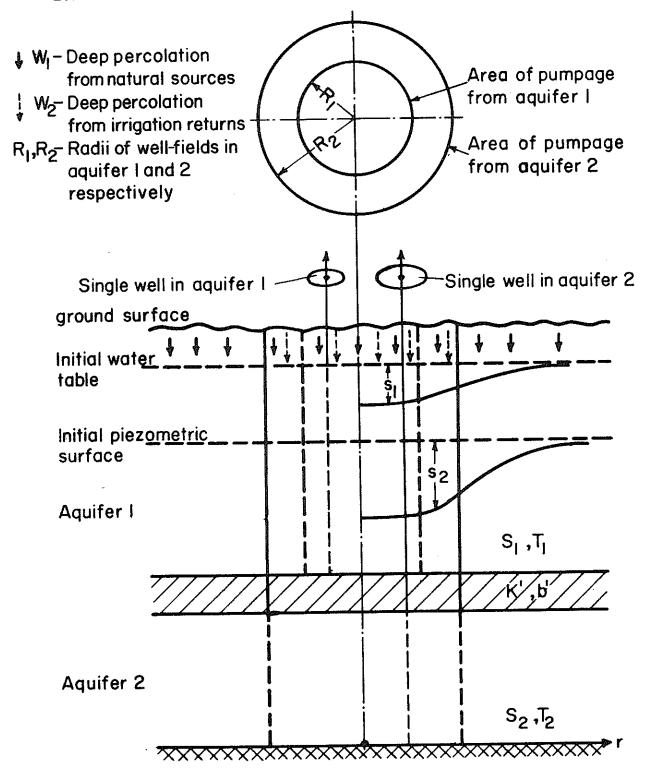


Figure A-1. Diagrammatic representation of well fields in coupled leaky aquifers.

when the two aquifers are confined. In the second case the drawdown in the upper aquifer, which is unconfined, is significant relative to its depth of saturation.

## Case I, Aquifer System Infinite

The problem is to determine the distribution of drawdowns, within and outside the radii of well fields in the coupled leaky aquifer system, as induced by the operation of one or both of the well fields. In addition to pumpage, aquifer I receives deep percolation from precipitation and from irrigation returns. The initial head distributions in the two aquifers can be different, that is, leakage may be taking place from one aquifer to the other. It is assumed here, in addition to the assumptions previously stated, that the induced drawdown in the unconfined aquifer is small relative to its saturated thickness. If both aquifers are confined, the upper aquifer does not receive deep percolation from precipitation and from irrigation returns.

It can be shown [see Hantush, 1967] that the flow system can be approximated by the following boundary value problem:

$$\frac{\partial^2 s_1}{\partial r^2} + \frac{1}{r} \frac{\partial s_1}{\partial r} - \frac{1}{B_1^2} (s_1 - s_2) + \frac{Q_1}{T_1} f_1(r) - \frac{W_1}{T_1} f_1(r)$$

$$-\frac{W_2}{T_1}g(r)-\frac{W_p}{T_1}f_2(r)=\frac{1}{v_1}\frac{\partial s_1}{\partial t}$$
 (1)

$$\frac{\partial^2 \mathbf{s}_2}{\partial \mathbf{r}^2} + \frac{1}{\mathbf{r}} \frac{\partial \mathbf{s}_2}{\partial \mathbf{r}} + \frac{1}{\mathbf{B}_2^2} (\mathbf{s}_1 - \mathbf{s}_2) + \frac{Q_2}{T_2} g(\mathbf{r}) = \frac{1}{v_2} \frac{\partial \mathbf{s}_2}{\partial \mathbf{r}}$$
 (2)

$$s_1(r,0) = s_1(r,0) = 0$$
 (3)

$$s_1(\infty, t) = s_2(\infty, t) = 0$$
 (4)

$$\partial s_1(0,t)/\partial r = \partial s_2(0,t)/\partial r = 0$$
 (5)

in which

$$s_1 = (h_{11}-h_1), \text{ and } s_2 = (h_{12}-h_2)$$
 (6)

$$v_n = T_n/S_n, \quad B_n^2 = T_n/(K'/b'), \quad n = 1, 2$$
 (7)

where subscripts 1 and 2 correspond to the aguifers 1 and 2 respectively;  $\underline{T}$ ,  $\underline{S}$ ,  $\underline{\nu}$  and  $\underline{h}$  are, respectively, transmissivity, storativity, hydraulic diffusivity, and the hydraulic head;  $h_{11}$  and  $h_{12}$  are the initial heads, and  $s_1$  and  $s_2$  are the drawdowns caused by operation of the well fields in the system; K'/b' is the leakage coefficient or leakance;  $\Omega_1$  and  $\Omega_2$  denote constant rates of withdrawal per unit area;  $W_1$  and  $W_2$  are the replenishment rates from irrigation returns from the two well fields, and  $W_1$  is recharge from precipitation to aquifer 1;  $R_1$  and  $R_2$  are radii of the well fields;  $R_1$  is the radius of the circular area around the well fields receiving recharge from precipitation; and  $\underline{r}$  is the radial distance from the center of the well fields to any point in the flow field. The other symbols are defined below:

#### Solution

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Applying successively Laplace transform with respect to  $\underline{t}$ , and infinite zero order Hankel transform with respect to  $\underline{r}$ , to the boundary value problem and after simplification one obtains:

$$\bar{F}_1(\alpha, p) = \{Q_1R_1J_1(\alpha R_1) - W_1R_1J_1(\alpha R_1) - W_2R_2J_2(\alpha R_2)\}$$

$$- W_{p}R_{e}J_{1}(\alpha R_{e}) \} \cdot \{ \frac{v_{1}(p+a_{2}v_{2})}{T_{1}\alpha p[(p+a)^{2}-\beta^{2}]} \} + \frac{Q_{2}v_{1}v_{2}R_{2}J_{1}(\alpha R_{2})}{\alpha T_{2}B_{1}^{2}p[(p+a)^{2}-\beta^{2}]}$$
(11)

$$\overline{F}_{2}(\alpha,p) = \frac{Q_{2}R_{2}v_{2}(a_{1}v_{1}+p)J_{1}(\alpha R_{2})}{T_{2}\alpha p[(p+a)^{2} - \beta^{2}]} + \{Q_{1}R_{1}J_{1}(\alpha R_{1}) - W_{1}R_{1}J_{1}(\alpha R_{1}) - W_{2}R_{2}J_{1}(\alpha R_{2}) - W_{p}R_{e}J_{1}(\alpha R_{e})\}$$

$$+ \{\frac{v_{1}v_{2}}{T_{1}\alpha B_{2}^{2}p[(p+a)^{2} - \beta^{2}]}\}$$
(12)

where  $\overline{F}_1(\alpha,p)$  and  $\overline{F}_2(\alpha,p)$  are the zero order Hankel transforms of the Laplace transforms of the variables  $s_1(r,t)$  and  $s_2(r,t)$ , respectively;  $\alpha$  and p are the parameters of the respective transformations and

$$a_1 = \alpha^2 + 1/B_1^2$$
,  $a_2 = \alpha^2 + 1/B_2^2$  (13)

$$a = 0.5(a_1v_1 + a_2v_2) = 0.5[\alpha^2(v_1 + v_2) + v_1/B_1^2 + v_2/B_2^2]$$
 (14)

$$\beta^2 = 0.25 \left[\alpha^2 (\nu_1 - \nu_2) + \nu_1 / B_1^2 - \nu_2 / B_2^2\right]^2 + \nu_1 \nu_2 / (B_1^2 B_2^2)$$
 (15)

$$(a^2 - \beta^2) = v_1 v_2 \alpha^2 (\alpha^2 + 1/B_1^2 + 1/B_2^2)$$
 (16)

Applying the inverse transforms [Erdelyi, 1954] to (11) and (12), we obtain:

$$s_{1}(r,t) = \frac{v_{1}v_{2}}{T_{1}} \int_{0}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha R_{1}) - W_{2}R_{2}J_{1}(\alpha R_{2}) - W_{p}R_{e}J_{1}(\alpha R_{e})\}$$

$$\cdot \{\frac{\alpha a_{2}}{(a^{2}-\beta^{2})} [1-e^{-at}(\cosh\beta t + \frac{aa_{2}v_{2}-a^{2}+\beta^{2}}{a_{2}v_{2}\beta} \sinh\beta t)]\}J_{0}(\alpha r)d\alpha$$

$$+\frac{Q_2v_1v_2R_2}{T_2B_1^2}\int_0^\infty \frac{\alpha}{(a^2-\beta^2)}\left\{1-e^{-at}(\cosh\beta t + \frac{a}{\beta}\sinh\beta t)\right\}J_1(\alpha R_2)J_0(\alpha r)d\alpha \qquad (17)$$

$$s_{2}(r,t) = \frac{Q_{2}R_{2}\nu_{1}\nu_{2}}{T_{2}} \int_{0}^{\infty} \frac{\alpha a_{1}}{(a^{2}-\beta^{2})} \{1-e^{-at}(\cosh\beta t + \frac{aa_{1}\nu_{1}-a^{2}+\beta^{2}}{a_{1}\nu_{1}\beta} \sinh\beta t)\}$$

$$\cdot J_{1}(\alpha R_{2})J_{0}(\alpha r)d\alpha + \int_{0}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha R_{1}) - W_{2}R_{2}J_{1}(\alpha R_{2}) - W_{p}R_{e}J_{1}(\alpha R_{e})\}$$

$$\cdot \left\{ \frac{\alpha v_1 v_2}{T_1 B_2^2 (a^2 - \beta^2)} \left[ 1 - e^{-at} \left( \cosh \beta t + \frac{a}{\beta} \sinh \beta t \right) \right] \right\} J_0(\alpha r) d\alpha$$
 (18)

where  $J_0$  and  $J_1$  are the zero and first order Bessel functions of the first kind, respectively.

## Case II, Aquifer System Infinite

An Unconfined Adulfer and a Confined Adulfer -- The problem analyzed in this section is essentially the same as the problem of the last section except that in the upper, unconfined, aquifer the induced drawdown is significant relative to its depth of saturation. All the assumptions applied to the last case are also implied here. The drawdown distribution for the system is given by equations 1 through 5 except that equations 1 and 2, the differential equations governing the flow in the two adulfers, are now different.

The differential equation governing the flow in an unconfined aquifer coupled to a confined aquifer can be approximated from continuity considerations as follows:

$$\nabla^{2}h_{1}^{2} - \frac{2 K'}{b'K_{1}} (h_{1} - h_{2}) = \frac{2\theta}{K_{1}} \frac{\partial h_{1}}{\partial t}$$
 (19)

where

$$\nabla^2 h_1^2 = \frac{\partial^2 h_1^2}{\partial r^2} + \frac{1}{r} \frac{\partial h_1^2}{\partial r}$$
 (20)

and  $\theta$  and  $K_1$  are specific yield and hydraulic conductivity of the unconfined aquifer, respectively;  $h_1$  and  $h_2$  are hydraulic heads in the unconfined and the confined aquifer, respectively. Other symbols were defined earlier.

Equation 19 is nonlinear in  $\underline{h}$  and is difficult to solve. By replacing  $(h_{i1}+h_1)/2$  with  $\underline{D}$ , a weighted mean depth of the flow profile, one derives the following differential equations

from equation 19, for the case when the upper aquifer is unconfined:

$$\frac{\partial^2 s_1}{\partial r^2} + \frac{1}{r} \frac{\partial s_1}{\partial r} - \frac{1}{\beta_1^2} \left( \frac{s_1}{2D} - s_2 \right) + \frac{2(Q_1 - W_1)}{K_1} F_1(r)$$

$$-\frac{2W_2}{K_1}g(r) - \frac{2W_p}{K_1}F_1(r) = \frac{1}{v_1}\frac{\partial s_1}{\partial t}$$
 (21)

$$\frac{\partial^2 \mathbf{s}_2}{\partial \mathbf{r}^2} + \frac{1}{\mathbf{r}} \frac{\partial \mathbf{s}_2}{\partial \mathbf{r}} + \frac{1}{B_2^2} \left( \frac{\mathbf{s}_1}{2D} - \mathbf{s}_2 \right) + \frac{\Omega_2}{T_2} g(\mathbf{r}) = \frac{1}{\nu_2} \frac{\partial \mathbf{s}_2}{\partial \mathbf{t}}$$
 (22)

in which

$$s_1 = (h_{11}^2 - h_1^2), \text{ and } s_2 = (h_{12} - h_2)$$
 (23)

$$\beta_1^2 = K_1/2(K'/b'), \qquad v_1 = K_1D/\theta.$$
 (24)

### Solution

The flow problem represented by equations 21 and 22 and equations 3 through 5 is solved exactly like the previous problem.

$$s_{1}(r,t) = \frac{2v_{1}v_{2}}{K_{1}} \int_{0}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha R_{1}) - W_{2}R_{2}J_{1}(\alpha R_{2}) - W_{p}R_{e}J_{1}(\alpha R_{e})\}$$

• { 
$$\frac{\alpha a_2}{(a^2-\beta^2)}$$
 [1-e<sup>-at</sup>(cosh\beta t +  $\frac{aa_2v_2-a^2+\beta^2}{a_2v_2\beta}$  sinh\beta t)]} J (\alpha r) d\alpha

$$+\frac{Q_2v_1v_2R_2}{T_2\beta_1^2}\int_0^\infty \frac{\alpha}{(a^2-\beta^2)} \left\{1-e(\cosh\beta t + \frac{a}{\beta}\sinh\beta t)\right\}J_1(\alpha R_2)J_0(\alpha r)d\alpha \qquad (25)$$

$$s_{2}(r,t) = \int_{0}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha R_{1}) - W_{2}R_{2}J_{1}(\alpha R_{2}) - W_{p}R_{e}J_{1}(\alpha R_{e})\}$$

$$\cdot \left\{ \frac{\alpha v_1 v_2}{B_2^2 K_1 D(a^2 - \beta^2)} \left[ 1 - e^{-at} \left( \cosh \beta t + \frac{a}{\beta} \sinh \beta t \right) \right] \right\} J_0(\alpha r) d\alpha$$

$$+ \frac{Q_{2}v_{1}v_{2}R_{2}}{T_{2}} \int_{0}^{\infty} \frac{\alpha a_{1}}{(a^{2}-\beta^{2})} \left\{1 - e^{-at}(\cosh\beta t + \frac{aa_{1}v_{1} - a^{2} + \beta^{2}}{a_{1}v_{1}\beta} \sinh\beta t)\right\} J_{1}(\alpha R_{2}) J_{0}(\alpha r) d\alpha$$
(26)

Equations 25 and 26 are guite similar to equations 17 and 18 except that now

$$v_1 = KD/\theta; \quad B_1^2 = K_1D/(K'/b'); \quad \beta_1^2 = K_1/2(K'/b')$$
 (27)

$$a_1 = \alpha^2 + \frac{1}{K_1 D/(K'/b')}$$
 (28)

Equations 15 and 16 are to be redefined in view of equations 27 and 28;  $\mathbf{s}_1$  and  $\mathbf{s}_2$  are defined by equation 23.

## Special Solutions

The solutions for special cases can be derived from the solutions derived for the two cases described previously.

One Well Field Operating -- If only one of the well fields is operating, say for example the well field in aquifer 1, the expressions for drawdowns in the two aquifers for cases I and II are obtained by substituting  $Q_2 = 0$ , in the respective solutions. The fractions of total drawdowns which are due to pumpage  $Q_2$  from aquifer 2 will vanish. It should be noted that the drawdown in the aquifer which is not being operated is not zero. There will be some induced drawdown because of pumpage from the other aquifer. This is because the two aquifers are coupled through the semipervious layer.

Well Field in a Single Aquifer -- In the case of a single isolated aquifer, the drawdown due to the operation of a well field is obtained by letting  $1/B_1^2$ ,  $1/B_2^2$ , and  $Q_2$  approach zero and by putting  $v_1 = v_2 = v$  in the appropriate solutions. For example, from the solution for case II, by making the above substitutions in equations 25 and 26, we get

$$s (r,t) = \frac{2}{K} \int_{0}^{\infty} \{(Q_1 - W_1)RJ_1(\alpha R) - W_pR_eJ_1(\alpha R_e)\} \cdot (1 - e^{-\alpha^2 vt}) \frac{J_0(\alpha r)}{\alpha^2} d\alpha$$
 (29)

Equation 29 gives the drawdown caused by the operation of a well field of radius  $\underline{R}$  in an unconfined aquifer of hydraulic conductivity  $\underline{K}$ . The aquifer receives deep percolation from precipitation and from irrigation returns. If, instead of pumping, the aquifer is receiving recharge at the rate V, equation 29 becomes

$$h^{2}-h_{1}^{2} = \frac{2V}{\pi K} \int_{0}^{\infty} (1-e^{-qr^{2}}) J_{1}(r) J_{0}(\rho r) \frac{dr}{r^{2}} = (2V/\pi K) f(q,\rho)$$
 (30)

in which

$$V = \pi R^2 W$$
,  $q = \nu t/R^2$ ,  $\rho = r/R$ .

Equation 30 is the same as derived by Hantush [1967] for a single aquifer.

Maximum Drawdown -- The maximum drawdown due to the operation of well fields occurs at the center of well fields. The expressions for maximum drawdowns for the various cases can be derived by putting r=0 in the appropriate solution. When r=0,  $J_0(0)=1$  and so  $J_0(\alpha r)$  is replaced with unity in the solutions.

Effect of Boundaries -- If there are hydraulic boundaries near the well fields, the solutions can be derived by the method of images [Jacob, 1950; Carslaw and Jaeger, 1959] from the above solutions.

Well Fields in Closed Circular Aquifers

The coupled aquifers analyzed here are finite and have zero flux across their circular boundaries. The flow problems for both case I and case II are then described by equations 1 through 5, and 21 through 24 except that equation 4 is replaced with the following boundary condition:

$$\partial s_1(r_e, t)/\partial r = \partial s_2(r_e, t)/\partial r = 0$$
 (31)

where  $r_{\rm e}$  is the radius of the aquifer system.

#### Solution

In order to solve the flow problem, the zero-ordered Hankel transform of a function f(r) is modified and defined as

$$H_0[f(r)] = V(\alpha_n) = \frac{\sqrt{2}}{r_e} \int_0^{r_e} rf(r) \frac{J_0(\alpha_n r/r_e)}{J_0(\alpha_n)} dr$$
 (32)

where <u>n</u> is positive roots of  $J_0'(\alpha)=0$ . By expanding a function in a Fourier-Bessel series, it can be shown that the inversion formula for the transformation represented by equation 32 is

$$f(r) = \frac{\sqrt{2}}{r_e} \sum_{m=1}^{\infty} \frac{V(\alpha_n) J_0(\alpha_n r/r_e)}{J_0(\alpha_n)}$$
(33)

where  $V(\alpha_\eta)$  is the transform of the function f(r) defined by equation 32 and  $J_0$  is the zero-order Bessel function of the first kind.

Applying equation 32 to  $\nabla^2 \overline{s}$  and on integration by parts, one obtains

$$H_{0}[\nabla^{2}\overline{s}] = H_{0}\left[\frac{1}{r}\frac{d}{dr}\left(r\frac{d\overline{s}}{dr}\right)\right] = \left[r\frac{d\overline{s}}{dr}\cdot\frac{J_{0}(\alpha_{n}r/r_{e})}{J_{0}(\alpha_{n})}\right]_{r=0}^{r=r_{e}}$$

$$-\int_{0}^{r_{e}} r\frac{d\overline{s}}{dr}\cdot\frac{\alpha_{n}}{r_{e}}\frac{J_{0}(\alpha_{n}r/r_{e})}{J_{0}(\alpha_{n})}dr \qquad (34)$$

The first term on the right hand side vanishes if  $(d\bar{s}/dr) = 0$  or  $r=r_e$ 

$$H_0\left[\frac{1}{r}\frac{d}{dr}\left(r\frac{d\overline{s}}{dr}\right)\right] = -\left[\overline{s} \ r\frac{\alpha_n}{r_e}\frac{J_0(\alpha_n r/r_e)}{J_0(\alpha_n)}\right]_{r=0}^{r=r_e}$$

$$+\frac{\alpha_{\rm n}}{r_{\rm e}}\int_{0}^{r_{\rm e}}\frac{\bar{s}}{J_0(\alpha_{\rm n})}\left[J_0(\alpha_{\rm n}r/r_{\rm e})+\frac{\alpha_{\rm n}}{r_{\rm e}}J_0''(\alpha_{\rm n}r/r_{\rm e})\right]dr\tag{35}$$

Again the first term on the right side vanishes because  $J_1(\alpha_n)=0$  and using the fact that  $J_0(\alpha_n r/r_e)$  satisfies the differential equation

$$J_0''(\alpha_n r/r_e) + \frac{r_e}{\alpha_n r} J_0'(\alpha_n r/r_e) + J_0(\alpha_n r/r_e) = 0$$
 (36)

Equation 35 becomes

$$H_0\left[\frac{1}{r}\frac{d}{dr}\left(r\frac{d\overline{s}}{dr}\right)\right] = -\frac{\alpha_n^2}{r_e^2}\overline{F}(\alpha_n)$$
 (37)

in which  $\overline{F}(\alpha_n)$  is the integral transform, defined by equation 32, of  $\overline{s}(r)$ .

# Case I, Aquifer System Finite

Applying the Laplace transform and then the Bessel transform, defined by equation 32, to the flow problem defined by equations 1 through 7, with equation 4 replaced by equation 31, and simplifying, one obtains

$$\vec{F}_1(\alpha_n, p) = \{(Q_1 - W_1)R_1J_1(\alpha_nR_1/r_e) - W_2R_2J_1(\alpha_nR_2/r_e) - W_pr_eJ_1(\alpha_n)\}$$

$$\cdot \left\{ \frac{\sqrt{2} \, \nu_1 \, (p+d_2\nu_2)}{T_1\alpha_n J_0(\alpha_n) p [(p+d)^2 - \beta^2]} \right\} + \frac{Q_2 \, \sqrt{2} \, R_2 J_1(\alpha_n R_2/r_e) \nu_1 \nu_2}{T_2\alpha_n J_0(\alpha_n) B_1^2 p [(p+d)^2 - \beta^2]}$$
(38)

$$\bar{F}_2(\alpha_n,p) = \{(Q_1 - W_1)R_1J_1(\alpha_nR_1/r_e) - W_2R_2J_1(\alpha R_2/r_e) - W_pr_eJ_1(\alpha_n)\}$$

• { 
$$\frac{\sqrt{2} v_1 v_2}{B_2^2 \alpha_n T_1 J_0(\alpha_n) p[(p+d)^2 - \beta^2]}$$
 } +  $\frac{\sqrt{2} Q_2 R_2 v_2 J_1(\alpha_n R_2/r_e) (p+d_1 v_1)}{T_2 \alpha_n J_0(\alpha_n) p[(p+d)^2 - \beta^2]}$  (39)

where  $F_1(\alpha_n,p)$  and  $F_2(\alpha_n,p)$  are the Bessel transforms defined by equation 32, of the Laplace transforms of the variables  $s_1(r,t)$  and  $s_2(r,t)$  respectively;  $\alpha_n$  and p are the parameters of respective transformations and

$$d_1 = \alpha_n^2/r_e^2 + 1/B_1^2, \qquad d_2 = \alpha_n^2/r_e^2 + 1/B_2^2$$
 (40)

$$d = 0.5 (d_1 v_1 + d_2 v_2)$$
 (41)

$$\mu^{2} = 0.25 \left[\alpha_{D}^{2}(v_{1}-v_{2})/r_{e}^{2} + v_{1}/B_{1}^{2} - v_{2}/B_{2}^{2}\right]^{2} + v_{1}v_{2}/(B_{1}^{2}B_{2}^{2})$$
 (42)

(43)

 $\alpha_n$  are positive roots of  $J_1(\alpha) = 0$ .

Applying the inverse Laplace transform and the inverse Bessel transform, defined in equation 33, to equations 38 and 39, we obtain:

$$s_{1}(r,t) = \frac{2}{r_{e}} \sum_{n=1}^{\infty} \{ (Q_{1} - W_{1}) R_{1} J_{1}(\alpha_{n} R_{1} / r_{e}) - W_{2} R_{2} J_{1}(\alpha_{n} R_{2} / r_{e}) - W_{p} r_{e} J_{1}(\alpha_{n}) \}$$

$$\cdot \{ \frac{d_{2} v_{1} v_{2}}{T_{1} \alpha_{n} (d^{2} - \mu^{2})} [1 - e^{-dt} (\cosh \mu t + \frac{d d_{2} v_{2} - d^{2} + \mu^{2}}{d_{2} v_{2} \mu} \sinh \mu t)] \}$$

$$\frac{J_{0}(\alpha_{n} r / r_{e})}{J_{0}^{2}(\alpha_{n})} + \frac{2}{r_{e}} \sum_{n=1}^{\infty} \frac{Q_{2} R_{2} v_{1} v_{2} J_{1}(\alpha_{n} R_{2} / r_{e})}{T_{2} B_{1}^{2} \alpha_{n} (d^{2} - \mu^{2})}$$

$$[1 - e^{-dt} (\cosh \mu t + \frac{d}{\mu} \sinh \mu t)] \frac{J_{0}(\alpha_{n} r / r_{e})}{J_{0}^{2}(n)}$$

$$(43)$$

$$s_{2}(r,t) = \frac{2}{r_{e}} \sum_{n=1}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha_{n}R_{1}/r_{e}) - W_{2}R_{2}J_{1}(\alpha_{n}R_{2}/r_{e}) - W_{p}r_{e}J_{1}(\alpha_{n})\}$$

• { 
$$\frac{v_1v_2}{B_2^2T_1\alpha_n(d^2-\mu^2)}$$
 [1-e<sup>-dt</sup>(cosh\u00e4t +  $\frac{d}{\mu}$  sinh\u00e4t)]}

$$\cdot \frac{J_0(\alpha_n r/r_e)}{J_0^2(\alpha_n)} + \frac{2}{r_e} \sum_{n=1}^{\infty} \frac{Q_2 R_2 v_1 v_2 d_1 J_1(\alpha_n R_2/r_e)}{T_2 \alpha_n (d^2 - \mu^2)}$$

$$[1-e^{-dt}(\cosh\mu t + \frac{dd_1\nu_1 - d^2 + \mu^2}{d_1\nu_1\mu} \cdot \sinh\mu t)] \frac{J_0(\alpha_n r/r_e)}{J_0^2(\alpha_n)}. \tag{44}$$

## Case II, Aquifer System Finite

The flow problem is described by equations 21 through 24 and equations 3 through 5, with equation 4 replaced by equation 31. Approaching the problem exactly like the problem of Case I for the aquifer system finite, we obtain:

$$s_{2}(r,t) = \frac{2}{r_{e}} \sum_{n=1}^{\infty} \{(Q_{1}-W_{1})R_{1}J_{1}(\alpha_{n}R_{1}/r_{e})-W_{2}R_{2}J_{1}(\alpha_{n}R_{2}/r_{e})-W_{p}r_{e}J_{1}(\alpha_{n})\}$$

• { 
$$\frac{v_1v_2}{B_2^2K_1D\alpha_n(d^2-\mu^2)}$$
 [1-e<sup>-dt</sup>(cosh\(\mu\) +  $\frac{d}{\mu}$  sinh\(\mu\)]}

$$\frac{J_0(\alpha_n r/r_e)}{J_0^2(\alpha_n)} + \frac{2}{r_e} \sum_{n=1}^{\infty} \frac{Q_2 v_1 v_2 R_2 d_1 J_1(\alpha_n R_2/r_e)}{T_2 \alpha_n (d^2 - \mu^2)}$$

$$[1-e^{-dt}(\cosh\mu t + \frac{dd_1v_1 - d^2 + \mu^2}{d_1v_1\mu} \sinh\mu t)] \cdot \frac{J_0(\alpha_n r/r_e)}{J_0^2(\alpha_n)}$$
(46)

where d,  $d_1$  and  $d_2$  and  $\mu$  are defined by equations  $^{40}$  through  $^{42}$ .

### Conclusion

The drawdown distribution in any one member of a coupled leaky aquifer system is affected by the withdrawal of groundwater from either of the aquifers. The magnitude of the drawdown induced in one of the aquifers while pumping from another depends on the discharge, and on the hydraulic characteristics of aquifers and aquitards.

Solutions derived for the drawdown distribution due to discharge from either one or from both well fields in a coupled system of two aquifers and an aquitard can be numerically evaluated on a digital computer. It is not practical to tabulate the integrals that occur in the solutions because several parameters are involved.

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#### Appendix B

TIME OF TRAVEL FOR AN IMPULSE RESULTING FROM AN INSTANTANEOUS LINE SOURCE IN LEAKY AND NONLEAKY AQUIFERS

### Introduction

The idea of an instantaneous source comes from the theory of heat flow and was successfully used by Thomson (Lord Kelvin) [1883]. When a finite amount of heat is instantaneously released in each unit of area of a plane surface in a body, this surface becomes an instantaneous source of heat. In this paper the idea of an instantaneous source of water is used and a calculation is made of the time taken by an impulse due to such a source to propagate through a leaky aquifer and a nonleaky aquifer.

#### Analysis

Consider a coupled leaky aquifer system comprised of two aquifers separated by an aquitard. Unless otherwise stated, the following assumptions are implied in the derivations: (1) The aquifers are effectively infinite in area and are homogeneous, isotropic, and resting on a horizontal impermeable bed; (2) the aquifer parameters remain constant with time and in space; (3) contrasts between the hydraulic conductivities of the two aquifers and the semipervious layer are so great that the flow is essentially vertical in this layer and horizontal in the two aquifers; (4) the storativity of the semipervious layer is negligible; and (5) effects of withdrawal of groundwater from the aquifers are not considered.

The geohydrologic diffusion equation governing the flow of groundwater in an aquifer is [as, for example, Hantush, 1964]:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{1}{v} \frac{\partial h}{\partial t}$$
 (1)

where h is the hydraulic head and  $\nu$  is the hydraulic diffusivity and is equal to T/S, transmissivity over storativity. The solution of the diffusion equation for an instantaneous point source of heat is given by Carslaw and Jaeger [1959]. They have also derived the solution for an instantaneous line source by integrating the point sources along a line. By analogy, the head in an infinite aquifer due to an instantaneous line source is

$$h = \frac{V}{4\pi Tt} \exp(-\frac{a^2}{4\nu t})$$
 (2)

where <u>a</u> is the distance from the line source,  $\underline{V}$  is the amount of water released, and  $\underline{t}$  is the time since the release of water. Following the procedure of Carslaw and Jaeger [1959], it can be shown that the head due to an instantaneous line source in a leaky aquifer is given by

$$h = \frac{V}{4\pi Tt} \exp[-(u + \frac{a^2}{hB^2u})]$$
 (3)

where  $u = \frac{a^2}{4vt}$  and  $B^2 = T/(K'/b')$ , B is the leakage factor, (K'/b') is the leakage coefficient or leakance, and K' and b' are the hydraulic conductivity and the thickness of the aquitard, respectively.

The time taken by the head due to an instantaneous line source to reach its maximum value at a distance <u>a</u> from the line source is calculated from equation (3) by equating to zero the first derivative of the head with respect to time:

$$\frac{v}{B^2} t^2 + t - \frac{a^2}{4v} = 0 \tag{4}$$

Solving equation (4) for  $\underline{t}$  and choosing the positive solution,

$$t = \frac{B^2}{2\nu} \left[ \sqrt{1 + \frac{a^2}{B^2}} - 1 \right]$$
 (5)

The time t for a nonleaky aquifer is derived from equation (4) by letting 1/B approach zero.

$$t = a^2/4v \tag{6}$$

Application of Results to the Roswell Basin

The results derived in the preceding section were applied to the Roswell Basin. Using equation (5), travel times of an impulse due to an instantaneous line source in the shallow aquifer and in the principal confined aquifer were calculated for different areas of the basin and are shown in tables B-l and B-2, respectively. The hydraulic characteristics of the two aquifers and of the aquitard for different areas of the basin are summarized by Hantush [1955].

The travel time in the shallow aquifer varies from 3.2 to about 4.5 years for a distance of 6 miles, and the time varies from 4.2 to 7.5 years for a distance of 8 miles. Impulses travel fastest in the Roswell area and slowest in the Dexter area. The impulse travels at a much faster rate in the principal confined aquifer than in the shallow aquifer. The travel time in the principal confined aquifer

Table B-1. Time of travel of an impulse due to an instantaneous line source in the shallow aquifer of the Roswell Basin, New Mexico.

Area	Distance a	<u>a</u> Leakage Factor <u>B</u>	Hydraulic Diffusivity ⊻	Time <u>t</u>
	(miles)	(feet)	(feet <sup>2</sup> per day)	(days) (years)
Roswell	6 8	$9.5 \times 10^3$ $9.5 \times 10^3$	$133.7 \times 10^3$ $133.7 \times 10^3$	1,180 1,540 3.2 4.2
Dexter	6 8	40.0 x 10 <sup>3</sup> 40.0 x 10 <sup>3</sup>	$133.7 \times 10^3$ $133.7 \times 10^3$	1,650 4.5 2,720 7.5
Artesia	6 8	$20.0 \times 10^{3}$ $20.0 \times 10^{3}$	$133.7 \times 10^3$ $133.7 \times 10^3$	1,310 3.6 2,000 5.5
Lakewood	6 8	$36.0 \times 10^3$ $36.0 \times 10^3$	$133.7 \times 10^{3}$ $133.7 \times 10^{3}$	1,610 4.4 2,630 7.2

Table B-2. Time of travel of an impulse due to an instantaneous line source in the principal confined aquifer of the Roswell Basin, New Mexico.

Area	Distance		Hydraulic Diffusivity <u>v</u>	Time <u>t</u>
	(miles)	(1,000 feet)	(feet <sup>2</sup> per day)	(days) (years)
Roswell	6 8	9.5 9.5	$1.87 \times 10^{10}$ $1.87 \times 10^{10}$	0.0060 1.6 x 10 <sup>-5</sup> 0.0086 2.4 x 10 <sup>-5</sup>
Dexter	6 8	40.0 40.0	2.01 x 10 <sup>8</sup> 2.01 x 10 <sup>8</sup>	1.10 3.0 x 10 <sup>-3</sup> 1.81 5.0 x 10 <sup>-3</sup>
Artesia	6 8	20.0	4.01 x 10 <sup>8</sup> 4.01 x 10 <sup>8</sup>	0.44 1.2 x 10 <sup>-3</sup> 0.67 1.8 x 10 <sup>-3</sup>
Lakewood	l 6 8	36.0 36.0	8.82 x 10 <sup>7</sup> 8.82 x 10 <sup>7</sup>	$\begin{array}{cccccccccccccccccccccccccccccccccccc$

varies from about 10 minutes to 2.4 days for a distance of 6 miles.

The travel time in the intake area of the principal confined aquifer was calculated using equation (6). The time varies from about 2.0 to 3.7 years for a distance of 6 miles for hydraulic diffusivities of 334,000 and 187,000 feet<sup>2</sup> per day, respectively. The travel time in the intake area is of importance because it governs the recharge to the principal confined aquifer.

#### Discussion

The time of travel derived using equation (5) or (6) is the time for the head due to an instantaneous line source to reach its maximum at a given distance. Travel times derived for the two aquifers in the Roswell Basin are based on the hydraulic characteristics derived by Hantush [1955] from pumping tests that were carried out in the developed part of the basin. Travel time increases with increase in the leakage factor and decreases as the hydraulic diffusivity increases.

The hydraulic diffusivity of the principal confined aquifer is expected to decrease as one moves away from the developed part of the basin. The travel time per unit distance would increase as one moves toward the intake area of this aquifer.

#### Conclusions

Equations (5) and (6) can be used to calculate times of travel of an impulse due to an instantaneous line source in leaky and in nonleaky aquifers, respectively. For leaky aquifers, time of travel increases with increase in leakage factor and decreases as diffusivity increases. Time of travel is directly proportional to the square of the distance and inversely proportional to the hydraulic diffusivity for nonleaky aquifers.

The time of travel of an impulse due to a line source 6 miles from the observation point is about one day in the principal confined aquifer of the Roswell Basin, and about 3 to 7 years in the shallow aquifer. Times of travel for both aquifers are the shortest in the Roswell area and greatest in the Dexter area. The time of travel in the intake area of the Roswell Basin is about 2.0 to 3.7 years for a distance of six miles. The width of the area, from the confined-unconfined boundary of the principal confined aquifer to the western limit of water in this aquifer, is about 8 to 10 miles, just west of the developed areas.

Therefore, the use of three-year effective precipitation for the calculation of recharge to the principal confined aquifer of the Roswell Basin seems reasonable. This is supported by the study of hydrographs of the recorder wells in the principal confined aquifer.

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#### Appendix C

SALT WATER ENCROACHMENT IN A CONFINED LEAKY AQUIFER DUE TO A STEADILY OPERATING WELL FIELD

### Introduction

Salt water encroachment because of withdrawal of fresh water from aquifers is a problem occurring in several regions of the world. The movement of the interface between fresh water and salt water has been investigated extensively. Bear and Dagan [1964] and Hantush [1968] have made literature surveys in addition to their own studies of the different aspects of the interface. The discussion below presents an approximate approach to the lateral encroachment of salt water caused by operation of a well field in a confined leaky aquifer. This procedure was discussed by Hantush in one of his classes for a single well in a nonleaky aquifer.

### Analysis

Consider a coupled leaky aquifer system comprised of two aquifers separated by an aquitard. Unless otherwise stated, the following assumptions are implied in the derivations: (1) The aquifers are effectively infinite in areal extent and are homogeneous, isotropic, and resting on a horizontal impermeable bed; (2) contrasts between the hydraulic conductivities of the two aquifers and the semipervious layer are so great that the flow is essentially vertical in this layer and horizontal in the two aguifers; (3) the aquifer parameters remain constant with time and in space; (4) the storativity of the semipervious layer is negligible; (5) a well field is an area in which several pumping wells are located and where pumping is uniformly distributed over the area; (6) the induced drawdown in the aquifer which is not being pumped is negligible; and (7) the interface is sharp between fresh water and salt water.

Let  $\underline{R}$  be radius of the well field (figure C-1) that is being pumped steadily at a rate  $\underline{V}$ . Rate of movement of a particle of water is given by

$$\frac{\mathrm{d}\mathbf{r}}{\mathrm{d}\mathbf{t}} = \frac{\mathbf{K}}{\mathbf{n}} \frac{\partial \mathbf{h}}{\partial \mathbf{r}} \tag{1}$$

where <u>n</u> is the porosity of the aquifer, <u>K</u> is the hydraulic conductivity, and <u>h</u> is the hydraulic head in the aquifer above the base of the aquifer. The drawdown <u>s</u> in a leaky aquifer during steady operation of the well field is given by Hantush [1964], for r > R:

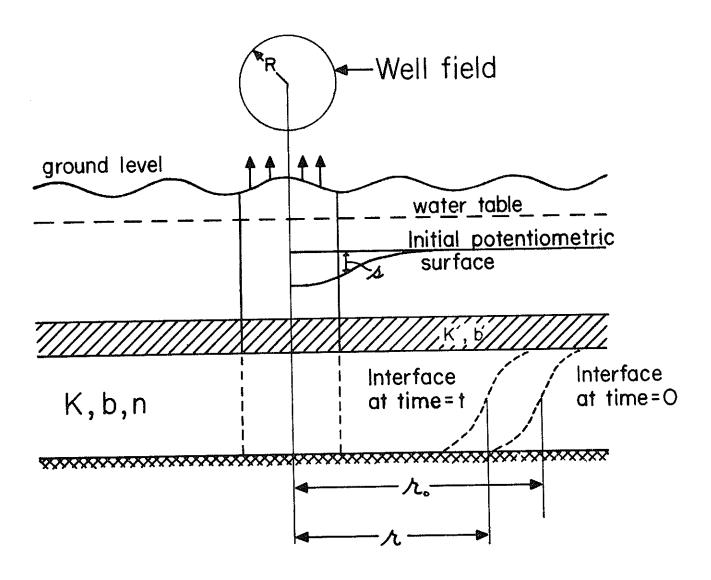


Figure C-1. Encroachment of the interface between fresh water and salt water owing to the operation of a well field in a leaky aquifer.

$$s(r,t) = \frac{VB}{\pi RT} I_1 (R/B) K_0(r/B)$$
 (2)

where  $s(r,t) = h_0 - h(r,t)$ , and  $h_0$  is the initial head,  $\underline{h}$  is the head at any time  $\underline{t}$  at a distance  $\underline{r}$ ,  $\underline{B}$ , is the leakage factor and is equal to  $\sqrt{T/(K'/b')}$ , K'/b' is the leakance or leakage coefficient, K' and b' are the vertical hydraulic conductivity and thickness of the aquitard, respectively,  $\underline{T}$  is the transmissivity and is equal to Kb,  $\underline{b}$  is the thickness of the aquifer,  $\underline{I}_1$  is the modified Bessel function of first kind and first order, and  $K_0$  is the modified Bessel function of second kind and zero order.

From equations (1) and (2)

$$\frac{d\mathbf{r}}{dt} = \frac{V}{\pi R n b} I_1(R/B) K_1(\mathbf{r}/B)$$
 (3)

where  $K_1$  is the modified Bessel function of second kind and first order.

Integrating equation (3) with respect to time as time goes from zero to  $\underline{t}$  and as  $\underline{r}$  goes from  $\underline{r}_0$  to  $\underline{r}$ :

$$\int_{0}^{t} dt = \frac{\pi Rnb}{VI_{1}(R/B)} \int_{r_{0}}^{r} \frac{dr}{K_{1}(r/B)}$$
(4)

where  $r_0$  is the initial position of the interface when t=0 (figure C-1). After simplification, equation (4) becomes

$$t = -\frac{\pi R n b B}{V I_1(R/E)} \left[ \int_0^{r_0/B} \frac{d\alpha}{K_1(\alpha)} - \int_0^{r/B} \frac{d\alpha}{K_1(\alpha)} \right], \text{ for } r > R.$$
 (5)

Equation (5) gives the time taken by the interface to travel from  $r_0$  to  $\underline{r}$ . The negative sign indicates that, as the distance from the center of the well field to the interface decreases, the time increases.

In order to determine the motion of the interface in a particular time, equation (4) is solved inversely for r using the values of the integral given in table C-1. The distance r is calculated for several points on each of the

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salinity contours, and plotted, thus giving the new position of the interface.

### Discussion

The procedure described is useful for predicting the encroachment of a salt water front in an aquifer and can be used for other flow systems for which analytical drawdown expressions are available. The procedures are only approximate in nature because some of the assumptions are not quite true in nature. The interface between salt water and fresh water is usually not sharp. For some flow systems, it may be difficult to simplify the final expression for time to a form such as equation (5) which can readily be evaluated.

### References, Appendix C

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Appendix D
DATA TABLES AND MAPS

Table D-1. Monthly Precipitation (in inches) at Roswell, New Mexico, 1905-1968.

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1952 1953 1954 1955 1956 1957 1958	0.21 0.24 0.21 0.29 0.02 0.02	0.21 0.49 0.0 0.0 1.42 0.64	0.13 0.25 0.0 0.10 0.03 0.80	1.02 0.72 0.11 0.19 0.03 0.31	0.30 0.70 2.56 0.41 0.40	0.14 0.48 0.09 0.15 0.04 0.06	2.11 2.48 0.33 2.25 0.54 0.87	3.17 2.11 1.61 0.61 1.13	0.0 0.47 2.95 0.16 1.18	0.0 0.30 4.44 1.71 0.54 2.91	0.55 0.26 0.0 0.05 0.0 0.80	0.04 0.21 0.27 0.0 0.04	8.64 8.24 10.18 8.71 4.35 9.32

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Table D-8. Monthly consumptive-use factors for Roswell, New Mexico, 1905-1968.

Yr	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
19 5 19 6	2.75	3.00	4.40 4.18	5.09	6.49	7.44	7.37 7.39	7.31 7.04	5.95 5.90 5.87	4.46	3.41 2.98	3.12	58•92 59•87
19 7 19 8	2.90	3.43	4.85	5.12	6.07	7.11	7.66	7.35	5.87 5.64	4.66	3.13 3.18	2.96	61.11 60.14
19 9 1910	3.16	3.11	3.93	5.17	6.32	7.34	7.77	7.18	5.70	4.54	3.66	2.38	60.25
1911	3.40	2.95	4.56	5.31	6.52	7.34	7.62	7.23	6.14	4.52	3.07	2.45	61.11 58.67
1913	2.48	2.81	3.97		6.63	6.99	7.71	7.31	5.48	4.50	3.58	2.57	59.11 60.52
1915	3.35	2.96 3.03	4.08	5.09	6.46	7.24	7.63		5.80	4.74	3.53	2.93	59.23
1916 1917	2.99 2.78	3.04	4.71	5.03	6.59	7.23	7.63	6.95 7.22	5.89		3.62	2.93	60.93 60.26
1918	2.46	3.20 3.04	4.53	5.20	6.55	7.44	7.77	7.22	5.64 5.84	4.80	3.27	2.78	59.87 59.40
1920 1921	2.66 3.08	3.29 3.11	4.09		6.48	6.96	7.66 7.50	6.95 7.31	5.96	4.69 4.95	3.22 3.63	2.81 3.20	59.65 62.09
1922	2.64	2.99	4.09 3.90	5.12	6.65	7.23 7.48	7.82	7.50	6.06 5.86	4.71 4.47	3.31	3.09 2.29	61.16 60.30
1924	2.61	2.86 3.09 3.39	3.89 4.53	4.99 5.60	6.28	7.68	7.72 7.65 7.82	7.44	5.82 5.85	4.74	3.59	2.49	60-28 61-41
1926	2.51 3.10	3.36	3.86 4.23	4.88	6.34	7.28	7.51	7.27	5.99	4.83	3.42	2.72	59.97 62.67
1928	3.04	2.95	4.48 4.13	5.00	6.53	7.46	7.75	6.99 7.25	5.72	4.93	3.25	2.80	60.90 59.81
1930	2.36	3.45	3.93	5.58	6.38	7.40	7.81	7.43	6.11	4.77 5.00	3.32	2.65	61.20 60.93
1932	2.85 2.75 2.93	3.57	3.94 3.81 4.40	5.28	6.46	7.34	7.85 7.92	7.29	5.62	4.46	3.31	2.40	60.15 62.14
1934	2.93	3.22	4.29 4.58	5.47 5.37	6.90	7.70	8.12	7.57	5.99	5.21	3.67	3.03	64.10 61.54
1936	3.20	3.20	4.48	5 - 28	6.60	7.53	7.71	7.35	5.81	4.56	3.22	2.95	61.48
1937	2.61		4.53	5.21	6.61	7.44	7.61	7.40	5.97	4.91	3.37 3.17 3.24	2.79	62.00
1939 1940	3.04	3 . L !	4.38	5.16	6.73	7.15	7.76	7.21	0.06	4.90	3.21	3.4/11	62.08 61.49
1941 1942	2.99		3.95 4.07	5.14	6.47	7.44	7.55	7.23	5.81	4.80	3.72	3.14	60.60 61.23
1943 1944	3.00	3.25	4.23	5.04	6.60	7.53	7.75 7.83 7.69	7.72	5.74	4.71 4.82	3.34 3.42	2.59	62.38 61.27
1945 1946	3.05 2.73	3.11	4.40 4.50	5.69	6.87	1.53	7.84	7.46	6.04	4.72 4.94	3.31	2.85 3.11	62.29 62.86
1947 1948	2.75	2.91	4.18 3.81	5.10	6.69	7.45 7.52	7.86	7.33 7.38	6.02 5.90	5.02 4.65	3.08 3.13	2.68 3.05	61.09 60.73
1949	2.03	2.99	4.43	5.06 5.27	6.73	7.40	7.84 7.66	7.22	5.91 5.90	4.53 5.17	3.70 3.44	2.71 3.02	60.55 62.95
1951	2.73 3.24	7.99	4.02	5.09	6.70 6.58	7.51		7.57	6.05 5.89	4.63		2.91	62.02 61.83
1953	3.37	2.96	4.60	5.23	6.43	7.87	8.06	7.45	6.00	4.73	3.43	2.49	62.63 63.71
1955	3.01 2.76 2.99	2.84	4.26 4.37	5.35	6.63		7.69	7.41	6.09	4.83	3.39	2.95	61.52 62.14
1957	2.96 2.81	3.62	4.27	5.00 5.11	6.43 6. <u>7</u> 9	7.53	8.12 8.17	7.48	5.87	4.59	3.15	3.04	62.06
1959 1960	2.79	3.02	4.13	5.25	6.76	7.61	7.80	7.61 7.41	6.10	4.66		2.95	61.84 60.76
1961	2.66	2.93	4.14	5.09	6.69	7.38 7.28	7.68	7.23	5.80	4.67		2.78	59.69 60.68
1963	2.40	3.38	4.17	5.38	6.72	7.41	8.05	7.39	6.04	4.99 4.80	3.37 3.38	2.44	61.22
1965	2.46 3.15	2.52	3.76	5.44	6.67	7.44	7.95 8.29	7.23	5.90	4.66	3.70	2.92	61.57 60.75
1967	2.19 2.93 2.78	2.66 3.07	4.28 4.75 3.72	5.24 5.74 4.55	6.76 6.62 6.00	7.27	7.86	7.06	5.74 5.40	4.68	3.34	2.45	61.53 56.42
1968	2.18	3.00	3.12	サップフ	0.00	0.77	1 + 1 4	0.01	J 6 T U	7.JO	2.70	2 . 03	JU+72

Table D-9. Monthly consumptive-use factors for Artesia, New Mexico, 1905-1968.

Yr 56789011234	222223333222	3 · 09 · 42 · 12 · 12	Mar 244745256511	22459892 2555555555555555555555555555555555	Moses 44 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4	7.12 7.28 7.36 7.66 7.47 7.21 7.16	Jul 7.4189777.68.17777.8.87	Aug 7.25 7.03 7.29 7.12 7.51 7.14 7.44 7.49	Sept 0920523275	Oct 480 4.5690 4.55752327 4.7787	Nov 35511989852	Dec 2.38 3.27 2.99 2.40 3.67 2.67 2.69 2.69 2.69	Annual 58.89 60.76 61.10 60.14 60.23 64.83 63.66 63.66
19118901233456 1991991223456 199919992223456	4720558162556 4720558162556 923322232323222	333337 33333333	34444444444444444444444444444444444444	55555555555555555555555555555555555555	66666666666666666666666666666666666666	7.57416239 7.77777777777777777777777777777777777	7.99 9.99 1.99 1.99 1.99 1.99 1.99 1.99	7.24657.0921.0777777777777777777777777777777777777	979843682670 9798990198789	9873238231249 9880087979 98804444444444444444444444444444444444	33333333333333333333333333333333333333	190609682343 190608315767 190608315767	63333227 663333227 664666666666666666666666666666666666
19239 19239 19333 19333 19333 19338 19338 19338	932063121034 ••9489781748 ••489781748	4945 4954	49075592149 44444444434	3678881676883 55555555555555555555555555555555555	9729296 •9754496 •13801 •666666766666	77777777777777777777777777777777777777	7.84 6.04 7.99 7.99 7.77 7.87 7.77 7.67	7.08 7.45 7.53 7.47 7.14 7.43 7.30	7032399533956 767017795797	8596224807583 4544545544444	8385130064408 8304537644408 8333333333333333333333333333333333	2222222222579 599772298887 2222222222222222222222222222	62608 6126187 612618925 612614935 611925 611925
1939 19412 19442 19445 19447 19449 19449 1950	8487843307087 8487868765287 8487868765287	70606100790841 73612822129125 73233333333333333333	4.4.07556990649 4.4.3.4.4.4.4.4.4.4.4.4.4.4.4.4.4.4.4.4	264824263476 442361183926 555555555555555	512544534722 785656969998	7.68440 3.5591 6.677 7.777 7.777 7.777 7.777 7.777	7.8389 7.0561 7.77777777777777777777777777777777777	7.1121849777777777777777777777777777777777777	55575755555555555555555555555555555555	444444455445 444444455445	313761749862	019955808333 019955808391	61
19555555555555555555555555555555555555	233322332222222	03028012789970 03050971099970 030333333333333333333333333333333	7500852991257 327355483347 44444434444	3458633081786 •••••355555555555555555555555555555555	666666676666667 • • • • • • • • • • • • • • • • • • •	7.68740789007977777777777777777777777777777	87.8.1443544498099 87.8.144354498099	777777777777777777777777777777777777777	6.09 5.85 6.29 6.04 5.91	545555544445555	14279946872702 1427761253627 1427761253627	18359901856714 1904190414 10181904 1018	63.899 64.446 653.4.474 653.3.13.13.13.13.13.14.1663.13.14.166
1963 1964 1965 1966 1967 1968	2.50 2.76 3.18 2.49 2.86 3.00	3 · 28 2 · 85 3 · 01 2 · 83 3 · 08 3 · 38	4.50 4.34 3.94 4.47 4.77 4.30	5.87 5.50 5.75 5.46 5.78 5.15	7.02 7.06 6.87 6.74 6.53 6.57	7.63 7.55 7.61 7.41	8.31 8.26 8.17 8.31 7.97 7.49	7.55 7.66 7.35 7.19 7.18 7.01	6.12 5.99 5.93 5.75 5.37	5.32 5.06 4.92 4.75 4.96 4.78	3.72 3.67 3.97	2.75 3.05 3.11 2.77 2.78 2.73	64.55 63.75 63.82 61.93 62.76 60.61

Table

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Table D-18. Daytime hours and monthly percentages at Artesia, New Mexico.

Dec.	10.08 10.07 10.07 10.06	10.04 10.03 10.02 10.02	10.01 10.01 10.00 10.00	00000 00000 00000	00000 00000 000000
Nov.	10.51 10.49 10.47 10.46	10.43 10.41 10.39 10.38	10.34 10.32 10.31 10.30 10.29	10.27 10.26 10.24 10.23	10.20 10.18 10.17 10.16 10.15
Oct.	11.51 11.49 11.47 11.45	11.41 11.39 11.37 11.35	11.31	11.21 11.19 11.18 11.16	11.11 11.09 11.08 11.06 11.04
Sept.	12.51 12.49 12.47 12.45	12.41 12.39 12.36 12.36	12.31 12.30 12.27 12.25 12.24	12.21 12.19 12.18 12.15	12.12 12.09 12.07 12.06 12.03 12.03
Aug.	13.46 13.44 13.43 13.42	1113 133 133 133 133 133 133 133 133 13	113.23 113.23 13.23	13.22 13.20 13.17 13.16	13.12 13.00 13.00 13.06
July	14.18 14.18 14.16 14.16	14.15 14.15 14.14 14.13	14.11 14.11 14.09 14.08	14.06 14.06 14.04 14.04	14.01 14.00 13.59 13.57
June	14.11 14.12 14.13 14.13	14.15 14.16 14.16 14.17 14.17	14.18 14.18 14.18 14.19	14.19 14.20 14.20 14.20	14.19 14.20 14.20 14.20 14.20
 May	13.29 13.32 13.32 13.32	13.36 13.42 13.42 13.42	13.45 13.44 13.50	11133.53 133.55 135.55 8	13.59 14.01 14.02 14.03
Apr.	12.32 12.34 12.37 12.38	12.42	12.51 12.54 12.56 12.57 12.59	13.02 13.04 13.05 13.07	13.13
Mar.	11.28 11.31 11.34 11.36	11.39 11.42 11.44 11.47	11.49 11.51 11.53 11.55	11.59 11.01 12.03 12.05	12.09 12.11 12.14 12.15
Feb.	10.38 10.38 10.40 10.42	10.45 10.47 10.49 10.51	10.53 10.55 10.57 11.01	11.03 11.05 11.07 11.09	11.12 11.14 11.17 11.19
Jan.	10.01 10.02 10.03 10.03	10.04 10.05 10.06 10.07	10.08 10.09 10.10 10.12	10.14 10.15 10.17 10.18	10.21 10.22 10.23 10.25
Day	HUMTU	984	125 125 125 125 125 125 125 125 125 125	118 118 20	222222222222

9.59 9.59 10.00 6.99 10.00 310.30 7.08 10.14 10.13 10.12 10.10 314.16 7.09 New Mexico (continued) ŧ 11.02 11.00 10.58 10.57 10.53 7.95 .93 352.05 12.01 11.59 11.59 11.57 360.11 .11  $^{\circ}$ ı 13.03 12.53 12.57 12.557 •34 .32 Artesia, 413.4 S 12, 436.44 133.55 133.55 133.55 400 .86 ·84  $\infty$ ⇉ a t m percentages .68 99. 428.5 1 g 9 .10 .70 .68 429.4 monthly 9 8.90 .92 395.1 t and  $\infty$ 4444 Daytime hours 12.20 12.22 12.23 12.26 2.29 32 34 369.4 φ. 8 11.22 11.24 11.26 11.28 .20 96.9 308.17 319.5 D-18. 10.27 10.28 10.30 10.31 .95  $\mathfrak{C}$ 35 9 days307.5 10. 9 Table days days Percentage Feb. 28 days Total 284 30 30 30 29 29 Feb Feb Feb

Table D-19. Daytime hours and monthly percentages at Roswell, New Mexico. 1

Dec.	10.06 10.05 10.05 10.04	10.02 10.01 10.01 10.01 10.00	0 $0$ $0$ $0$ $0$ $0$ $0$ $0$ $0$ $0$	00000 77777 77777	99.50
Nov.	10.51 10.49 10.47 10.45	10.41 10.39 10.38 10.36	10.33 10.32 10.30 10.29	10.24 10.24 10.22 10.21	10.19 10.17 10.15 10.14
Oct.	11.51 11.49 11.46 11.45	11.40 11.39 11.37 11.35	11.31 11.29 11.26 11.24 11.24	11.21 11.19 11.16 11.14	11.11 11.09 11.07 11.05 11.04
Sept.	12.52 12.50 12.48 12.46	12.42 12.40 12.38 12.36	12.31 12.30 12.28 12.25	17.22 12.19 12.18 12.15	12.12 12.09 12.07 12.08 12.03 (contil
Aug.	13.48 13.48 13.44 13.44	1133.4	11113 13.00 10.00	13.22 13.20 13.19	1133.08 133.09 13.08
July	14.20 14.20 14.19 14.19	14.14 14.16 14.16 14.16	14.13 14.13 14.11 14.11	14.08 14.08 14.06 14.05	14.03 14.01 14.00 13.59
June	14.14 14.14 14.15 14.15	14.17 14.18 14.18 14.19	14.20 14.20 14.20 14.21 14.21	14.22 14.22 14.22 14.22	14.22 14.22 14.22 14.22
May	13.32 13.32 13.32 13.36	13.44 13.44 13.44 13.44 13.44	133.47 133.57 13.57 13.57 13.57 13.57	13.55 13.55 13.55 14.00	14.01 14.02 14.04 14.04 14.06
Apr.	12.32 12.34 12.36 12.39	12.42 12.45 12.46 12.48	12.53 12.54 12.554 13.58	13.02 13.04 13.06 13.10	13.12 13.14 13.16 13.17
Mar.	11.28	11.38	11.49	12.01 12.01 12.03 12.05	12.09 12.11 12.14 12.15
Feb.	10.35 10.38 10.40 10.42	10.43 10.45 10.47 10.49	10.53 10.55 10.57 11.01	11.03 11.04 11.06 11.08	11.12 11.14 11.16 11.18
Jan.	9.59	10.02 10.03 10.04 10.05	10.07 10.07 10.08 10.10	10.12 10.13 10.15 10.16	10.19 10.20 10.21 10.23
Day	HUW71	10	H H H H H H	116 118 119	25 24 25 25

Table D-19. Daytime hours and monthly percentages at Roswell, New Mexicol (continued).

របស់របស់ស	9.58	309.3	6.95
10.12 10.11 10.00 10.09	I	313.30 3	7.04
11.02 11.00 10.57 10.55	10.52	351.48	7.90
12.01 11.59 11.57 11.55	l	371.8	8.34
13.04 13.00 12.58 12.58	12.54	419.8	9.42
13.55	13.49	437.4	9.83
14.22 14.22 14.21 14.21 14.21	i	429.5	9.66
14.07 14.08 14.09 14.11	14.21	430.1	9.66
13.21 13.24 13.24 13.25	l	390.3	8.77
12.20 12.22 12.23 12.26	12.30	371.20	8.30
11.22 11.24 11.26 11.27	ı	307.5	6.92
10.26 10.27 10.29 10.31	10.34	317.1	7.12
26 28 30 30	31	Total Feb. 28 days317.1 Feb. 29 days	ercentage eb. 28 days eb. 29 days
		Ĕ Ĕ	ជ័ ឝ ឝ

 $^{
m l}$  Computed from the sunrise and sunset tables supplied by the state climatologist.

Table D-20. Monthly percent of daytime hours at Roswell and Artesia, New Mexico. 1

	Roswe	ell	Arte	esia
	February 28 days	February 29 days	February 28 days	February 29 days
January	7.12	7.10	6.95	6.93
February	6.92	7.15	6.96	7.20
March	8.30	8.32	8.34	8.32
April	8.77	8.75	8.92	8.90
May	9.66	9.64	9.70	9.68
June	9.66	9.63	9.68	9.66
July	9.83	9.81	9.86	9.84
August	9.42	9.39	9.34	9.32
September	8.34	8.32	8.13	8.11
October	7.90	7.88	7.95	7.93
November	7.04	7.02	7.09	7.08
December	6.95	6.93	7.01	6.99

 $<sup>^{\</sup>mbox{\scriptsize l}}\mbox{\scriptsize Calculated}$  from the sunrise and sunset tables supplied by the state climatologist.

Table D-21. Monthly consumptive use coefficients for irrigated crops in the Roswell and Artesia areas, New Mexico.

		Ros	well		Artesia				
Month	Alfalfa	Cotton	Sorghum	Small Grains	Alfalfa	Cotton	Sorghum	Small Grains	
Jan.									
Feb.									
Mar.	0.31			0.35	0.39	0.06		0.35	
April	0.67	0.35		0.38	0.69	0.35		0.40	
May	0.80	0.40		0.50	0.80	0.40	0.19	0.50	
June	0.90	0.60	0.50	0.45	0.90	0.60	0.50	0.73	
July	1.00	0.90	1.00		1.00	0.90	1.00	•	
Aug.	1.00	1.00	0.85		0.90	1.00	0.85		
Sept.	0.80	0.95	0.19		0.80	0.95	0.19		
Oct.	0.65	0.58			0.70	0.75			
Nov.	0.09				0.18	0.05			
Dec.									

<sup>&</sup>lt;sup>1</sup>Adjusted for the growing seasons before and after the frost-free periods. Compiled from Blaney and Hanson [1965] and from Henderson and Sorensen [1968].

Table D-22. Yearly consumptive irrigation requirement (in acre-feet) for crops in Chaves County, New Mexico, 1923-1968.

Year	Alfalfa	Cotton	Sorghum	Small Grains	Total
1923	30,700	11,900	200	-4,300	38,500
1924	42,600	37,100	1,600	2,100	83,300
1925	35,400	34,100	900	200	70,600
1926	32,500	26,200	500	-2,500	56,800
1927	46,700	38,100	1,600	3,500	90,000
1928	30,000	30,700	700	-1,500	60,000
1929	31,700	36,600	1,300	-500	69,200
1937	35,600	63,900	7,700	-1,600	105,600
1938	59,500	42,200	20,800	1,000	123,400
1939	46,600	40,000	12,100	-600	98,100
1940	54,700	38,500	11,000	-1,400	102,800
1941	28,800	12,800	-14,800	-9,300	17,500
1942	50,100	39,300	10,200	-2,300	97,300
1943	70,500	53,200	26,900	1,400	152,000
1944	73,000	49,100	17,200	-200	139,100
1945	84,000	69,000	21,500	2,500	177,100
1946	75,000	59,000	15,800	-500	149,400
1947	66,300	93,200	17,100	1,500	178,000
1948	62,600	100,000	13,400	400	176,500
1949	52,800	82,300	10,200	-700	144,600
1950	77,600	54,000	8,700	-3,600	136,700
1951	91,600	125,500	16,200	1,200	234,500
1952	94,600	102,400	14,100	1,000	212,100
1953	96,600	120,900	11,700	1,300	230,600
1954	94,800	77,100	22,200	1,700	195,800
1955	93,800	63,900	22,400	2,800	182,900
1956	107,300	75,700	31,400	8,300	222,700
1957	91,700	60,900	23,900	1,800	178,300
1958	84,200	55,800	17,100	-2,800	154,200
1959	73,000	63,100	4,900	1,100	142,100
1960	71,600	66,300	3,400	-2,900	138,300
1961	90,800	69,300	8,500	4,100	172,700
1962	76,800	60,800	6,300	-900	143,000
1963	96,300	68,900	12,700	6,700	184,700
1964	101,000	66,700	11,600	4,800	184,100
1965	115,200	65,000	8,400	2,800	191,400
1966	108,900	47,100	8,600	1,000	165,500
1967	97,000	39,900	6,700	500	144,100
1968	79,500	34,100	2,600	-1,400	114,800

<sup>1</sup> Small grains and miscellaneous crops.

Table D-23. Yearly consumptive irrigation requirement (in acre-feet) for crops in Eddy County, New Mexico.

Year	Alfalfa	Cotton	Sorghum	Small Grains	Total
1959	71,400	69,800	3,400	4,300	148,900
1960	63,500	60,600	2,200	1,200	127,500
1961	79,900	68,500	2,700	4,200	155,300
1962	71,100	59,400	1,900	1,500	134,000
1963	88,000	65,200	2,800	4,800	160,800
1964	81,900	59,800	1,300	1,200	144,200
1965	91,300	56,600	1,900	1,400	151,100
1966	85,600	42,700	1,700	1,300	131,200
1967	94,500	44,800	0	600	140,000
1968	77,700	36,600	700	0	115,000

<sup>&</sup>lt;sup>1</sup>Small grains and miscellaneous crops.

Table D-24. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the principal confined aquifer, Roswell Basin, New Mexico.

	<del> </del>			
Well	Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
085	24E 05 34	3 3.788	3.351	37,150
	24E 28 22		2.080	22,900
	24E 28 12		0.927	15,380
	24E 33 41		2.330	67,650
		4.674*	1.970*	30,673*
098	24E 34	8.368	6.705	15,730
		8.368*	6.705*	15,730*
109	3 23E 27 22	2 2.975	0.240	50,030
	23E 24 14		0.491	53,400
	23E 34 43		2.327	14,430
		4.280*	0.650*	33,781*
109	3 24E 15 13	0.423	1.129	387,070
	3 24E 15 32		33,239	13,910
	3 24E 15 33		5.268	86,400
	3 24E 15 34		53.122	16,160
	3 24E 17 14		5.834	27,040
	3 24E 17 32		5.554	33,180
	3 24E 20 23		0.964	88,990
	3 24E 21 42		0.733	165,890
	3 24E 22 33		0.545	40,180
	3 24E 22 34		2.000	101,950
	S 24E 27 42		25.016	11,410
		3.454*	4.166*	49,782*
119	5 23E 03 10	00 8.110	20.699	16,850
	5 23E 12 44		4.619	198,720
113	5 23E 12 44	44 3.304	5.246	43,720
113	S 23E 13 2	32 3.774	2.781	38,790
119	3 23E 28 22	23 1.450	0.749	97,200
		2.592*	4.016*	56,024*
	S 24E <b>01</b> 3:		3.519	13,310
	S 24E 04 1		0.422	224,640

(continued)

Table D-24. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the principal confined aquifer, Roswell Basin, New Mexico (continued)

Well Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)		Transmissivity T (1,000 ft <sup>2</sup> /day)
11S 24E 06 310	9.123	4.152	15,120
11S 24E 08 124	0.399	1.002	410,400
11S 24E 08 124	8.781	2.187	15,730
11S 24E 11 243	2.233	1.603	68,260
11S 24E 12 113	0.251	0.874	669,600
11S 24E 12 231	2.324	7.612	64,890
11S 24E 13 233	7.525	1.687	18,580
11S 24E 14 213	4.055	1.356	35,860
11S 24E 14 324	0.261	1.137	639,630
11S 24E 14 343	4.959	1.120	27,300
11S 24E 15 431	13.363	1.064	10,540
11S 24E 18 242	1.271	0.340	98,500
11S 24E 18 333	7.091	29.462	19,530
11S 24E 19	6.740	3.045	20,910
11S 24E 20 313	1.517	2.249	101,090
11S 24E 26 433	2.682	0.511	55,300
11S 24E 28	6.418	21.588	21,600
11S 24E 28 313	15.308	12.241	7,600
11S 24E 36 211	8.134	21.551	16,680
	3.151*	2.391*	45,485*
11S 25E 29 333	8.063	4.673	17,110
11S 25E 32 133	2.208	3.007	65,840
	4.219*	3.749*	33,564*
12S 23E 01 413	2.111	1.299	71,110
12S 23E 06 214	6.735	0.210	21,080
	3.771*	0.522*	38,717*
12S 24E 21 333	3.885	4.182	37,410
	3.885*	4.182*	37,410*
12S 25E 13 111	0.804	0.629	204,770
128 25E 35 131	0.370	1.144	444,960
	0.545*	0.848*	301,852*
13S 25E 13 133	8.454	26.302	16,240
13S 25E 24 333		3.128	232,420
13S 25E 26 411	2.467	0.092	60,740
	2.424*	1.963*	61,204*
			(continued)

Table D-24. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the principal confined aquifer, Roswell Basin, New Mexico.(continued)

Well Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	efficient C	Transmissivity (1,000 ft <sup>2</sup> /day)	
13S 26E 06 331	9.369	19.297	14,600	
13S 26E 30 213		1.769	59,530	
13S 26E 31 211	5 <b>.7</b> 45	3.771	24,620	
13S 26E 31 214	0.422	1.329	387,070	
	2.751*	3.617*	53,646*	
14S 26E 09 313	5.896	0.434	24,360	
14S 26E 32 124		1.303	40,780	
	4.583*	0.752*	31,518*	
15S 25E 35 213	6.384	1.410	22,210	
15S 25E 35 311	1 <b>4.2</b> 46	0.186	9,070	
	9.537*	0.512*	14,193*	
15S 26E 04 123	2.706	1.107	55,120	
15S 26E 13 222	0.241	9.205	691,200	
	0.808*	3.192*	195,189*	
16S 24E 02 324	1.585	0.213	95,900	
	1.585*	0.213*	95,900*	
16S 25E 07 111	2.365	0.355	62,900	
	2.365*	0.355*	62,900*	
16S 26E 20 433	7.271	1.269	18,920	
16S 26E 20 433	9.132	5.552	15,030	
	8.149*	2.654*	16,863*	
17S 26E 08 431	11.286	0.756	12,530	
17S 26E 08 431		2.055	44,240	
17S 26E 08 444		2.797	14,430	
17S 26E 09 113		2.451	22,290	
17S 26E 17 233		0.601	9,940	
17S 26E 17 233		9.401	10,710	
17S 26E 20 431	5.225	1.961	28,340	
			(continued)	

Table D-24. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the principal confined aquifer, Roswell Basin, New Mexico (continued)

Well Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
17S 26E 32 133	4.858	5.334	29,380
17S 26E 32 213	5.070	1.007	28,600
	7.182*	2.048*	19,725*
18S 26E 10 313	2.090	0.761	72,580
18S 26E 34	7.920	3.458	17,630
18S 26E 34	6.622	4.844	21,170
	4.786*	2.336*	30,033*
19S 26E 27 221	29.173	6.983	2,680
	29.173*	6.983*	2,680*

<sup>\*</sup>Logarithmic average by township.

Table D-25. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the unconfined aquifer, Roswell Basin, New Mexico.

Well	Loca	ti	on	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
109	3 24E	36	413	22.550	4.730	2,770
				22,550*	4.730*	2,770*
119	S 24E	02	221	30.618	1.581	1,730
	5 24E			5.509	0.684	12,610
	S 24E				3.792	15,030
	S 24E			16.324	1.288	3,890
				10.671*	1.516*	5,976*
11	S 25E	06	332	20.370	2.007	2,850
	S 25E				1.684	1,900
	S 25E			27.308	0.968	2,070
	S 25E				37.210	7,950
	S 25E S 25E				7.366	1,810
				21.353*	3.895*	2,764*
12	S 25E	25	431	24.941	5.822	2,420
	S 25E				2.714	4,060
				20.117*	3.975*	3,134*
12	S 26E	32	133	11.689	1.507	5,700
				11.689*	1.507*	5,700*
13	3S 25E	35	133	7.739	2.576	8,810
				7.739*	2.576*	8,810*
7:7	3S 261	22	2 313	12.915	1.995	5,270
	3S 26				12.194	4,580
	3S 261				3.117	3,630
	3s 261				2.999	2,330
				17.080*	3.883*	3,780*

Table D-25. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the unconfined aquifer, Roswell Basin, New Mexico (continued)

Well	Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
	25E 13 311	5.714	0.345	12,180
		5.714*	0.345*	12,180*
3.40	000 02 422	3.178	6.828	23,140
	26E 03 433		1.104	9,420
	26E 06 142	7.360 1.987	1.914	39,140
	26E 06 211	31.131	38.920	1,900
	26E 08 342	10.331	0.953	6,570
	26E 08 433	10.896	2.011	6,220
	26E 09 221	0.504	0.868	181,440
	26E 10 133	19.419	21.342	3,110
	26E 14 113	4.680	0.644	15,210
	26E 17 233	0.323	11.745	286,850
	26E 17 444 26E 18 211	3.927	1.992	18,490
	3 26E 18 324	10.664	2.264	6,390
	3 26E 10 324	2.806	1.248	26,780
	3 26E 23 230	12.393	4.284	5,360
		4.806*	2.942*	14,955*
1.58	S 26E 10 112	3.398	23.842	21.770
	3 26E 20	8.140	3.695	8,290
	3 26E 29 321	0.486	2.735	184,030
	3 26E 29 344	6.940	10.361	9,760
	3 26E 32 344		9.491	10,970
		3.583*	7.498*	20,426*
169	S 25E 06 223	4.648	0.387	15,380
	S 25E 25 211		0.682	2,510
		10.672*	0.514*	6,213*
16	S 26E 19 411	12.986	0.884	5,010
	S 26E 29 331		5.680	4,750
	S 26E 32 213		0.143	3,460
	S 26E 32 311		4.783	3,020
		15.959*	1.361*	3,971*
17	S 26E 08	6.521	4.740	10,540
	S 26E 17 333		0.872	11,660
	S 26E 21	12.393	15.057	5,440
	S 26E 35 13		11.544	6,910
		7.852*	5.177*	14,661*

Table D-25. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the unconfined aquifer, Roswell Basin, New Mexico. (continued)

Well	Locatio		Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
188	26E 17 3	10.189	15.219	6,740
188	26E 18 2	21 3.558	1.231	20,740
		6.021*	4.328*	11,823*
208	26E 07 4	23 1.186	4.691	69,210
208	26E 08 1	16.726	8.696	3,540
		4.454*	6.387*	15,653*

<sup>\*</sup> Logarithmic average by township.

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Table D-26. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the shallow confined aquifer, Roswell Basin, New Mexico.

Well Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	efficient C	Transmissivity T (1,000 ft <sup>2</sup> /day)  3,970		
08S 24E 35 224	33.220	3.420			
08S 24E 35 343	2.472	0.955	60,310		
	9.062*	1.807*	15,470*		
09S 24E 02 414	2.397	1.177	62,900		
09S 24E 02 421	8.710	6.537	15,810		
09S 24E 11 133	2.961	2.748	49,940		
	3.954*	2.765*	36,760*		
10S 25E 31 413	13.416	0.514	10,200		
	13.416*	0.514*	10,200*		
11S 24E 01 334	0.581	1.740	281,660		
11S 24E 06 310	9.123	4.152	15,030		
11S 24E 06 423	0.846	0.673	184,030		
11S 24E 18 333	7.091	29.462	19,870		
11S 24E 18 444	3.452	7.102	42,420		
	2.559*	3.995*	58,000*		
11S 25E 08 123	17.080	3.272	7,950		
11S 25E 28 243	6.415	5.497	21,510		
11S 25E 28 234	15.543	12.852	8,810		
	11.941*	6.137*	11,460*		
12S 25E 05 111	0.533	14.041	301,540		
12S 25E 36 111	33.470	1.861	3,970		
	4.223*	5.112*	34,500*		
13S 24E 25 212	42.036	6.741	3,110		
	42.036*	6.741*	3,110*		

Table D-26. Well-loss coefficients, formation-loss coefficients, and transmissivities from routine step-drawdown tests in the shallow confined aquifer, Roswell Basin, New Mexico (continued)

Well Location	Formation-Loss Coefficient B ft/(ft <sup>3</sup> /sec)	Well-Loss Co- efficient C ft/(ft <sup>3</sup> /sec) <sup>2</sup>	Transmissivity T (1,000 ft <sup>2</sup> /day)
13S 25E 12 13S 25E 35 232 13S 25E 27 211	8.386 9.262 6.179	0.316 0.456 0.075	16,500 14,860 22,810
	7.829*	.221*	17,750*
14S 24E 18 222	7.517	0.597	18,580
	7.517*	0.597*	18,580*
14S 25E 12 331	11.199	4.573	12,440
	11.199*	4.573*	12,440*

<sup>\*</sup>Logarithmic average by townships.

0.51.02	10,8286								-	222	10.8281 10.0174 10.7354 10.7374 10.6822 10.5621 10.5651 10.5651
0.16-02	14.0474								14.0474 14.0471 14.0471 14.0462 14.0410 14.0410 14.0100	15.000 11.7500 11.7500 11.6151 11.6151 11.2675 11.1572 11.177	13.0031 12.4239 11.79880 11.77988 11.4083 11.2870 11.12810 11.12810
0.96-03	14,2581								14,2591 14,2591 14,2595 14,2595 16,2495 16,2495 16,2495	14.0432 13.8660 13.7058 13.5640 13.2553 13.2253 13.1307	11,0456 12,4464 12,0733 11,0020 11,8080 11,2637 11,0167
0.8E-03	14.4916								14.4936 14.4929 14.4899 14.4835 14.4605 14.4447	14.4073 11.9724 11.9724 11.7913 13.6353 13.3786 13.2710	13,0845 12,4667 12,0870 11,8924 11,5921 11,2697 11,1389 11,0234
0.7E-03	14.7607	leaky							14,7607 14,7604 14,7604 14,7873 14,7887 14,7887 14,7887 14,6912 14,6652	14.6084 14.3176 11.6721 13.7007 13.7007 13.4273 13.4273 13.3140	13,1194 12,4848 11,0997 11,02065 11,1436 11,1436 11,1436
0.68-03	15.0690	for	W(u,p)						15.0690 15.0669 15.0560 15.0342 15.0043 14.9690 14.8496	14.8088 14.4417 14.1633 13.9416 13.592 13.6045 13.404 13.404	13.1503 12.5006 12.1099 11.8296 11.6109 11.2796 11.1476 11.0311
0.55-03	15.4136	tion		ρ̈́ρ				15,4336	15,4334 15,4225 15,3901 15,2872 15,2872 15,2290 15,1702 15,1702	15,0014 14,5580 14,2443 14,0042 13,8102 13,5076 13,1848	13.1767 12.5141 12.1189 11.8364 11.6164 11.2835 11.1511 11.0341
0.4E-03	15.8799	function	FUNCTION	**************************************				15,8799 15,8798 15,8798 15,8794 15,8777	15,8762 15,8310 15,8310 15,7513 15,6606 15,9884 15,3984 15,3984	15,1776 14,6571 14,1132 11,8530 11,5386 13,4120 13,2997	13,1987 12,5252 12,1264 11,8420 11,6209 11,1361 11,1539 11,0366
0,3E-03	16.4553	well	FC					16,4553 16,4552 16,4547 16,4547 16,4512 16,4451 16,4451	16.4205 16.2767 16.1149 15.9648 15.900 15.7091 15.6002 15.5014	15,3279 14,7081 14,7081 14,0866 13,8868 111,5811 11,5811 11,4115	13,2159 12,5139 12,132 13,8464 11,2844 11,2852 11,0386
0.2E-03	17.2662	e of	ES OF	•				17.2662 17.2650 17.2576 17.2413 17.1879 17.1879 17.1198 17.1198	17.0460 16.7064 16.4373 16.2219 16.0416 15.0416 15.1596 15.6428 15.6428	15.4431 14.7981 14.7981 14.1297 13.9115 13.7125 13.4807 11.13.4490	13,2283 12,5401 12,1363 11,8495 11,4449 11,1376 11,0400
0.15-03	18,6525	tabl	VALUE				19,6525 18,6524 18,6522 18,6523 18,6464 19,6463 19,6413	18.6275 18.5060 18.2509 18.2202 19.0927 17.9772 17.8725 17.8725	17,6082 17,0291 16,6612 16,1956 16,1966 16,0104 15,8621 15,7100	15,5160 14,8154 14,8154 13,9264 13,9264 13,5914 13,5914 13,5914	13.7358 12.5439 12.1388 11.8514 11.4451 11.7850 11.1585 11.0408
0.98-04	18.8632	ıtary		<u> </u>			18,8632 18,8631 18,8631 18,6596 18,6596 18,6393 18,6393	18,6481 18,4711 18,1109 18,1109 19,0433 17,9304 17,5286	17.6507 17.0515 16.6784 16.4071 16.1939 16.0182 15.8688 15.7868 15.7868	15.5207 14.8326 14.4355 10.9274 13.7457 13.5921 13.4588	13,2161 12.5441 12.1390 11.8515 11.6284 11.462 11.2921 11.0400
0.20.0	19.0987	Supolementary anulfers					19.0987 19.0980 19.0950 19.0866 19.0787 19.0656	19.0124 18.7581 18.5775 18.3964 16.2404 18.1043 17.9839	17.6896 17.0719 16.6922 16.4922 16.2021 16.0252 15.9748 15.97441	15,5249 14,2269 14,2269 11,9286 11,5388 11,5382 11,5382 11,5383 11,5383	13.2367 12.5443 12.1391 11.6285 11.4463 11,1586 11,0409
10.12.0	19,3658	Supt					19,3658 19,3658 19,3537 19,3537 19,3392 19,3496 19,2703	19.2135 16.9227 18.6772 18.4754 18.3058 16.1600 13.0124 17.9191		15.5286 14.8416 14.4382 14.1515 13.9290 13.5997 13.5997 13.4599	
90.0	19.6741	-27.					19.6741 19.6720 19.6720 19.6391 19.6094 19.5094 19.5741 19.4955	19.4139 19.0468 18.7684 19.5467 18.2097 18.2097 17.9571	17.7554 17.1058 16.4346 16.2161 16.0367 15.8847 15.7528	15,5319 14,652 14,439 14,152 14,152 13,929 13,7476 13,7476 13,7476 13,7476	13.2374 12.5447 12.1394 11.6287 11.4664 11.2922 11.1587
10.10	20,0387	Table D				20.0387 20.0386	20.0365 20.0276 19.9951 19.9476 19.8923 19.8341 19.753	19,6065 19,1631 18,8693 18,6093 18,4153 18,2527 19,1127 17,9899	17.7819 17.1192 16.7241 16.2415 16.2715 16.0413 15.8886 15.7562	15,5346 14,8446 14,1402 14,1530 13,7461 13,5606 13,5606 13,5606	
	20.4850	Ta.				20.4850 20.4850 20.4845 20.4839	20.4812 20.4161 20.3564 20.2556 20.0744 20.0866 20.0035 19.9252	19.7827 19.2624 18.9281 18.6621 18.4581 18.2887 16.1437 17.9048	17,8038 17,1304 16,7315 16,4715 16,2560 16,0450 15,8918 15,7590	16.854 14.856 14.4569 14.15609 11.75609 11.5609 11.5609	
À-77-0	21.0604							55999995		2222222	
0.22.04	21,8713						222222222		2222222		
0.15.04	23,2576					23.2337 23.1313 22.9650 22.8650 22.6978 22.4550 22.1920 22.3920			77333325		
0.15-05	27.8606 27.8607 27.8608 27.8609 27.8610 27.8611	27.8613 27.8614 27.8616 27.8617	27.8635 27.8632 27.8622 27.8600 27.8565 27.8516 27.8546	27.8377 27.162 27.5703 27.4304 27.4304 27.1029 27.1874 26.9871	26.8194 26.2393 25.8034 25.0059 25.3948 25.0059 25.0059 24.9432 24.9298	24.7262 24.0455 23.6441 23.3585 23.1566 22.8052 22.6685 22.6685	22.4460 21.7541 21.3491 23.0616 20.8186 20.6593 20.503 20.3688 20.3510	20,1457 19,4527 19,0473 18,7596 18,7596 18,2565 18,2060 18,0665	17.8414 17.1502 16.7446 16.2340 16.2340 16.9517 15.9975 15.7640	15.5409 14.8427 14.1546 13.9314 13.7491 13.7491 13.4614 13.4614	13,5363 12,5451 11,6520 11,6260 11,4465 11,2924 11,2924
١/١	1 2 2 2 3 4 4 4 5 5 6 5 6 6 6 6 6 6 6 6 6 6 6 6 6	r. 66 4				100		1 0 1 4 4 4 4 4 6 6 6 7	10 1 10 1 11 10 10 10 10 10 10 10 10 10 10 10 10 1	400404040	

## Appendix E

## SYSTEM OF NUMBERING WELLS IN NEW MEXICO

All wells referred to in this report are identified by numbers used by the U. S. Geological Survey and the New Mexico State Engineer for locating water wells in New Mexico (figure E-1). The location number is a description of the geographic location of the well, based on the system of public land surveys. It indicates the location of the well to the nearest 10-acre tract, when the well can be located that accurately. The location description consists of a series of numbers corresponding to the township, the range, the section, and the tract within the section, in that order, as illustrated in figure E-1. If a well cannot be located accurately within a 10-acre tract, a zero is used as a third digit of the fourth segment of the location number. If the well cannot be located accurately within a 40-acre tract, zeros are used for both the third and second digits. All wells in the area covered by this report are east of the New Mexico Principal Meridian. The position of the wells north or south of the New Mexico base line is indicated by a headnote in each table. Springs are identified by the letter s preceding the township number.

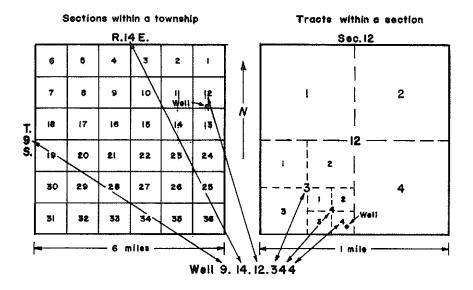


Figure E-1. System of numbering wells in New Mexico.