Report 239

GROUND-WATER RESOURCES AND MODEL APPLICATIONS FOR THE EDWARDS (BALCONES FAULT ZONE) AQUIFER IN THE SAN ANTONIO REGION

TEXAS DEPARTMENT OF WATER RESOURCES

REPORT 239

GROUND-WATER RESOURCES AND MODEL APPLICATIONS FOR THE EDWARDS (BALCONES FAULT ZONE) AQUIFER IN THE SAN ANTONIO REGION, TEXAS

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RESULTS AND CONCLUSIONS

The Edwards (Balcones Fault Zone) aquifer is a very important water resource in the San Antonio region supplying water for irrigation, to major spring systems, and to several municipalities, including the city of San Antonio. The aquifer consists of the Edwards and associated limestones of Cretaceous age which are in hydraulic continuity. The Edwards Formation is the most important rock unit in that it yields large quantities of water due to its extensive honeycombed and cavernous nature. The aquifer ranges in thickness from about 400 to 700 feet. The transmissibility of the Edwards ranges from less than 1,000 gallons per day per foot in the outcrop to over 20 million gallons per day per foot in the highly transmissive artesian zone within Bexar and Comal Counties. The average coefficient of storage in the outcrop of the Edwards is approximately 0.06. Downdip, where the aquifer is under artesian conditions, the average coefficient of storage approximates 0.0005.

The hydrologic boundaries of the Edwards (Balcones Fault Zone) aquifer in the San Antonio region are formed by the overlying Del Rio Clay and the underlying Glen Rose Formation. Lateral boundaries are as follows: (a) the northern edge of the Balcones fault zone on the north; (b) the ground-water divide northeast of Kyle in Hays County, that separates underflow toward Comal and San Marcos Springs from underflow to the Colorado River basin on the east; (c) the ground-water divide near Brackettville in Kinney County, that separates underflow toward Comal and San Marcos Springs from underflow to the Rio Grande basin on the west; and (d) an arbitrary line, commonly referred to as the "bad-water" line-south and southeast of this line the Edwards contains water having more than 1,000 milligrams per liter of total dissolved solids. This arbitrary line generally runs west-east through southern Kinney, Uvalde, and Medina Counties; the northern tip of Atascosa County; southeastern Bexar and Comal Counties; the western tip of Guadalupe County; and southeastern Hays County.

Water entering the Edwards (Balcones Fault Zone) aquifer moves generally southward across the reservoir and then eastward toward natural discharge points which include the following: (a) Leona River Springs near Uvalde; (b) San Antonio and San Pedro Springs in San Antonio; (c) Comal Springs at New Braunfels; and (d) San Marcos Springs at San Marcos. In addition, water is artifically discharged from the aquifer by hundreds of wells in the San Antonio region. The estimated average annual discharge from the aquifer by wells and springs was approximately 542,000 acre-feet and the estimated average annual recharge from precipitation and streamflow losses to the aquifer was approximately 531,000 acre-feet for the period 1934-71. Recharge to a lesser extent also occurs by lateral underflow from the Glen Rose Formation.

A digital computer model of the Edwards (Balcones Fault Zone) aquifer was successfully verified by taking historical input data (initial heads, pumpage, recharge, and other data) and computing water-level changes and spring flows which agree well with water-level declines and spring flows observed from actual field measurements.

The digital computer simulation for the period 1972 through 2049 of the Edwards (Balcones Fault Zone) aquifer in the San Antonio region indicates the following: (a) ignoring any water-quality constraints, the aquifer is capable of meeting projected demands through the year 2049; (b) natural flow of Comal and San Marcos Springs will cease before the year 2020, if projected pumpage in the region occurs at predicted rates; (c) the addition of currently proposed artificial recharge does not result in an appreciable increase in the aquifer's available water; (d) the drought-flood sequence of the 1950's was introduced at various points in the simulation and produced only minor effects when compared to the aquifer's long-term simulation; (e) spring flows at Comal Springs and San Marcos Springs can be maintained through ground-water management plans; and (f) there is sufficient storage in the aquifer to allow spring flow of Comal and San Marcos Springs to be replaced by augmentation pumpage through the year 2020.

GROUND-WATER RESOURCES AND MODEL APPLICATIONS FOR THE EDWARDS (BALCONES FAULT ZONE) AQUIFER IN THE SAN ANTONIO REGION, TEXAS

INTRODUCTION

Purpose and Scope

The purpose of this investigation was to determine the occurrence, availability, and dependability of the Edwards (Balcones Fault Zone) aquifer in the Nueces, San Antonio, Guadalupe-Blanco River basins and to develop a ground water resources management tool for use in a total water-resource management program for the three river basins.

The general scope of this investigation includes (a) the evaluation and synthesis, on a regional basis, of previously compiled geologic and hydrologic data; (b) the collection of additional geologic and hydrologic data in the field to be integrated with previously compiled data; and (c) the initiation of synthesis and analysis studies using digital computer modeling techniques in an effort to predict spring flows and future water levels under varying hydrologic and pumping conditions. The scope of the study was primarily directed toward the quantitative aspects of ground-water withdrawals, spring flows, and aquifer characteristics.

Also included within the scope of this study was the simultaneous initiation of the Edwards test well drilling investigation. The objectives of this investigation were (a) to describe the lithology of the stratigraphic units which make up the Edwards (Balcones Fault Zone) aquifer; (b) to determine the upper and lower hydrologic boundaries; (c) to determine the average total porosity for various aquifer levels; and (d) to determine the average effective porosity (specific yield) and the approximate artesian storage values for the aquifer through core analysis.

Location and Population

The area covered by this report will be referred to as the San Antonio region, whose boundaries coincide with the hydrologic boundaries of the Edwards (Balcones Fault Zone) aquifer in Atascosa, Bexar, Comal, Guadalupe, Hays, Kinney, Medina, and Uvalde Counties. The San Antonio region occurs within the Nueces, San Antonio, and Guadalupe-Blanco River basins. The aquifer extends approximately 175 miles from near Brackettville in Kinney County eastward to Kyle in Hays County and varies in width from about 5 to 40 miles. The location of the aquifer is illustrated in Figure 1. In this report, the Edwards (Balcones Fault Zone) aquifer is also referred to as the Edwards aquifer.

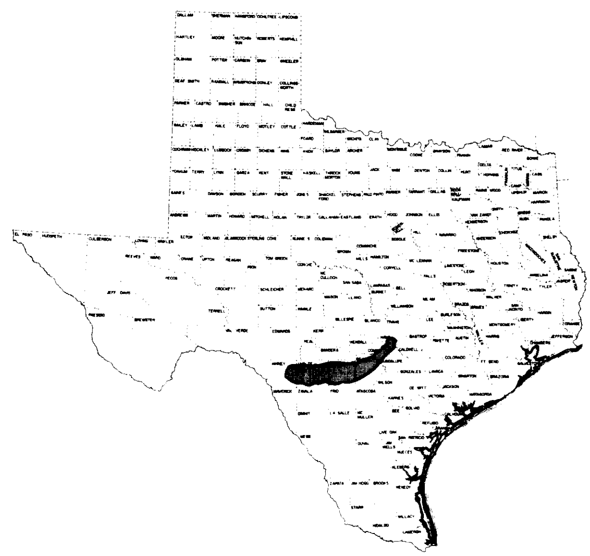

Figure 1.—Location of the Edwards (Balcones Fault Zone) Aquifer in the San Antonio Region

According to the 1972-73 Texas Almanac, the San Antonio region had an estimated 1970 population of 921,870. The largest trade center within the study area is San Antonio with a 1970 estimated population of 654,153. Other important industrial and agricultural centers are San Marcos, New Braunfels, Castroville, Hondo, Sabinal, Uvalde, and Brackettville. Population projections indicate Bexar County, in which San Antonio is located, will have a population of 1,260,900 by the year 2000 (oral communication, Arthur Simkins, December 1974), as compared with 830,460 in 1970.

Economy

The region derives its economy from military installations, governmental agencies, light industry, and from the production of various agricultural products. Ground water from the Edwards aquifer is used extensively for irrigation, public supply, and industry. Much of the light industry is concentrated in or near San Antonio and is related to the production of petroleum, natural gas, gravel, brick, tile, and cement.

In 1970 approximately 59,000 acres of land was irrigated from the aquifer, primarily in Bexar, Medina, and Uvalde Counties, in support of farming operations. The income during 1970 for the study area as reported in the 1972-73 Texas Almanac was in excess of $2.5 billion. The aquifer is essential to the present and future economic welfare of the San Antonio region, since it is presently the sole water supply for almost one million people.

Climate

Long hot summers and short mild winters are characteristic of the San Antonio region. Climatic conditions vary within the region from semiarid in the western part to subhumid in the eastern part. The mild climate with temperatures usually above freezing allows a growing season that averages about 262 days per year (Dallas Morning News, 1971).

The mean annual precipitation ranges from about 20 inches per year at Brackettville to about 33 inches per year at Kyle and generally occurs as isolated thundershowers. Most of the precipitation falls during the summer and early fall months as shown on Figure 2. Figure 2 also illustrates the location of selected precipitation and stream-gaging stations along with graphs of mean annual and average monthly precipitation for the period of record.

Personnel

This report was prepared under the general direction of Lewis B. Seward and under the direct supervision of Robert L. Bluntzer and William A. White. Data were collected and assembled by the authors with assistance from Board staff members Henry J. Alvarez, Tommy Barnes, Gail L. Duffin, Leonard (Nick) Carter, Glenn Merschbrock, Roger Wolff, Glenn Marquardt, and Eulogio Rodriguez, Jr.

The digital computer program used to simulate the Edwards (Balcones Fault Zone) aquifer was developed by T. A. Prickett and C. G. Lonnquist of the Illinois State Water Survey. The Texas Water Development Board staff, under the direction of Lial F. Tischler and William A. White, later modified the Prickett-Lonnquist program to simulate the complex properties of the aquifer. Recharge data necessary for making future application runs of the aquifer model. were supplied by Loyd W. Hamilton, and core drilling and laboratory testing of cores were done by the Board's Materials Testing Laboratory, under the direction of Henry Sampson.

Acknowledgements

The Texas Water Development Board appreciates the cooperation extended to its staff by the property owners in the San Antonio area. In most cases, this cooperation consisted of supplying information about their wells; however, in many instances it also included access to their property and the use of their wells to monitor water-level changes and to conduct pumping tests. Acknowledgement is also extended to city officials, water superintendents, officials of independent water districts, pump companies, water-well drillers, and to other consultants in the area for their assistance and cooperation during this investigation.

Mr. Robert P. Van Dyke of the San Antonio City Water Board, and William F. Guyton and Associates, Austin, made available to the Texas Water Development Board certain hydrological reports by various consultants for the City Water Board. This is greatly appreciated.

The Board also appreciates the cooperation extended to its staff by Mr. Porter Montgomery, Jr., consulting geologist. Mr. Montgomery was called upon numerous times for geological data and geophysical logs in the San Antonio area which he supplied most generously.

A debt of gratitude is extended to Mr. R. W. Bartlett, manager, Hill Country Water Works, and to Mr. Charles Lewis for the use of their property, equipment, and existing wells that enabled the Texas Water Development Board to drill test holes and conduct pumping tests.

The Board is likewise grateful to the State Department of Highways and Transportation, San

Antonio and Del Rio Districts, for the use of highway right-of-way as drilling sites for test holes and, in certain instances, the installation of permanent observation wells.

Finally, special acknowledgement is extended to Mr. Robert W. Maclay, United States Geological Survey, San Antonio Office, for his assistance and cooperation which contributed toward the successful completion of this investigation.

Definitions of Terms

The following definitions are intended to acquaint the reader with some of the terms used in this report. These definitions were derived from similar sections of previous publications; Glossary of Geology and Related Sciences (American Geological Institute, 1960); Handbook of Applied Hydrology (Chow, ed., 1964); and A Dictionary of Mining, Mineral, and Related Terms (Thrush and U.S. Bureau of Mines, 1968).

Acre-feet per year-One acre-foot per year equals 892.13 gallons per day.

Acre-foot-The volume of water required to cover 1 acre to a depth of 1 foot (43,560 cubic feet), or 325,851 gallons.

Alluvium or alluvial deposits-Sediments deposited by streams; includes flood-plain deposits.

Aquifer-A formation, group of formations, or part of a formation that is water bearing. An underground stratum that will yield water in sufficient quantity to be of value as a source of supply.

Aquifer test, pumping test-The test consists of the measurements, at specific time intervals, of the discharge and drawdown of the water level of the well being pumped and the drawdowns of the water levels in nearby observation wells. Formulas have been developed to show the relationship of the well yield to the shape and extent of the cone of depression and to calculate the hydraulic properties of the aquifer which are the coefficients of permeability, transmissibility, and storage.

Artesian aquifer, confined aquifer-An aquifer which is overlain (confined) by an impermeable layer so that the water is under hydrostatic pressure. The water level in an artesian well will rise above the top of the aquifer to the level of the piezometric surface; however, the well may or may not flow.

Cell-A rectangular subarea which resulted from segmenting the San Antonio region into smaller areas for the purpose of simulating the Edwards (Balcones Fault Zone) aquifer using a digital computer.

Coefficient of permeability-The rate of flow of water, in gallons per day, through a cross sectional area of 1 square foot under a unit hydraulic gradient.

Coefficient of storage-The volume of water an aquifer releases from or takes into storage per unit of surface area of the aquifer per unit change in the component of head normal to that surface.

Coefficient of transmissibility-The amount of water, in gallons, that will move in 1 day through a vertical strip of the aquifer 1 foot wide and having the height of the aquifer when the hydraulic gradient is unity. It is the product of the field coefficient of permeability and the saturated thickness of the aquifer.

Cone of depression-Depression of the water table or piezometric surface surrounding a discharging well which is more or less the shape of an inverted cone.

Confining bed or formation-One which, because of its position and its impermeability or low permeability relative to that of the aquifer, keeps the water in the aquifer under artesian pressure.

Dip of rocks-The angle or amount of slope at which a bed- is inclined from the horizontal; direction is also expressed (such as 1 degree, southeast; or 90 feet per mile, southeast).

Drainage basin-A surface stream or body of impounded surface water, together with all surface streams and bodies of impounded surface water that are tributary to it.

Drawdown-The lowering of the water table or piezometric surface caused by pumping or artesian flow. It is the difference, in feet, between the static level and the pumping level.

Electric log-A geophysical log showing the electrical properties of the rocks and their fluid contents penetrated in a well. The electrical properties are natural potentials and resistivities to induced electrical currents, some of which are modified by the presence of the drilling mud in and near the borehole.

Facies, lithologic-The “aspect” belonging to a geological unit of sedimentation including mineral composition, type of bedding, fossil content, etc. (such as sand facies). Sedimentary facies are areally segregated parts of differing nature belonging to any genetically related body of sedimentary deposits, and usually reflect differing conditions of deposition.

Fault-A fracture or fracture zone in a rock or body of rock along which there has been displacement of the two sides relative to one another parallel to the fracture.

Formation-A body of rock that is sufficiently homogeneous or distinctive to be regarded as a mappable unit.

Fresh water-Water containing less than 1,000 mg/l (milligrams per liter) of total dissolved solids.

Ground water-Water in the ground that is in the zone of saturation from which wells, springs, and seeps are supplied.

Head, or hydrostatic pressure-The height of the water table or piezometric surface above the base of the aquifer.

Hydraulic gradient-The slope of the water table or piezometric surface, usually given in feet per mile.

Irrigation-The controlled application of water to arable lands to supply water needs not satisfied by rainfall.

Lithology-The description of rocks, usually from observation of hand specimen or outcrop.

Node-The centers of the subareas (cells) used in the digital computer simulation of the Edwards (Balcones Fault Zone) aquifer in the San Antonio region.

Outcrop-That part of a rock layer which appears at the land surface.

Permeable-Pervious or having a texture that permits water to move through it perceptibly under the head differences ordinarily found in subsurface water. A permeable rock has communicating interstices of capillary or super-capillary size.

Porosity-The ratio of the aggregate volume of interstices (openings) in a rock or soil to its total volume, usually stated as a percentage.

Recharge of ground water-The process by which water is absorbed and is added to the zone of saturation. Also used to designate the quantity of water that is added to the zone of saturation.

Resistivity (electrical log)-The resistance of the rocks and their fluid content penetrated in a well to induced electrical currents. Permeable rocks containing fresh water have high resistivities.

Specific capacity-The rate of yield of a well per unit of drawdown, usually expressed as gallons per minute per foot (gal/min/ft) of drawdown. If the yield is 250 gallons per minute and the drawdown is 10 feet, the specific capacity is 25 gal/min/ft.

Specific capacity/foot of penetration-The rate of yield of a well per unit of drawdown per foot of borehole penetrating the aquifer, usually expressed as gallons per minute per foot of drawdown per foot of penetration.

Specific yield-The quantity of water that an aquifer will yield by gravity if it is first saturated and then allowed to drain; the ratio expressed in percentage of the volume of water drained to volume of the aquifer that is drained.

Storage-The volume of water in an aquifer, usually given in acre-feet.

Structural feature, geologic-The result of the deformation or dislocation (such as faulting) of the rocks in the earth's crust. In a structural basin, the rock layers dip toward the center or axis of the basin. The structural basin may or may not coincide with a topographic basin.

Water level-Usually expressed as the elevation of the water table or piezometric surface above mean sea level. Under artesian conditions the water level may be below or above the land surface.

Water table-The upper surface of a zone of saturation except where the surface is formed by an impermeable body of rock.

Water - table aquifer (unconfined aquifer) -An aquifer in which the water is unconfined; the upper surface of the zone of saturation is under atmospheric pressure only and the water is free to rise or fall in response to the changes in the volume of water in storage. A well penetrating an aquifer under water-table conditions becomes filled with water to the level of the water table.

Yield of a well-The rate of discharge, usually expressed in gallons per minute (gal/min). In this report, yields are classified as small, less than 50 gal/min; moderate, 50 to 500 gal/min; and large, more than 500 gal/min.

GEOLOGY AS RELATED TO THE OCCURRENCE OF GROUND WATER

Stratigraphy

R. T. Hill (1891) developed the traditional stratigraphic nomenclature for the geologic units which make up and are associated with the Edwards (Balcones Fault Zone) aquifer. Hill's work was done in north Texas and then applied to the stratigraphic units in south Texas. Later, Rose (1972) proposed new nomenclature which is more comprehensive and applicable to the depositional environments, facies and hydrogeologic units within the aquifer. Hill's and Rose's works are summarized in Table 1 which gives the old and new stratigraphic units, hydrogeologic units, depositional areas, and approximate thickness of the various units. The more recent nomenclature proposed by Rose is used in this report.

Structure

The Balcones fault zone is an area of extensive faulting that occurs in the San Antonio region. These faults are generally downthrown to the south and southeast and are related to the occurrence of ground water in the aquifer. The major faults trend east-northeastward, and the displacement is greater near the middle and diminishes toward the ends. The regional structure map (Figure 3) indicates maximum displacement to be about 600 feet at the Comal Springs fault, whereas the maximum single fault displacement in Uvalde and Medina Counties is about 200 feet. The location of the Balcones fault zone and outcrop of the major stratigraphic units are located on the generalized geologic map (Figure 4). The regional subsurface dip of the Edwards aquifer in the San Antonio region is about 100 feet per mile. Generally the aquifer dips to the south and southeast.

The following examples of major folding can be seen on the regional structure map, Figure 3: (a) the Culebra anticline which plunges from north central Bexar County southwest into northeastern Medina County; (b) an elongate anticlinal trend several miles in length east of San Antonio in the vicinity of Cibolo Creek; (c) a structural high northeast of Uvalde with associated faulting and basaltic intrusives; and (d) a structural high about 6 miles south of Sabinal.

The structural, stratigraphic, and hydrologic relationship of the various geologic units which make up the Edwards aquifer are shown in Figures 5, 6, 7, and 8. These geohydrolgic cross-sections were constructed from drillers' and geophysical logs and portray an interpretation of a vertical section of the earth's crust along a given line.

Regional Topographic and Land-Use Features

North of the Balcones fault zone, the Edwards and associated limestones and the Glen Rose Formation outcrop. These units form broad valleys, moderate to flat terrain, and alluvial filled streambeds. Maximum topographic relief ranges from 500 to 1,500 feet. The Glen Rose forms typical “hill” country terrain while the Edwards forms much of the grazing lands. Land use includes hunting, fishing, hiking, and the grazing of livestock. The Medina and Canyon Lake areas support recreation and community activities.

The resistant Edwards and Glen Rose Formations do not outcrop south of the Balcones fault zone. South of the major faults, maximum relief is about 100 feet, and the outcrops consist of soft upper Cretaceous strata and broad, extensive sand and gravel fan plains. These fans extend southward and eastward developing rich and well-drained loamy soils (Wermund, 1974). Land use includes the following: (a) cultivated land; (b) grazing land; (c) urban and community areas; and (d) recreation.

GEOHYDROLOGY OF THE EDWARDS (BALCONES FAULT ZONE) AQUIFER

The Edwards (Balcones Fault Zone) Aquifer Concept

The portion of the Edwards aquifer included in this study is approximately 175 miles in length extending from Brackettville in Kinney County eastward to Kyle in Hays County. The aquifer varies in width from 5 to 40 miles. Lateral boundaries of the aquifer are as follows: (a) the northern edge of the Balcones fault zone on the north; (b) the ground-water divide northeast of Kyle in Hays County, that separates underflow toward Comal and San Marcos Springs from underflow to the Colorado River basin on the east; (c) the ground-water divide near Brackettville in Kinney County, that separates underflow toward Comal and San Marcos Springs from underflow to the Rio Grande basin on the west; and (d) an arbitrary line, commonly referred to as the “bad-water” line, south and southeast of which the Edwards contains water having more than 1,000 milligrams per liter (mg/l) of dissolved solids. This arbitrary line generally runs west-east through southern Kinney, Uvalde, and Medina Counties; the northern tip

Table 1.—Comparison of Old and New Nomenclature Used for Stratigraphic Units Associated With the Edwards (Balcones Fault Zone) Aquifer and the Approximate Thickness of Each Unit

Hydrogeologic unit			Kinney County and western Uvalde County		Eastern Uvalde County and western Medina County		Eastern Medina County, Bexar, Comal, and Hays Counties	
			HILL (1891)	ROSE (1972)	HILL (1891)	ROSE (1972)	HILL (1891)	ROSE (1972)
Confining formation			Del Rio Clay 100 feet	Del Rio Clay 100 feet	Del Rio Clay 70 feet	Del Rio Clay 70 feet	Del Rio Clay 50 feet	Del Rio Clay 50 feet
Edwards (Balcones Fault Zone) Aquifer	Edwards and Associated Limestones	Upper	Georgetown Formation 380 feet Kiamichi Formation 150 feet	Salmon Peak Formation 380 feet McKnight Formation 150 feet	Georgetown Formation 50 feet Edwards Formation 500 feet	Georgetown Formation 50 feet Devils River Formation 550 feet	Georgetown Formation 25 feet Edwards Formation 420 feet	Georgetown Formation 25 feet Person Formation 200 feet
		Middle		Regional Dense Bed Equivalent 40 feet	?	Regional Dense Bed Equivalent 30 feet		Regional Dense Bed Member 20 feet
		Lower	Edwards Formation 70 feet Comanche Peak Formation 70 feet	West Nueces Formation 140 feet	Comanche Peak Formation 30 feet Walnut Formation 20 feet	?	Comanche Peak Formation 30 feet Walnut Formation 10 feet	Kainer Formation 260 feet
Confining formation			Glen Rose Formation 1,000 feet	Glen Rose Formation 1,000 feet	Glen Rose Formation 1,000 feet	Glen Rose Formation 1,000 feet	Glen Rose Formation 900 feet	Glen Rose Formation 900 feet

of Atascosa County; southeastern Bexar and Comal Counties; the western tip of Guadalupe County; and southeastern Hays County.

The “bad-water” line is thought to be related primarily to a change in lithology in the aquifer and to a decrease in permeability. The location of the “bad-water” line does not appear to be significantly associated with the structure as shown in Figure 3.

The Edwards aquifer consists of the Edwards and associated limestones of Cretaceous age as illustrated in Table 1. The aquifer is a heterogeneous unit composed of hard, porous and fossiliferous limestones and dolomites that are dissected by faults and joints throughout the San Antonio region. The relatively high porosity and permeability of the aquifer have resulted from the enlargement of vugs, faults, and joints by solution. Solution channels along bedding planes and the recrystallization of limestone have also contributed to greater porosity and permeability.

Water in the aquifer is under both unconfined and confined conditions. The confined portion is the most extensive and productive. Well yields range from small to large in the artesian zone while wells in the outcrop yield small to moderate amounts of ground water.

Recharge, Movement, and Discharge

Recharge to the Edwards (Balcones Fault Zone) aquifer occurs primarily by infiltration of surface water from streams which traverse the outcrop. All of the streams which cross the outcrop lose water to the aquifer except the Guadalupe River. Recharge to a lesser extent occurs by direct infiltration of precipitation on the outcrop and by lateral underflow from the Glen Rose Formation. Based on a 2O-year average, Lowry (19551 calculated additional recharge equal to 5.4 percent of precipitation and streamflow losses to the aquifer was derived from the Glen Rose Formation. However, the authors estimate that 6 percent would be nearer the actual amount. This amount of recharge reaches the aquifer without ever having been measured at a stream gage.

Precipitation measurements at stations near the outcrop and discharge measurements of the streams and rivers at stream-gaging stations above and below the outcrop of the aquifer provide data from which estimates of recharge to the aquifer are made. Referring to Figure 2, the periods of large stream loss (recharge) occur during periods of high precipitation. The estimated average annual recharge from precipitation and stream loss for the period 1934-71 was about 531,400 acre-feet. The annual recharge for each subbasin, which is monitored by the U.S. Geological Survey, is given in Table 2 for the period 1934-71.

Artificial recharge to the aquifer has been attempted on a small scale for many years in Uvalde and Medina Counties with good success. Dams have been built across intermittent streams in the outcrop to retard floodwaters so that the water could enter the aquifer. The best examples of this technique are Medina Lake and Diversion Dam Lake in northeastern Medina County. These dams were not constructed as recharge structures; however, they do recharge approximately 42,000 acre-feet per year. The Edwards Underground Water District constructed Parker Creek Dam in northern Medina County in 1974 as a recharge structure designed to recharge approximately 500 acre-feet per year. Montell, Concan, and Sabinal Dams were proposed by the U.S. Army Corps of Engineers as recharge structures in the Nueces River basin. These dams could contribute approximately 63,900 acre-feet of recharge per year. Numerous structures in Uvalde, Medina, Bexar, Hays, and Comal Counties are planned by the U.S. Soil Conservation Service and Edwards Underground Water District to control floodwaters and provide recharge to the aquifer. These structures could contribute approximately 16,000 acre-feet of recharge per year.

Water entering the aquifer in the outcrop generally moves south and southeastward under steep hydraulic gradients and low permeabilities toward the confined part of the aquifer. When reaching the artesian zone, the water moves under low hydraulic gradients and high permeabilities toward the east and northeast where it is discharged through wells and springs. Figures 9 and 10 show the approximate depth to and altitude of water levels in the Edwards aquifer for January 1947 and January 1972, respectively.

The water in the aquifer is discharged naturally at the following locations: (a) the Leona River Springs near Uvalde; (b) San Antonio and San Pedro Springs in San Antonio; (c) Comal Springs in New Braunfels, and (d) San Marcos Springs in San Marcos. The correlation of water levels in the Landa Park well at New Braunfels with the discharge at Comal Springs is shown in Figure 11. Although not shown, the other springs in the study area display similar correlation. These springs issue along faults that have been developed into open cracks and solution channels.

In 1969 approximately 700 high-capacity wells in Uvalde, Medina, Bexar, Comal, and Hays Counties discharged water from the aquifer. The water was used for irrigation, industrial, and municipal purposes. Population centers which rely solely on the Edwards

Table 2.—Estimated Annual Recharge to the Edwards (Balcones Fault Zone) Aquifer, 1934-71

(Recharge is given in thousands of acre-feet by drainage basin.)

Year	Nueces and West Nueces River basins	Frio and Dry Frio River basins	Sabinal River basin	Area between Sabinal and Medina River basins	Medina River basin	Area between Cibolo Creek and Medina River basins	Cibolo and Dry Comal Creek basins	Blanco River basin and adjacent area	Total
1934	8.6	27.9	7.5	19.9	46.5	21.0	28.4	19.8	179.6
1935	411.3	192.3	56.6	166.2	71.1	138.2	182.7	39.8	1,258.2
1936	176.5	157.4	43.5	142.9	91.6	108.9	146.1	42.7	909.6
1937	28.8	75.7	21.5	61.3	80.5	47.8	63.9	21.2	400.7
1938	63.5	69.3	20.9	54.1	65.5	46.2	76.8	36.4	432.7
1939	227.0	49.5	17.0	33.1	42.4	9.3	9.6	1.1	389.0
1940	50.4	60.3	23.8	56.6	38.8	29.3	30.8	18.8	308.8
1941	89.9	151.8	50.6	139.0	54.1	116.3	191.2	57.8	850.7
1942	103.5	95.1	34.0	84.4	51.7	66.9	93.6	28.6	557.8
1943	36.5	42.3	11.1	33.8	41.5	29.5	58.3	20.1	273.1
1944	64.1	76.0	24.8	74.3	50.5	72.5	152.5	46.2	560.9
1945	47.3	71.1	30.8	78.6	54.8	79.6	129.9	35.7	527.8
1946	80.9	54.2	16.5	52.0	51.4	105.1	155.3	40.7	556.1
1947	72.4	77.7	16.7	45.2	44.0	55.5	79.5	31.6	422.6
1948	41.1	25.6	26.0	20.2	14.8	17.5	19.9	13.2	178.3
1949	166.0	86.1	31.5	70.3	33.0	41.8	55.9	23.5	508.1
1950	41.5	35.5	13.3	27.0	23.6	17.3	24.6	17.4	200.2
1951	18.3	28.4	7.3	26.4	21.1	15.3	12.5	10.6	139.9
1952	27.9	15.7	3.2	30.2	25.4	50.1	102.3	20.7	275.5
1953	21.4	15.1	3.2	4.4	36.2	20.1	42.3	24.9	167.6
1954	61.3	31.6	7.1	11.9	25.3	4.2	8.8	10.7	160.9
1955	128.0	22.1	.6	7.7	16.5	4.3	3.3	9.5	192.0
1956	15.6	4.2	1.6	3.6	6.3	2.0	2.2	8.2	43.7
1957	108.6	133.6	65.4	129.5	55.6	175.6	397.9	76.4	1,142.6
1958	266.7	300.0	223.8	294.9	95.5	190.9	268.7	70.7	1,711.2
1959	109.6	158.9	61.6	96.7	94.7	57.4	77.9	33.6	690.4
1960	88.7	128.1	64.9	127.0	104.0	89.7	160.0	62.4	824.8
1961	85.2	151.3	57.4	105.4	88.3	69.3	110.8	49.4	717.1
1962	47.4	46.6	4.3	23.5	57.3	16.7	24.7	28.9	249.4
1963	39.7	27.0	5.0	10.3	41.9	9.3	21.3	16.2	170.7
1964	126.1	55.1	16.3	61.3	43.3	35.8	51.1	22.2	411.2
1965	97.9	83.0	23.2	104.0	54.6	78.8	115.3	66.7	623.5
1966	169.2	134.0	37.7	78.2	50.5	44.5	66.5	34.6	615.2
1967	82.2	137.9	30.4	65.0	44.7	30.2	57.3	19.0	466.7
1968	130.8	176.0	66.4	198.7	59.9	83.1	120.5	49.3	884.7
1969	119.7	113.8	30.7	84.2	55.4	60.2	99.9	46.6	610.5
1970	112.6	141.9	35.4	81.6	68.0	68.8	113.8	39.5	661.6
1971	263.4	212.4	39.2	150.3	68.7	81.4	82.4	22.2	920.0
Average	100.8	91.2	32.4	75.1	51.8	57.6	90.5	32.0	531.4

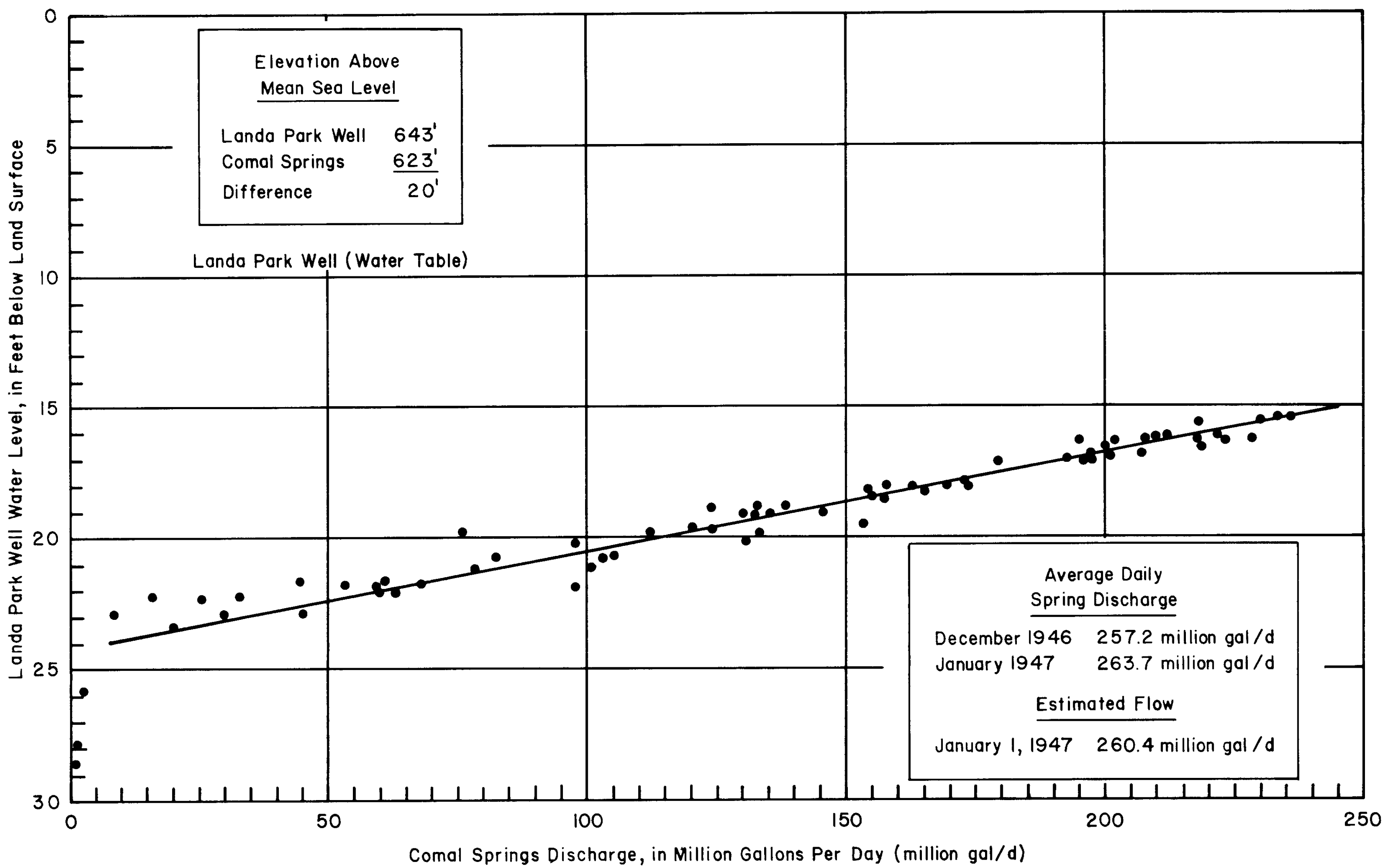

Figure 11

Correlation of Water Levels in Landa Park Well With the Discharge of Comal Springs

aquifer for their water include Uvalde, Sabinal, D'Hanis, Hondo, Castroville, San Antonio, New Braunfels, San Marcos, and Kyle. The estimated annual discharge for the period 1934-71 was about 641,900 acre-feet. Table 3 gives the total discharge by county for the years 1934-71. Figure 12 shows the relationship of well discharge and spring discharge to recharge from 1934 through 1971.

Analysis of Test Wells

The test wells cored in the Edwards aquifer and upper Glen Rose Formation are located on Figure 3. Table 4 presents the results obtained from the borehole geophysical logs, well-site logs, and laboratory core analysis data collected during the study. The laboratory core analysis includes the following:

> *adjusted laboratory porosity*-the ratio of the volume of the void space to the total volume of the sample, in percent by volume, adjusted to include cavities penetrated by assuming that all cavities are 100 percent porous;
>
> *absorption*-the amount of water absorbed by a sample as compared to its dry weight, adjusted to percent by volume;
>
> *estimated effective porosity*-the difference between the adjusted laboratory-measured porosity and the absorption, in percent by volume;
>
> *unit weight or bulk density*-the weight per unit volume, in pounds per cubic foot (lb/ft^3);
>
> *vertical permeability*-a measure of the sample's vertical flow rate of water, in gallons per day per square foot ($gal/d/ft^2$);
>
> *compressive strength*-the load per unit area at which the sample fails by shear or splitting, in pounds per square inch (psi); and
>
> *modulus of elasticity*-the ratio of the mean normal stress to the change in volume per unit volume, in pounds per square inch (psi).

The aquifer results are given for the upper, middle, and lower units; a weighted average value for each aquifer parameter is also given based upon the thickness of the three units. Results for the Glen Rose Formation provide hydrogeologic parameters for the lower boundary of the Edwards aquifer. All values are based on an average per foot.

Core analysis indicates the following: (a) there are three porosity horizons within the aquifer-upper, middle, and lower; (b) the effective porosity of the upper unit is greater than the middle or lower units; (c) the average effective porosity (specific yield) of the aquifer is approximately 6.3 percent; (d) the average total porosity is approximately 17.2 percent; and (e) the average artesian storage of the aquifer is approximately 3×10^{-4}, according to Sieh.[1] The authors estimate that, based on aquifer thickness and taking into account the cavities and solution channels which could not be tested, the artesian storage coefficients should range from 4×10^{-4} to 8×10^{-4}.

The following generalizations are based on observations of the cores at the test well sites: (a) only the sandy or sugary appearing samples have primary porosity; (b) the crystalline, hard, and dense samples are associated with secondary porosity more than any other matrix type; (c) the earthy or chalky samples are least likely to be associated with secondary porosity; (d) the presence of iron or manganese in the samples usually indicates good permeability and effective porosity; (e) the best effective porosity appears to develop near fractures; and (f) the majority of the effective porosity is secondary (channels, vugs, fractures, and molds).

Analysis of Pumping Tests, Step Drawdown, and Specific Capacities

The inadequacy of common analytical methods to describe ground-water flow in carbonate rocks has been a topic of discussion for a long time due to the following: (a) the basic assumption of most equations is that flow takes place in a homogeneous medium; (b) carbonate rocks have little primary porosity; voids are in the form of joints, fractures, and solution channels; and (c) some flow in a carbonate aquifer is similar to flow through a rough pipe rather than a homogeneous medium.

However, Eagon and Johe (1972) state, "One of the difficulties in working with carbonate-rock aquifers is the seeming inconsistency in the hydraulic characteristics of wells within a small area. To a great extent this is caused by conditions in the vicinity of the borehole. These irregularities may be of great consequence initially, but usually dwindle to small importance as the cone of depression becomes very large. The larger the area considered, the more nearly

[1] Sieh, T. t-t., 1975, Edwards (Balcones Fault Zone) aquifer test well drilling investigation: Texas Water Devel. Board unpublished file rept., 127 p.

Table 3.—Estimated Annual Pumpage and Total Spring Flow, From the Edwards (Balcones Fault Zone) Aquifer, 1934-71

(Pumpage and spring flow is given in thousands of acre-feet)

	Estimated pumpage						
Year	Uvalde and eastern Kinney Counties	Medina County	Bexar County	Comal County	Hays County	Total spring flow*	Total pumpage and spring flow
1934	2.8	1.3	95.7	1.0	1.1	336.0	437.9
1935	2.2	1.5	97.8	1.0	1.1	416.0	519.6
1936	2.6	1.5	107.8	1.6	1.1	483.6	598.2
1937	3.2	1.5	113.4	1.0	1.0	451.1	571.2
1938	2.8	1.5	113.2	2.0	1.1	437.2	557.8
1939	3.2	1.6	112.0	1.3	1.0	313.9	433.0
1940	3.0	1.6	113.6	2.0	1.1	295.3	416.6
1941	2.9	1.6	129.7	1.5	1.1	464.4	601.2
1942	3.3	1.7	136.1	2.2	1.2	450.2	594.7
1943	4.2	1.7	140.0	2.1	1.2	390.1	539.3
1944	2.9	1.7	140.6	2.3	1.3	418.6	567.4
1945	4.1	1.7	143.7	2.3	1.4	461.6	614.8
1946	4.3	1.7	145.5	2.1	1.4	428.9	583.9
1947	4.8	2.0	157.1	1.6	1.5	426.5	593.5
1948	6.6	1.9	157.0	2.4	1.7	281.0	450.6
1949	8.2	2.0	165.1	2.4	1.7	300.4[1]	479.8
1950	10.5	2.2	177.3	2.0	1.8	272.9	466.7
1951	16.9	2.2	186.8	1.9	1.8	216.0[2]	425.6
1952	22.7	3.1	187.1	1.1	1.9	209.0	424.9
1953	27.6	4.0	193.7	2.6	2.0	238.4	468.3
1954	26.7	6.3	208.8	2.5	2.0	178.0	424.3
1955	28.4	11.1	215.2	3.6	2.7	127.8	388.8
1956	59.6	17.7	229.6	10.5	3.8	69.7[3]	390.9
1957	28.2	11.9	189.4	7.4	2.7	216.9	456.5
1958	20.1	6.6	185.5	4.6	2.5	398.2	617.5
1959	25.6	8.3	193.3	4.9	2.5	384.4	619.0
1960	24.1	7.6	189.0	4.6	2.1	428.0	655.4
1961	25.7	6.4	188.4	5.3	2.5	455.2	683.5
1962	40.2	8.1	211.3	5.5	2.9	321.0	589.0
1963	41.4	9.7	216.8	5.4	3.2	239.5	516.0
1964	42.9	8.6	201.0	4.9	2.8	213.8	474.0
1965	39.6	10.0	197.4	5.7	3.4	322.8	578.9
1966	40.2	10.4	195.9	5.3	3.9	315.1	570.8
1967	74.0	15.2	239.2	8.0	4.6	216.0	557.0
1968	41.4	9.9	190.2	6.5	3.7	408.3	660.0
1969	70.8	13.6	211.5	7.5	4.1	351.2	658.7
1970	75.8	16.5	223.7	8.0	5.1	397.4	726.5
1971	97.1	32.3	260.8	9.3	7.3	272.7	679.5
Average	24.8	6.5	172.6	3.8	2.4	331.8	541.9

*Total spring flow includes Leona River, San Antonio, San Pedro, Comal, and San Marcos Springs. Approximately 90 percent of the total spring flow is from Comal and San Marcos Springs.

[1] San Antonio and San Pedro Springs did not flow from 1949-1957 and 1963-1964.

[2] Leona River Springs did not flow from 1951-1957.

[3] Comal Springs ceased flowing during June-November 1956.

Table 4.—Results of Core Tests Penetrating the Edwards (Balcones Fault Zone) Aquifer and the Upper Glen Rose Formation

Unit	Adjusted laboratory porosity (percent by volume)	Absorption (percent by volume)	Estimated effective porosity (percent by volume)	Unit weight or bulk density (lb/ft^3)	Vertical permeability (gal/ft^2)	Compressive strength (psi)	Modulus of elasticity (psi)	Cavities penetrated (ft/ft)
Bexar County								
Test Well AY-1								
Edwards[1]								
Upper	17.7	5.5	12.2	158.0	0.038	7,590	1,850,000	0.013
Middle	7.1	4.9	2.2	163.0	.000	8,080	2,491,000	.000
Lower	18.9	12.5	6.4	155.0	.006	5,540	1,360,000	.016
Weighted Average	17.9	9.3	8.6	157.0	.019	6,500	1,620,000	.014
Glen Rose	19.7	15.0	4.7	153.0	.029	5,510	1,470,000	.000
Comal County								
Test Well DX-2								
Edwards[1]								
Upper	14.6	6.2	8.4	158.0	.000	11,000	2,140,000	.020
Middle	15.3	9.1	6.2	159.0	.000	12,500	2,410,000	.000
Lower	15.9	7.8	8.0	158.0	.001	10,000	2,160,000	.004
Weighted Average	15.4	7.3	8.1	158.0	.000	10,500	2,160,000	.010
Glen Rose	15.7	10.6	5.1	160.0	.000	4,450	1,120,000	.000

See footnote at end of table.

Table 4.—Results of Core Tests Penetrating the Edwards (Balcones Fault Zone) Aquifer and the Upper Glen Rose Formation—Continued

Unit	Adjusted laboratory porosity (percent by volume)	Absorption (percent by volume)	Estimated effective porosity (percent by volume)	Unit weight or bulk density (lb/ft^3)	Vertical permeability (gal/ft^2)	Compressive strength (psi)	Modulus of elasticity (psi)	Cavities penetrated (ft/ft)
Kinney County								
Test Well RP-2								
Edwards								
Upper	25.5	15.7	9.8	142.0	0.060	4,700	921,000	0.051
Middle	3.2	2.2	1.0	155.0	.000	9,000	1,240,000	.000
Lower	6.2	5.0	1.2	159.0	.000	10,900	1,620,000	.000
Weighted Average	20.0	12.6	7.4	146.0	.043	6,300	1,090,000	.037
Glen Rose	3.6	1.6	2.0	164.0	.000	15,400	1,310,000	.000
Medina County								
Test Well TD-3								
Edwards								
Upper	21.4	12.1	9.3	151.0	.238	4,420	1,150,000	.011
Middle	21.3	16.2	5.1	149.0	.060	7,380	1,240,000	.000
Lower	16.3	11.8	4.6	154.0	.096	6,150	1,120,000	.003
Weighted Average	18.7	12.2	6.5	152.0	.152	5,520	1,140,000	.006
Glen Rose	9.2	9.0	.2	157.0	.000	10,000	1,580,000	.000

See footnote at end of table.

Table 4.—Results of Core Tests Penetrating the Edwards (Balcones Fault Zone) Aquifer and the Upper Glen Rose Formation—Continued

Unit	Adjusted laboratory porosity (percent by volume)	Absorption (percent by volume)	Estimated effective porosity (percent by volume)	Unit weight or bulk density (lb/ft^3)	Vertical permeability (gal/ft^2)	Compressive strength (psi)	Modulus of elasticity (psi)	Cavities penetrated (ft/ft)
				Uvalde County				
				Test Well YP-4				
Edwards								
Upper	16.5	13.0	3.5	146.0	0.053	5,860	984,000	0.005
Middle	10.6	10.1	.5	153.0	.000	8,590	1,020,000	.000
Lower	7.3	7.6	.0	162.0	.000	14,300	2,040,000	.003
Weighted Average	13.7	11.4	2.3	151.0	.036	8,260	1,270,000	.004
Glen Rose	6.3	5.1	1.2	161.0	.000	10,700	1,660,000	.000

[1] Various intervals were roller-bit drilled with no returns. Estimates were made from borehole geophysical logs, core testing in other sections of the hole, and drilling rate for these intervals.

some carbonate-rock aquifers effectively assume the hydraulic characteristics of a homogeneous medium."

The Edwards aquifer derives most of its permeability from secondary porosity (joints, fractures, vugs, and solution channels) which are interconnected on an areal basis. With this in mind, the authors propose reasonable results can be obtained using standard analytical methods to approximate the transmissibility of the aquifer in the San Antonio region.

Aquifer tests were conducted at three of the test well sites (water-table conditions) to determine coefficients of storage and transmissibility. This work involved pumping a nearby irrigation or public supply well and making periodic measurements of water-level drawdowns in the test well (observation well) and if possible in the pumping well. From the data obtained, the coefficients of transmissibility and storage were calculated using the nonleaky artesian formula (Walton, 1962). The equation derived by Jacob (1944) was used to adjust drawdown data for the decrease in transmissibility due to dewatering.

The following table summarizes the pumping test results associated with the Board's test well drilling investigation.

Test well	Saturated thickness (ft)	Coefficient of transmissibility (gal/d/ft)	Coefficient of storage	Coefficient of permeability (gal/d/ft^2
AY-1	255	35,800	0.02	140
AY-2	214	12,500	.0007	58
TO-3	481	386,000	.0004	802

Aquifer tests utilizing the test wells as observation wells gave good results when estimating the coefficients of transmissibility and permeability. Only the storage coefficient obtained at well AY-1 appears reasonable as the aquifer is under water-table conditions at that location. Storage values obtained from wells AY-2 and TD-3 approximate artesian storage and are therefore too low as the aquifer is under water-table conditions at these locations. These inconsistent storage-coefficient values could be caused by the interconnected fractures between the test sites allowing rapid communication between the pumping well and observation well.

Many contractor step tests were collected and analyzed using the following equation which considers both laminar and turbulent flow (Jacob, 1946):

$$s=BQ+CQ^2,$$

where

s = drawdown, in feet;

B = aquifer constant, in set per ft^2 ;

C = well-loss constant, in sec^2 per ft^5; and

Q= pumping rate, in ft^3 per sec.

Bruin and Hudson (1955) developed a graphical method for solving the above equation which affords the advantage of being able to average the collected data and also does not require the conversion of the pumping rate to cubic feet per second. The BQ term approximates drawdown that would occur in a well having no well loss. The CQ^2 terms approximates well loss. The objective of this analysis was to determine the BQ term in order to approximate a no well loss specific capacity for each well.

Contractor specific capacities were also collected and corrected in order to approximate the no well loss condition. This was done by using pipe friction tables to estimate the CQ^2 and BQ terms. The BQ term was then used to estimate the no well loss specific capacity.

Specific capacities per foot of penetration were computed for each test using the drillers' log and the pump-test data provided by the contractor. Large values of specific capacity per foot of penetration were noted in many cases where the penetration was small. In order to compare values with like penetration, the values of specific capacity per foot of penetration and percent of the aquifer penetrated were graphed to establish an empirical correlation factor. When penetration was under 55 percent, the values for specific capacity per foot of penetration were multiplied by these correction factors in order to obtain adjusted values. Only a small percentage of the tests used had penetration values under 55 percent. Values for specific capacity per foot of penetration were plotted on a grid map. These values were averaged in both north-south and east-west directions, thus producing a series of moving averages which were the final plotted values.

The transmissibility of the Edwards aquifer was obtained by the following three steps: (a) the moving average specific capacity per foot of penetration was multiplied by 1,990 (a factor for artesian conditions) or 1,460 (for water-table conditions) to approximate the permeability (Thomasson and others, 1960); (b) these permeability values were then multiplied by the aquifer thickness to obtain transmissibilities, which were then plotted and contoured; and (c) later the map transmissibilities were input into the Edwards digital computer model and revised on an areal basis in order for the model to better simulate water-level changes.

In those areas where artesian conditions exist, transmissibility is thought to be greater in the east-west direction than the north-south direction. The estimated composite transmissibility of the aquifer is shown on Figure 13. This map illustrates that the highly transmissive center portion of the aquifer is bounded by relatively low transmissibilities in the outcrop and adjacent to the "bad-water" line.

Changes in Water Levels

The most significant causes of water-level fluctuations are changes in aquifer storage which is regulated by recharge and discharge. During periods of drought, recharge is reduced and some of the water discharged from the aquifer must be withdrawn from storage. This causes water levels to decline. However, when adequate rainfall resumes, the volume of water drained from storage will be replaced and water levels will rise accordingly.

Large, localized withdrawals of ground water occur in the Edwards aquifer, however, the aquifer's response is in terms of regional water-level fluctuations. This is due primarily to the high transmissibility of the Edwards which allows large volumes of water to move over wide areas to points of discharge. Response in the outcrop (water-table conditions) is generally less pronounced than in the artesian areas due to the coefficient of storage which is approximately 100 times larger than the artesian storage coefficient.

The fluctuations of static water levels for representative wells are illustrated in Figure 14 for the period 1940 through 1971. The largest long-term fluctuation occurred during the widespread drought of the 1950's when water levels slowly declined due to removal of water from storage (1947 through August 1956). Later after the drought was broken, large volumes of ground water were recharged to the aquifer and water levels rose rapidly until returning to the 1947 level. Smaller seasonal fluctuations, which are generally the result of seasonal changes in recharge and discharge, are also illustrated in Figure 14.

THE DIGITAL COMPUTER MODEL OF THE EDWARDS (BALCONES FAULT ZONE) AQUIFER

An objective of this study was to develop a ground-water management tool for use in a total water resource management program for the Nueces, San Antonio, and Guadalupe-Blanco River basins. The management tool developed was a digital computer model of the Edwards (Balcones Fault Zone) aquifer. This model simulates water levels and spring flows based on the physical constants of the system and on the recharge and pumpage rates for the aquifer.

The computer program written to perform the Edwards simulation was called *GWSIM, Groundwater Simulation Program*, and the program documentation and user's manual was prepared in 1974 by the Texas Water Development Board. The basic simulation program was written by T. A. Prickett and C. G. Lonnquist, Illinois State Water Survey (Prickett and Lonnquist, 1971). Modifications were made to the basic program to allow better simulation of an aquifer containing both artesian and water-table zones.

Derivation of Governing Equations

The numerical simulation of the aquifer is based on a mathematical approximation of the basic ground-water flow equation. The equation for nonsteady flow in a nonhomogeneous aquifer was used by Prickett and Lonnquist (1971) and may be written as follows:

$$\frac{\partial}{\partial x}\left[T\frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[T\frac{\partial h}{\partial y}\right] = S\frac{\partial h}{\partial t} + Q,$$

where

T = aquifer transmissibility (Length squared/Time);

h = hydraulic head (Length);

S = aquifer storage coefficient;

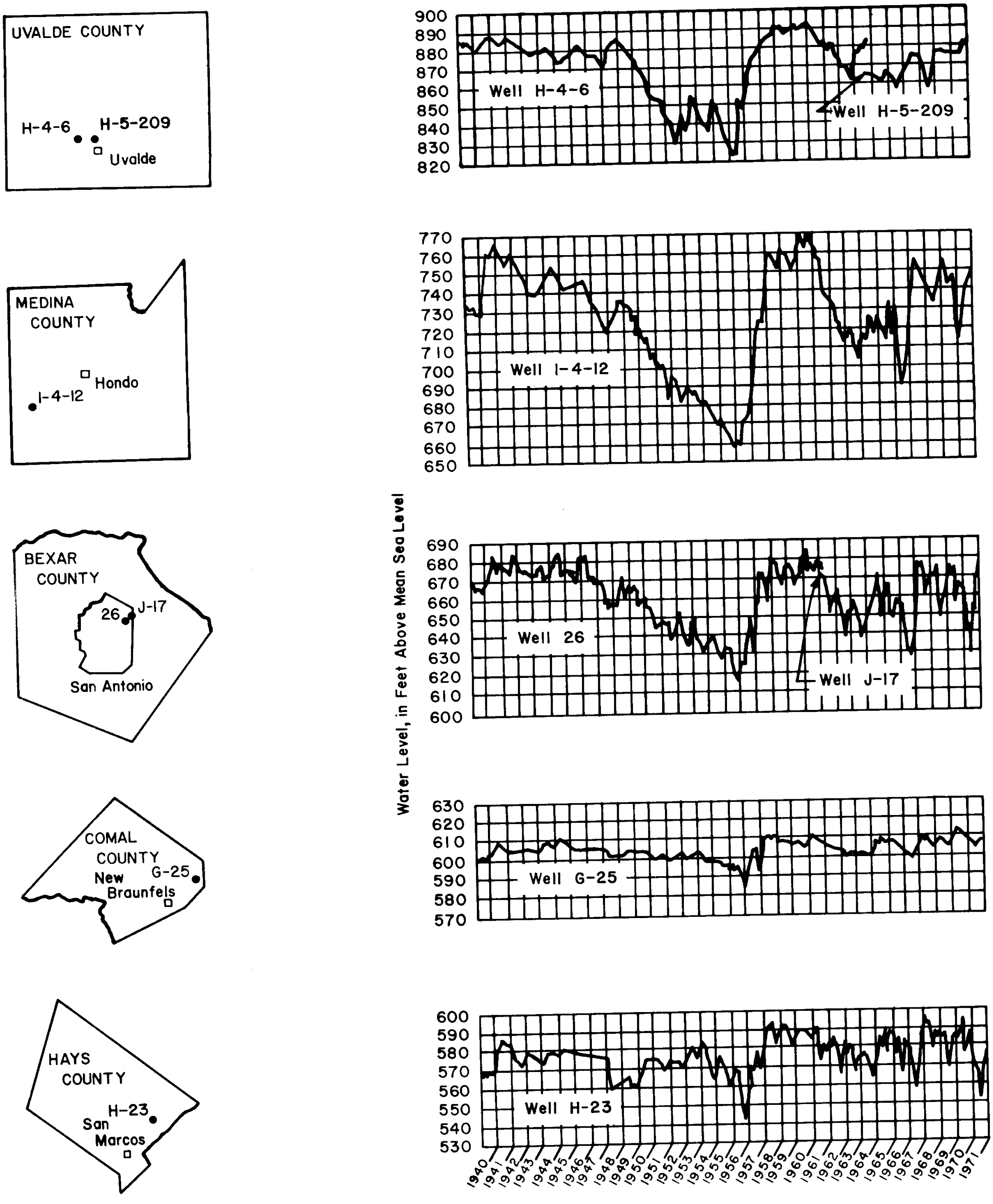

Figure 14

Hydrographs of Representative Wells in the Edwards (Balcones Fault Zone) Aquifer, 1940-71 (Modified From Garza,1966)

t = time (Time);

Q = net ground-water flux per unit area (Length/Time);

x,y = rectangular coordinates (Length).

Solution Technique

The numerical solution to this equation can be obtained by applying a finite difference approach. The basic assumption underlying the finite difference approach is that partial differentials can be approximated by a difference quotient.

The steps in applying the finite difference approach to ground-water movement are as follows:

(a) a finite difference grid is superimposed upon a map showing the extent of the aquifer, with the coordinate axes aligned with the principal directions of the transmissibility tensor, thus allowing the finite difference grid to replace the continuous aquifer with an equivalent set of discrete elements;

(b) the governing partial differential equation is written in finite difference form for each of the discrete elements; and

(c) the resulting set of linear finite difference equations are solved numerically for the head with the aid of a digital computer.

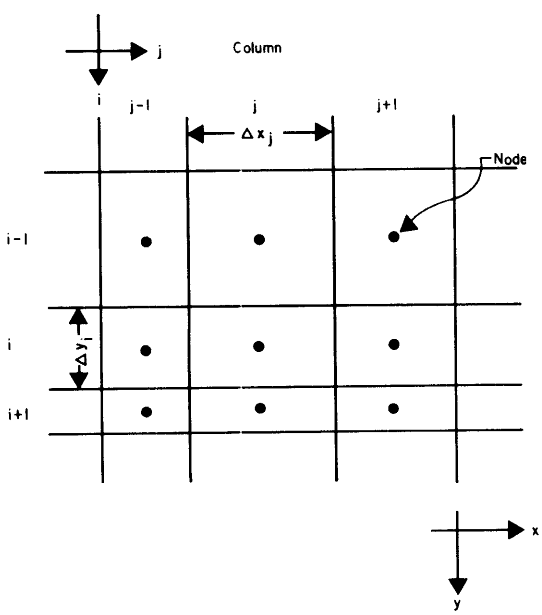

Figure 15.—Finite Difference Grid

A portion of a finite difference grid which could be superimposed upon a map is illustrated in Figure 15. Each of the grid elements is referred to as a cell, and the center of each cell is called a node. Each of the cells has dimensions $m\Delta x\Delta y$, where m is the thickness of the cell and Δx and Δy are the grid dimensions in the x and y directions, respectively. Each of the cells, or nodes, may be referenced by its row (i) and column (j) numbers which correspond to the y and x dimensions.

The spring flows were treated in the same manner as spills from a reservoir are approximated. That is, once the head is above some minimum level, the springs would begin to flow. The spring flow was assumed to increase linearly with head. Once the head is above a minimum level the spring would flow as a linear function of head.

The finite difference approximation of the basic ground-water flow equation which was used to simulate the Edwards aquifer may be expressed as follows:

$$
\begin{aligned}
& T_{i,j-1,1}\,(h'_{i,j-1} - h'_{i,j})\,\frac{\Delta y}{\Delta x}, \\
& + T_{i,j,2}\,(h'_{i+1,j} - h'_{i,j})\,\frac{\Delta x}{\Delta y}, \\
& + T_{i,j,1}\,(h'_{i,j+1} - h'_{i,j})\,\frac{\Delta y}{\Delta x}, \\
& + T_{i-1,j,2}(h'_{i-1,j} - h'_{i,j})\,\frac{\Delta x}{\Delta y}, \\
& = S_{i,j}\,\Delta x_j\,\Delta y_i\,(h_{i,j} - h\phi_{i,j})\,/\Delta t \\
& + Q_{i,j} \\
& + R_{i,j}\,(h_{i,j} - RD_{i,j}),
\end{aligned}
$$

where

$T_{i,j,1}$ = aquifer transmissibility between cell i,j and cell i,j+1;

$T_{i,j,2}$ = aquifer transmissibility between cell i,j and cell i+1,j;

$h'_{i,j}$ = average of the calculated hydraulic head at the end of the time step, $h_{i,j}$, and the hydraulic head at the beginning of the time step, $h\phi_{i,j}$;

$\Delta x'$ = distance in x-direction separating nodes;

Δx = cell dimension in x-direction;

$\Delta y'$ = distance in y-direction separating nodes;

Δy = cell dimension in y-direction;

$S_{i,j}$ = storage coefficient for cell i,j;

$h_{i,j}$ = hydraulic head for node i,j at end of time step;

$h\phi_{i,j}$ = hydraulic head for node i,j at beginning of time step;

Δt = size of time increment;

$Q_{i,j}$ = net rate of water withdrawal for cell i,j;

$R_{i,j}$ = slope of the spring-flow response line for cell i,j; and

$RD_{i,j}$ = minimum head for which spring flow occurs.

A more detailed discussion of the derivation of the finite difference equation is presented in the Board's program documentation and users manual entitled *GWSIM, Groundwater Simulation Program*.

The finite difference equation is written for each cell in the aquifer model. This results in a large system of simultaneous equations with the hydraulic head for each node, $h_{i,j}$, as the unknowns. This system of equations is solved by an iterative alternating direction implicit procedure which reduces the large system of equations into several small sets of systems of equations. One set of systems of equations is generated by assuming that each column in the finite difference grid is isolated so that only the hydraulic heads along the column are unknown. The second set of systems of equations is generated by assuming that each row is isolated and that only the head values along the row are unknown. Once the sets of systems of equations have been solved for the hydraulic head, one iteration of the solution procedure has been completed. The process is repeated until it has converged to a solution. The terms $h_{i,j}$ are the simulated heads at the end of the time step and they are used as the beginning heads for the following time step. For a more detailed discussion of the iterative alternating direction implicit procedure, see Peaceman and Rachford (1955) or Prickett and Lonnquist (1971).

For this model work, the solution procedure was considered converged when the total head changed from one iteration to the next is less than a specified value. During the model calibration study the convergence criterion was set equal to 15 feet. If all the nodes in the system changed uniformly, this error criterion represents a head change of approximately 0.02 foot per node.

Application to Edwards (Balcones Fault Zone) Aquifer

The Edwards (Balcones Fault Zone) aquifer was discretized into a finite difference grid containing 2,480 cells as illustrated in Figure 16. The grid contains 31 rows and 80 columns. The cell spacings are variable with the smallest spacing equal to 0.90 mile and the largest spacings equal to 5.0 miles. The smallest cell contains 1.2 square miles and the largest cell contains 18.5 square miles. As illustrated in Figure 16, only a portion of the finite difference grid actually overlies the aquifer. Only 856 of the 2,480 cells in the grid are considered as part of the Edwards system and take part in the simulation process.

The Edwards model contains three types of cells; outcrop, artesian, and boundary. The outcrop and artesian cells were defined by the water level at each of the nodes. The outcrop declaration was assigned to cells whose water level was below the top of the aquifer, and the artesian declaration was assigned to cells whose water level was above the top of the aquifer. The water levels as of January 1, 1947, were used in the initial declarations; however, the simulation program has the ability to change declarations if the simulated water levels dictate such during simulation. The boundary cells are considered exterior to the ground-water system and do not enter into the simulation.

As illustrated in Figure 3, the Edwards aquifer is cut by many fault trends which generally strike in the same direction. The finite difference grid was superimposed over the map shown in Figure 3 so that the rows of the grid would be aligned with the major fault trends. This alignment allows a convenient procedure for the representation of the faults. Since the simulation program allows directional permeability, the permeability may be reduced between two cells whose common face is to represent a fault.

The assignment of recharge to cells is an important step in model building. The recharge zone used in this model is that portion of the aquifer which is flagged as outcrop in Figure 16. These nodes exhibited water-table conditions for January 1947. For the Edwards aquifer, annual recharge data were obtained from the U.S. Geological Survey for the 22 subbasins which cross the recharge zone. For several of the subbasins, the recharge was divided into direct stream loss and directly infiltrated precipitation. If the data were no divided as mentioned above, approximately 80 percent of the subbasin recharge was assumed to be direct stream loss and the remainder of the recharge was assigned to the infiltrated precipitation category. In the initial assignment of recharge, the direct stream recharge was evenly distributed to the cells which contain stream reach. The remainder of the recharge was distributed evenly to the remainder of the recharge cells for that subbasin.

The second step in the assignment was a slight redistribution of the recharge in each subbasin so that the stream cells close to the artesian zone received a larger share of the recharge. The cells which were distant from the artesian zone had their assigned recharge reduced by 20 percent. This increment of recharge was then uniformly added to the recharge of the stream cells adjacent to the artesian zone.

As mentioned during the discussion of the finite difference equation, the response of spring flow was assumed to be linear with head. Figure 11 illustrates how well this assumption was substantitated for Comal Springs. This figure is a plot of the flow rate for the spring against the water level in an observation well located nearby. The equation for the spring flow response curve was developed from this figure. A similar graph and equation were developed for each major spring in the system.

The assignment of pumpage values to cells is a very critical step in model building. The pumpage must be assigned to cells so that the distribution of pumpage in the model approximates the distribution of pumpage which actually occurred.

For the verification stage, pumpage values were obtained from the U.S. Geological Survey and consisted of annual pumpage rates by well. Each of these wells was located on a map and the finite difference grid was superimposed over the map. The pumpage value assigned to each cell represents total pumpage of all wells located within the cell boundaries.

RESULTS OF MODEL OPERATION

Model Calibration Phase

The calibration phase of model development is concerned with the simulation of the aquifer for a time period when the response of the aquifer is known. Water levels are known at the start and end of the verification period along with the pumpage and recharge values for the same time period. A comparison of the observed and simulated water levels for the verification period is an indicator of how well the model is simulating the aquifer's response. If springs are an important feature of the aquifer, as they are for the Edwards, a comparison of observed and simulated spring flows also can be used as an indicator of how well the model simulated the aquifer.

For this study, the calibration period extended from 1947 through 1971. January water-level data were available for the years 1947, 1957, 1959, and 1972. These water levels allow several opportunities to gage the accuracy of the simulation results. This period was chosen because of the availability of data and because the water levels during this period showed a very large variance. During the years 1947 through 1956, the San Antonio region suffered a very severe drought. The years 1957 and 1958 were years of extremely high recharge, as indicated in Figure 12. As illustrated in Figure 14, the water levels for the representative wells showed large declines and recoveries during this time period (1947 through 1971). It was felt that if the simulated water levels could track the observed water levels during this period, the model would be well calibrated. The simulation period was continued through 1971 to allow the longest possible calibration period. Small calibration errors should become evident after this 25-year calibration period, and thus, could be identified and corrected.

The distribution of the simulation errors is an important indicator as to the validity of a model. The simulation error equals the simulated water level minus the measured water level. The mean error for January 1957 was 6.81 feet and the mean error for January 1959 was 2.85 feet with more than 75 percent of the simulation errors smaller than 25 feet. The mean error for January 1972 was 0.68 foot with more than 70 percent of the simulation errors smaller than 25 feet.

Figure 17 is a plot of the cumulative simulated and measured spring flows for the calibration period. It is important to note how well the simulated curve tracks the measured curve. At the end of the simulation, the simulated flow was less than the measured flow.

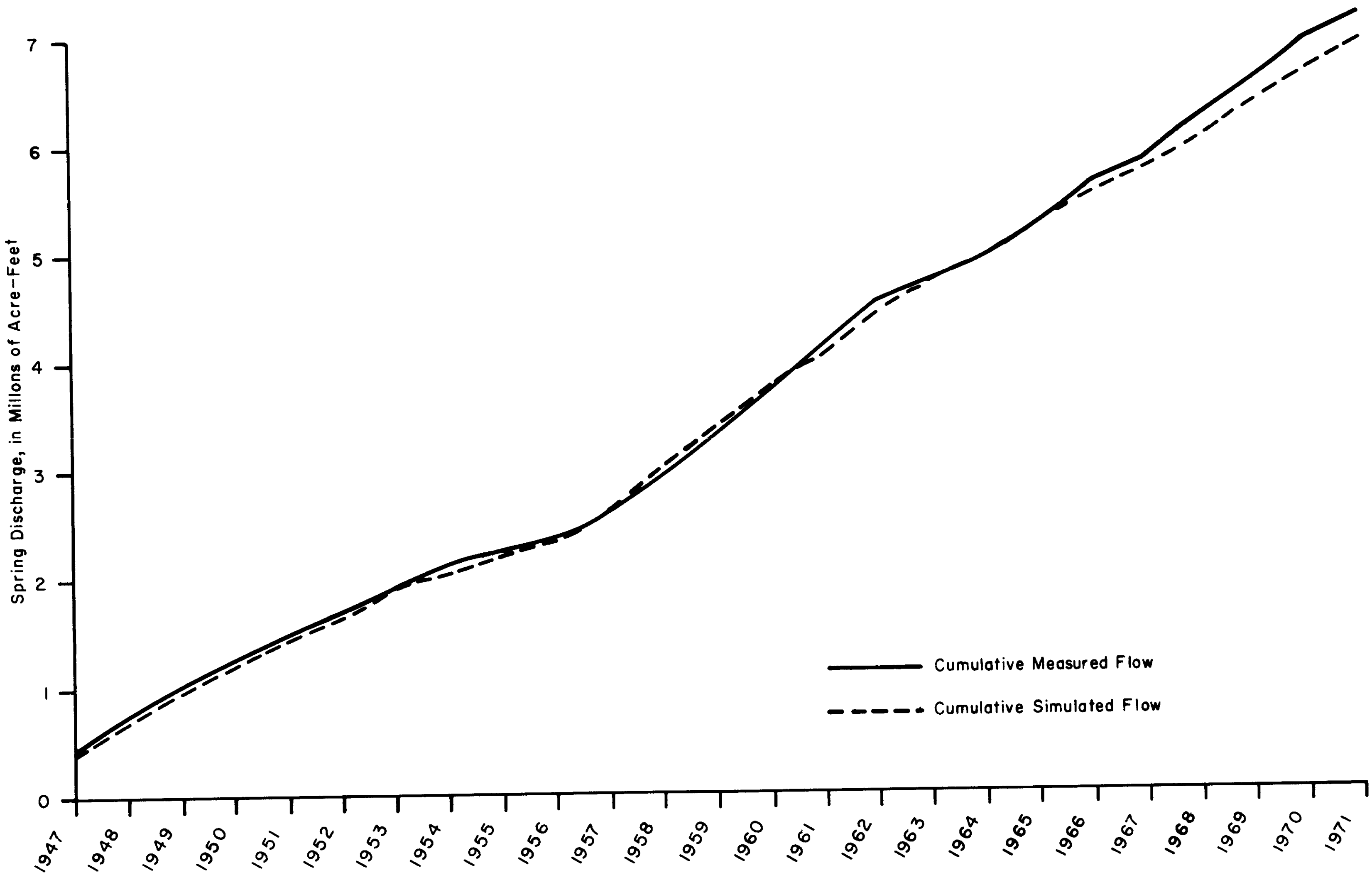

Figure 17

Historical and Simulated Sum of Spring Flows, 1947-71

However, the difference amounts to only 4.3 percent of the total flow. For the last year of simulation, the simulated spring flow for Comal Springs equaled 159,970 acre-feet and the reported flow was 159,182 acre-feet. The difference amounts to less than one-half of one percent.

Based on the above comparisons, it was decided that the digital model of the Edwards (Balcones Fault Zone) aquifer was calibrated to a degree of accuracy sufficient to reproduce past events and, consequently, that the model could be used to predict future aquifer conditions as a tool in evaluating management plans.

Future Simulation Phase

Several model applications were performed to simulate the aquifer response to projected pumpage and recharge rates. The aquifer response was indicated by the simulated water levels and spring flows. The simulation period began in 1972 and extended through 2049.

Projected Recharge

For these model applications, the basic sequence of projected recharge was based on a historical sequence of precipitation. A procedure was devised which correlates precipitation on the recharge zone with recharge. This procedure was calibrated so that for the period 1934-71, when recharge to the aquifer was known, the total projected recharge was equal to the total measured recharge.

After the projection procedure was calibrated, the measured precipitation for the period 1902 through 1950 was used to generate the recharge for the period 1972 through 2020. The recharge sequence was repeated after 2020 so that the recharge for 2021 equals the recharge of 1972, and the 2022 recharge equals the recharge projected for 1973. The recharge was generated for each year by drainage basin. Table 5 gives the projected recharge for selected years based on the distribution of recharge used during the model calibration phase of the study. Each basin's recharge was assigned to the cells comprising the basin. This procedure maintained the distribution of recharge used in the calibration of the model.

It was noted that this projected recharge sequence does not show the yearly fluctuation that the observed data show. This is primarily due to the generalizations made in correlating precipitation and recharge. However, the average of the projected sequence agrees with the average of the measured recharge. Since the model is to simulate end-of-year water levels far advanced into the future (50 years) this type of recharge sequence is appropriate.

The recharge sequence used in the future projections was based on the assumption that the hydrologic sequence of the past would occur in the future resulting in the repetition of the historical recharge sequence. During the future simulations, the water levels in the outcrop zone of the aquifer are different from the water levels which were present when the measured recharge occurred. It is possible that this change in water level could affect the amount of water entering the aquifer. Future water levels, which are lower than the historical levels, could allow more water to recharge the aquifer if given the same amount of water in the recharging source. It is believed that if this condition did occur, the effects on simulated water levels would be minimal.

Projected Withdrawals

The pumpage projections were determined for municipal and industrial, irrigation, and domestic and livestock needs for the period 1970-2020.

The municipal and industrial pumpage projections were made for cities by decade. The projected pumpage rates were assigned to the cells which contain the municipal wells. Time and space allowances were made for new wells and modifications to existing wells.

The domestic and livestock pumpage rates were projected for each county by decade. This demand was distributed uniformly to each active cell in the county. For the municipal and industrial and domestic and livestock pumpages, a straight-line interpolation procedure was used to determine pumpage values for nondecade years and a straight-line extrapolation procedure was used to determine pumpage values for the years following 2020.

The projection of irrigation pumpage was based on past pumpage history and on soils considered to be potentially irrigable. If the trend of 1958 through 1969 continues, all of the area which could be irrigated would be under irrigation by the year 2042. Since one of the ideas to be investigated was how the aquifer would respond under maximum irrigation pumpage and since this maximum would not occur until after 2020, the simulation period was extended through 2049 to include the period of maximum pumpage.

Table 5.—Projected Annual Recharge to the Edwards (Balcones Fault Zone) Aquifer, for Selected Years, 1975-2049

(Recharge is given in thousands of acre-feet by drainage basin.)

Year	Nueces and West Nueces River basins	Frio and Dry Frio River basins	Sabinal River basin	Area between Sabinal and Medina River basins	Medina River basin	Area between Cibolo Creek and Medina River basins	Cibolo and Dry Comal Creek basins	Blanco River basin and adjacent area	Total
1975	120.4	104.8	37.7	75.5	61.6	68.3	99.0	34.8	602.1
1980	69.7	60.7	21.8	43.7	35.8	39.7	57.6	21.1	350.1
1985	86.7	75.5	27.2	54.3	52.9	58.6	84.9	27.2	467.3
1990	94.4	81.3	29.6	59.2	46.5	51.5	74.7	29.7	466.9
1995	95.6	80.9	22.2	38.2	32.0	32.5	55.6	18.1	375.1
2000	108.4	96.1	35.8	66.8	51.9	52.6	89.8	29.3	530.7
2005	173.4	156.0	60.8	142.1	86.4	90.7	143.4	40.2	893.0
2010	91.9	84.7	36.5	85.5	59.6	64.2	94.9	38.6	555.9
2015	72.3	73.7	34.3	79.1	51.1	58.6	95.3	32.9	497.3
2020	72.9	63.9	24.6	60.5	36.4	39.6	72.7	24.6	395.2
2025	108.0	94.1	33.8	67.7	48.8	54.1	78.4	23.2	508.1
2030	80.7	70.3	25.3	50.6	39.6	43.8	63.5	22.4	396.2
2035	108.4	94.4	34.0	68.0	56.5	62.6	90.8	28.5	543.2
2040	80.1	73.2	11.5	74.9	51.6	60.3	91.4	29.9	472.9
2045	95.0	88.6	40.8	80.0	55.3	63.1	105.1	36.8	564.7
2049	108.4	96.1	35.8	66.8	51.9	52.6	89.8	29.3	530.7
Average	97.9	87.1	32.0	69.5	51.1	55.8	86.7	29.2	509.3

Table 6.—Projected Annual Pumpage From the Edwards (Balcones Fault Zone) Aquifer, for Selected Years, 1975-2049

(Pumpage is given in thousands of acre-feet.)

Year	Uvalde and eastern Kinney Counties	Medina County	Bexar County	Comal County	Hays County	Total
1975	90.3	23.0	228.2	11.2	5.8	358.5
1980	117.4	27.8	277.9	12.5	7.4	443.0
1985	128.3	30.1	284.7	12.7	8.2	464.0
1990	147.4	33.6	314.3	13.4	9.6	518.3
1995	170.1	39.5	376.8	14.7	12.2	613.3
2000	185.1	40.9	382.1	14.9	13.1	636.1
2005	188.4	41.7	384.0	15.0	14.0	643.1
2010	223.1	48.1	463.0	16.4	17.8	768.4
2015	242.1	51.8	505.6	17.1	20.8	837.4
2020	261.2	55.6	548.2	17.7	23.7	906.4
2025	280.1	59.1	575.3	18.5	24.8	957.8
2030	321.1	65.8	657.0	20.0	28.1	1,092.0
2035	318.1	66.2	637.7	20.0	27.6	1,069.6
2040	337.0	69.8	672.4	20.7	29.5	1,129.4
2045	352.9	75.8	765.0	22.3	33.7	1,249.7
2049	345.2	71.9	734.7	22.0	32.9	1,206.7
Average	231.8	50.1	487.9	16.8	19.3	805.9

The irrigation pumpage was projected to increase only in Uvalde and Medina Counties. Based on the 1970 irrigation pumpage figures, the annual Uvalde County increase was projected to be 5.51 percent and the annual Medina County increase was 3.88 percent.

The municipal and industrial water requirements were projected for three rainfall patterns; low, median, and high. The water requirements assuming low rainfall were larger than were the projections assuming median rainfall, and the high rainfall water requirements were less than the median rainfall requirements. It was assumed that the pumpage values associated with the other two pumpage categories, domestic and livestock and irrigation, would show the same type of fluctuations. That is, for high rainfall conditions, the demands would decrease and for low rainfall, the demands would increase. The rainfall data used to project recharge were also used in adjusting the pumpage demands placed upon the aquifer. A listing of the projected pumpage is shown in Table 6.

All projected withdrawals assumed that the present water quality of the aquifer would remain constant in time. Possible changes in water quality were not considered in sizing or in locating pumpage centers.

Figure 18 gives a graphical representation of how the pumpage is projected to increase with time. The most noticeable area of increase is in the San Antonio area. The irrigation pumpage increase in the western portion of the aquifer is also quite apparent. The pumpage centers representing New Braunfels and San Marcos are discernible in the eastern portion of the aquifer.

Application Results

Aquifer Simulations Using Projected Conditions

Simulation Run One used the basic projections of recharge and pumpage rates for the period 1972-2049. The simulation period extended through the time when irrigation pumpage would have reached its maxirnum value. Figure 19 illustrates the resulting change in water levels through the year 2020. It is important to note that the entire central zone of the aquifer has been subjected to drastic water-level declines.

Figure 20 gives data which could be used to evaluate the model application. This figure shows the total recharge and pumpage for each year of the simulation. The three categories of pumpage are shown for each year with a table listing the amounts for each category for the years 2020 and 2049. The figure also illustrates the simulated flow for the two major springs along with the water levels for the area adjacent to each spring. The water level for a node located approximately on the Uvalde-Medina County line is shown to illustrate the water level in the irrigation area. A node located in the city of San Antonio is listed to illustrate, the water level in the area of high municipal and industrial pumpage.

The simulated spring flows for Comal Springs for the years 1987 and 1988 were less than the recorded minimum flow. This indicates that, for these 2 years, the spring flow would be intermittent. The last year for which any spring flow for Comal Springs was simulated is 1994. The water level adjacent to Comal Springs experienced a downward trend, once the springs ceased to flow. This is expected since spring flow from the Edwards is assumed to be equivalent to spillage from a reservoir.

The simulated flows from San Marcos Springs show a declining trend through the last year for which spring flow was simulated, 2009. The first year for which the spring flow was less than the minimum reported flow is simulation year 2000. The water levels adjacent to the springs show a downward trend similar to the trend observed for the water levels adjacent to Comal Springs.

The water levels for the irrigation area show a steady decline. The water levels for years 2020 and 2049 are listed in Figure 20. For the year 2020, approximately 650 feet of artesian head remains in the irrigation area. At the end of the simulation period (2049), approximately 390 feet of artesian head would remain above the top of the aquifer.

The water levels for the San Antonio area show a steady decline as would be expected with the increased pumpage. For the year 2020, approximately 250 feet of artesian head would remain and for the year 2048, the node changed from the artesian condition to the water-table condition. For the year 2049, the saturated aquifer thickness for the node equaled approximately 460 feet.

The results obtained from Simulation Run One indicate that, using the projected sequences of pumpage and recharge, the springs will cease to flow; however, all pumpage demands on the aquifer could be met through the year 2049.

Simulation Run Two was performed to determine how artificial recharge could affect the aquifer. The additional recharge would come from newly established and proposed reservoirs designed to increase recharge to the Edwards. The total recharge increase would amount to 80,400 acre-feet per year which is the sum of the average annual increases in recharge attributable to each reservoir. For each year of the simulation, the recharge for each appropriate basin was adjusted to reflect the artificial recharge. The pumpage sequence is identical to the sequence used for Simulation Run One. Figure 21 shows the recharge and pumpage sequences along with other pertinent data for this model application. The springs continue to flow for a longer period of time with the addition of the artificial recharge. The simulation for Comal Springs does not indicate the intermittent flows for the years 1987 and 1988 as shown in Simulation Run One. The San Marcos Springs simulation indicates that the last year of flow would be in year 2015. The 2020 water levels for the irrigation area and for the San Antonio area indicate rises of 55 and 38 feet, respectively. This represents approximately 15 percent reduction in pumpage lift for these areas. The addition of this artificial recharge had a measurable effect on the spring flows, but the water availability from the aquifer was not increased appreciably.

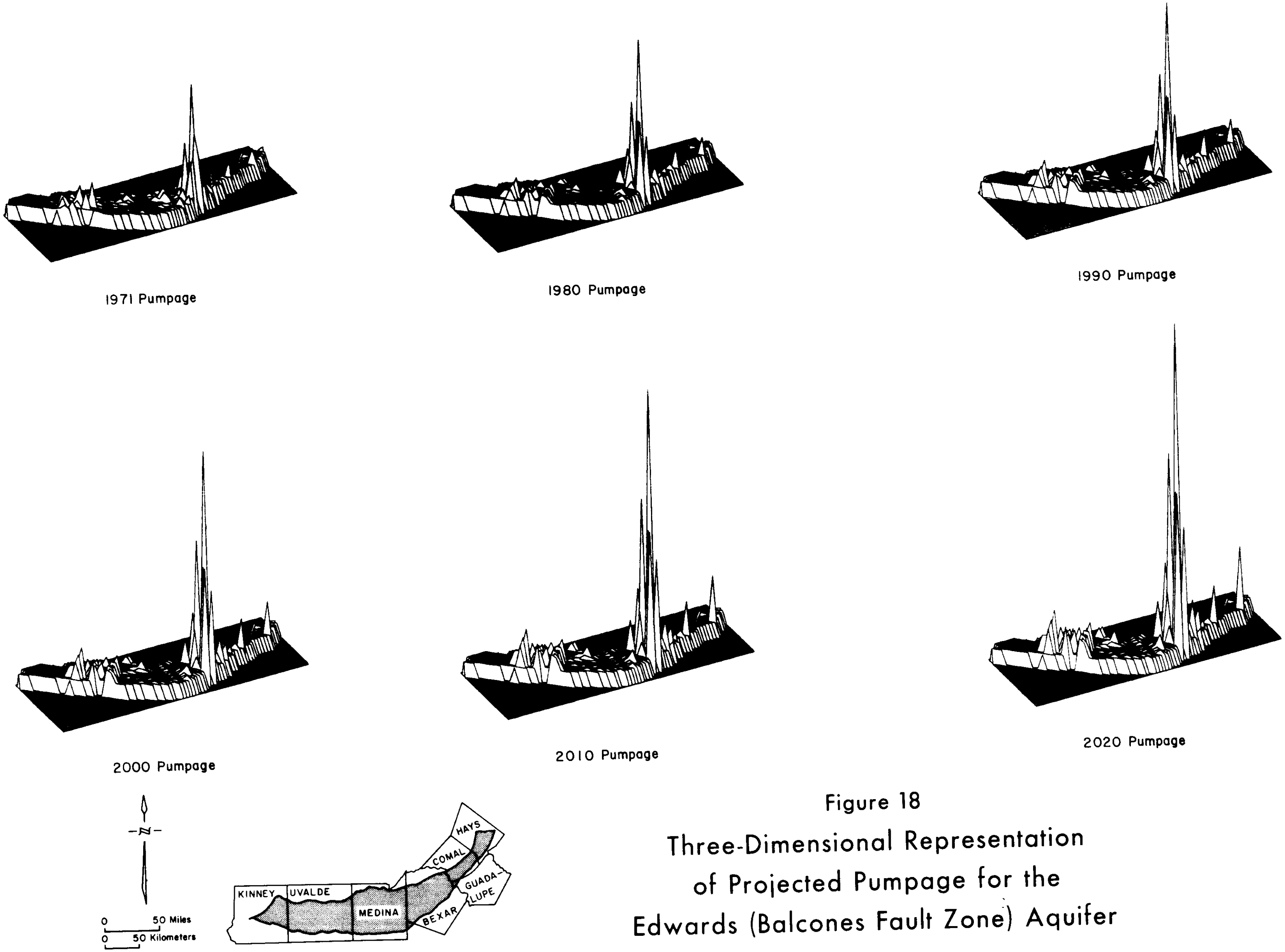

Figure 18

Three-Dimensional Representation of Projected Pumpage for the Edwards (Balcones Fault Zone) Aquifer

Simulation Run Three was performed to determine the future water levels and spring flows if the irrigation pumpage increased at only one-fourth of the projected trend. Since the irrigation pumpage would not reach its maximum until well into the future using this one-fourth projected trend, the simulation period stopped with year 2020. As shown in Figure 22, the irrigation pumpage for the year 2020 equals only one-half the value for that year under the full projected trend. As compared with the results of Simulation Run One, the 2020 water levels show a significant rise. The largest change occurred for the irrigation area of the aquifer. The spring-flow simulation indicates that Comal Springs would flow approximately 1 year longer and San Marcos Springs would flow for an additional 8 years. By comparing this run with Simulation Run One, it is apparent that this change in the irrigation pumpage sequence would have a significant effect on the aquifer.

Simulation Run Four was performed to simulate the aquifer's response through the year 2020 if the irrigation pumpage remained constant at the 1970 level. Figure 23 illustrates some of the pertinent data obtained from this run. As compared to run three, the water levels are higher and the springs continue to flow for a longer period of time. The lack of response of simulated spring flows to changes in irrigation pumpage indicates that the other two categories of pumpage, municipal and industrial and domestic and livestock, have the major influence on spring flow.

Aquifer Simulations Using Drought and Flood Conditions

Simulation Runs Five through Nine were performed to determine the effects that a 12-year drought and flood sequence of recharge events would have on the aquifer. The recorded recharge for the period 1947-1958 was used as the drought and flood sequence. As illustrated in Figure 12, the first 10 years of the sequence show an overall reduction in recharge. Recharge for the year 1956 was approximately 44,000 acre-feet, which is about 8 percent of the average annual recharge. This drastic reduction in recharge indicates a severe drought.

The recharge for the years 1957 and 1958 was approximately 1,143,000 acre-feet and 1,711,000 acre-feet, respectively, representing 212 percent and 319 percent of the average annual recharge. These values indicate that these were 2 years of abnormally high recharge. This 12-year sequence of recharge follows the general trend in Texas of a drought belng broken by a period of relatively high rainfall.

The 12-year recharge sequence was superimposed at various times in the sequence of projected recharge. Simulation Run Five simulated the aquifer if the drought and flood sequence occurred during the years 1972-1983. The height of the drought would occur in simulation year 1981. Figure 24 illustrates some of the results of this simulation. It is important to note that the simulation of Comal Springs indicate intermittent flow for 1 year followed by 2 years of no flow. Any simulated spring flow which is less than the reported flow for the year 1956 was considered to be intermittent since the spring did not flow during the summer of 1956. Comal Springs began to flow during the years of high rechage, but the springs ceased to flow in the year 1995.

The hydrographs illustrate a dip in water levels during the drought period, as expected. The 2 years of high recharge refilled the aquifer, and the water levels at the end of simulation were basically unchanged from the basic simulation values. This indicates that once the drought and flood period is passed, the residual effects of the sequence are minimal.

Additional runs were performed with the drought and flood sequence ending in 1990, 2000, 2010, and 2020. As would be expected, as the pumpage becomes larger with time, the effects of the drought are more pronounced. Simulation Run Nine simulated the aquifer response to a drought and flood sequence ending in 2020. The pumpage during the last year of this drought (2018) equals 950,000 acre-feet or approximately twice the pumping rate for the last year of the drought for Simulation Run Five (1981). The hydrographs from Simulation Run Nine, Figure 25, illustrate a much deeper dip than do the hydrographs from Simulation Run Five, Figure 24. The spring-flow volumes were sharply reduced during the drought periods.

Aquifer Simulations Under Management Plans

Several model applications (Simulation Runs Ten through Seventeen) were performed to illustrate how the model could be used as a tool in evaluating the effects a management plan would have on the aquifer. The model was used to simulate the aquifer under a given set of external stimuli, such as pumpage and recharge, which were developed according to the management plan to be evaluated. That is, once the management plan alternative to be studied is developed, its effects on the pumpage and recharge sequence are determined, and the aquifer response to the pumpage and recharge is simulated. By comparing the aquifer response to various alternatives, the best alternative could be selected.

The management plans to be investigated were (1) maintain San Marcos Springs flow rate of 100 cubic feet per second (ft^3/s) by reducing pumpage and (2) maintain Comal Springs flow rate of 100 ft^3/s by reducing pumpage. Both management plans were to be evaluated assuming the projected sequence of recharge occurs. One hundred ft^3/s represents an annual flow of 72,397 acre-feet, which is approximately 50 percent greater than the minimum recorded yearly flow for San Marcos Springs, but is 25,000 acre-feet less than the average annual flow.

Maintain San Marcos Springs

The purpose of Simulation Run Ten was to determine whether San Marcos Springs would continue to flow at the rate of 100 ft^3/s (72,397 acre-feet per year) if the municipal and industrial pumpage was reduced so that the total pumpage from the aquifer is limited to 540,000 acre-feet per year. Referring to Simulation Run One, 540,000 acre-feet of water was pumped for the year 1994 when the San Marcos Springs flow rate dropped below 72,397 acre-feet per year. The pumpage values and some of the simulation results are shown in Figure 26. The first year of reduction was year 1994 and the reduction for the year 2020 amounted to 366,000 acre-feet. This implies that 68 percent of the projected full development demand would have to be supplied from other sources.

The 2020 simulated spring flow for San Marcos Springs equaled 69,400 acre-feet per year (96 ft^3/s) which is sufficiently close to say that if the total pumpage was restricted to 540,000 acre-feet per year by the reduction of municipal and industrial pumpage, San Marcos Springs would continue to flow at the plan value through the year 2020. No work was done on maintaining the spring flow under a severe drought sequence. The total pumpage plus spring flows is in excess of the average recharge to the aquifer. As shown in Figure 26, the declining water levels for the irrigation area and the San Antonio area indicate that water is continually being removed from storage. When comparing the results of Simulation Runs One and Ten, the 2020 water level for the irrigation area shows a rise with the decreased municipal and industrial pumpage. This indicates that the municipal and industrial pumpage, which is concentrated in the eastern portion of the aquifer, is removing water from the western portion of the aquifer.

Another analysis, Simulation Run Eleven, was performed to determine if the management plan could be accomplished by constraining the irrigation pumpage. For the last three years of the simulation period, the total municipal and industrial and domestic and livestock pumpage exceeded the 540,000 acre-feet per year limit resulting in zero irrigation pumpage. The 2020 spring flow for San Marcos Springs equaled 18,000 acre-feet which is significantly less than the plan value. Using a maximum pumpage limit of 540,000 acre-feet per year and restricting only irrigation pumpage, this simulation indicates that the planned flow rate for San Marcos Springs would not be met.

The investigation by Simulation Run Twelve was to see if the plan objective would be accomplished by limiting the total pumpage to 540,000 acre-feet per year by the joint reduction of municipal and industrial and irrigation pumpage. When a reduction in total pumpage is required, both categories of pumpage would be reduced by the same percentage. As illustrated in Figure 27, San Marcos Springs continued to flow through the year 2020 with the last year's flow equal to 51,800 acre-feet. This flow represents more than 70 percent of the desired flow. The results of this simulation run indicate that the limitation of municipal and industrial and irrigation pumpage so that total pumpage does not exceed 540,000 acre-feet per year would result in significant flow for San Marcos Springs.

Two additional simulation runs (Simulation Runs Thirteen and Fourteen) were performed to determine the spring flow if the municipal and industrial and irrigation pumpage were restricted so that the total annual pumpage would not exceed 600,000 acre-feet and 650,000 acre-feet. Using 600,000 acre-feet per year maximum pumpage, the 2020 spring flow from San Marcos Springs was 22,300 acre-feet and the 2020 flow under the 650,000 acre-feet plan limit was only 1,800 acre-feet. These simulation runs indicate that, using joint reduction of municipal and industrial and irrigation pumpage, the maximum pumpage rate which would allow some flow for San Marcos Springs through the year 2020 is between 600,000 and 650,000 acre-feet per year.

Maintain Comal Springs

Several model runs were performed to evaluate the alternative methods of reducing pumpage to allow Comal Springs to flow. Simulation Run Fifteen was performed to determine whether Comal Springs would continue to flow at the rate of 100 ft^3/s through the year 2020 if the total pumpage was limited to 450,000 acre-feet per year by the reduction of municipal and industrial pumpage. Referring to Simulation Run One, 450,000 acre-feet of water was pumped for the year 1984 when the flow for Comal Springs dropped below the 72,397 acre-feet level. The first year of pumpage reduction is 1984, and the 2020 municipal and industrial pumpage reduction equaled 85 percent of the basic projected demands.

As shown in Figure 28, Simulation Run Fifteen indicates that Comal Springs continued to flow through the simulation period. The flow for the last year (2020) was 69,300 acre-feet which is very close to the plan value. It is important to note that the reduction in pumpage to maintain Comal Springs also resulted in continuous flow for San Marcos Springs. As previously stated, the total discharge from the aquifer exceeds the recharge, resulting in the mining of water from the aquifer. The continuing decline in water levels confirms this. The declining water levels indicate that a maximum pumpage rate of 450,000 acre-feet per year would not allow the springs to flow indefinitely. However, this simulation run shows that the 450,000 acre-feet per year maximum would satisfy the plan objective of 100 cubic feet per second flow for Comal Springs through the year 2020.

Simulation Run Sixteen was performed to determine whether the plan objective would be met if the pumpage was limited to 450,000 acre-feet per year by reducing irrigation pumpage. For the year 2007, the sum of municipal and industrial and domestic and livestock pumpage exceeds 450,000 acre-feet per year, so the irrigation pumpage is reduced to zero. For this same year, the flow for Comal Springs was considered to be intermittent. It appears that the location and size of the municipal and industrial demand are such that the pumpage for these purposes is the controlling category for the preservation of Comal Springs. These results indicate that the management alternative of the unilateral restriction of irrigation pumpage would not result in continuous flow for Comal Springs through the year 2020.

Simulation Run Seventeen was performed to determine whether Comal Springs would continue to flow at the rate of 100 ft^3/s through the year 2020 if the total pumpage was limited to 450,000 acre-feet per year by the joint reduction of municipal and industrial and irrigation pumpage. When a reduction in total pumpage is required, both categories of pumpage would be reduced by the same percentage. Figure 29 shows the pumpage values and some of the simulation results from this run. Comal Springs continued to flow during the simulation but the rate of flow was less than desired. The total flow during the last year of simulation (2020) was approximately 42,000 acre-feet or 58 percent of the plan value. This indicates that the maintenance of a 450,000 acre-feet per year pumpage rate would result in some flow for Comal Springs through the year 2020.

Aquifer Simulations Using Augmentation Pumpage

One alternative to aquifer-wide management for the preservation of spring flow is the pumpage of water to augment natural flow. Instead of allowing water to spill from the aquifer at the springs, water could be artificially removed from the aquifer and released immediately downstream from the spring. This would maintain the flow of water but would not restrict the water levels in the aquifer. It was assumed that once the spring-flow rate dropped below the 100 ft^3/s level, water would be pumped at the rate of 100 ft^3/s.

Simulation Run Eighteen was performed to evaluate the effect of pumpage for augmentation of San Marcos Springs. The first year of pumpage was year 1994. The results of this run indicate that, in the vicinity of the springs, the aquifer could yield the required water through the year 2020. As would be expected with the increased pumpage, the water levels in the aquifer are lower than the water levels of Simulation Run One.

Simulation Run Nineteen simulated the aquifer response to augmentation pumpage in the Comal Springs area. This pumpage began in 1986 and the water levels indicated that the aquifer could yield the necessary water through the year 2020. During Simulation Run Twenty, augmentation pumpage began in 1986 for both San Marcos and Comal Springs. Figure 30 indicates some of the results of Simulation Run Twenty. As compared to Simulation Run One, the additional drawdown for the San Antonio area and for the irrigation area indicates that a large amount of the water being pumped was coming from the central and western portions of the aquifer. These simulations indicate that there is sufficient storage in the aquifer, and the transmissibility in the spring areas will allow the additional pumpage through the year 2020.

The digital model of the Edwards (Balcones Fault Zone) aquifer could be used to make other simulation runs as new data become available or as additional management alternatives arise. Also, the model could be used to simulate the aquifer using a different recharge sequence. Updated pumpage estimates could be implemented into future simulations, and the effects of other management alternatives could be evaluated.

LIMITATIONS AND RECOMMENDATIONS

The ground-water model developed during this study is based on the assumption that the continuous aquifer may be divided into many discrete elements, called cells. The model simulates a water level in the center of each cell based on the value of the hydraulic parameters of the cell and of all other cells in the aquifer. Since each cell represents a large Land area, the value for each hydraulic parameter must represent the average or composite value of the hydraulic coefficients for the entire area. The pumpage and recharge are assumed to be spread uniformly across the cell. There are no point sources (recharge wells) or point sinks (pumping wells) in the model. Each square foot of the cell is assumed to have its portion of pumpage and recharge. These facts require that the water level simulated by the model be considered as the representative value for the water level for the entire cell. Therefore, one limitation to this model is that the simulated water levels represent regional values and do not represent the water level in a producing well. This limitation in no way restricts the use of the model in evaluating the long-term effects of pumpage and recharge on the aquifer.

One-year time steps were used in the model applications. This length of time was used because the data for pumpage and recharge were based on one-year time steps. This means that only end of year values for water levels are available from the simulations and, therefore, is a limitation of the model in that the seasonal variations in pumpage, recharge, spring flow, and water levels do not appear. It is possible for the model to simulate a spring-flow total for a year, but during a portion of the year, the spring may have ceased to flow.

A study of water quality in the aquifer was not one of the objectives of this project, but certain assumptions had to be made concerning water quality. The first assumption made was that the “bad-water” line could be treated as an impermeable aquifer boundary. The low transmissibility below this line makes this a good assumption if the gradient across this boundary is small. It is possible that during future sirnulations when the drawdowns in the artesian zone are excessive, some water may be transmitted across the barrier, but it is believed that the simulated water levels would not be greatly affected.

The second assumption made concerning water quality is that the spatial distribution present in 1972 would not change. All pumpage assignments were made according to this water-quality distribution. In particular, no modifications of the pumpage pattern due to encroachment of water of unacceptable quality from below the “bad-water” line were made because any encroachment of poor quality water would be minor and would cause only slight modifications in the projected pumpage distributions.

Sound ground-water resources development and management decisions concerning the Edwards (Balcones Fault Zone) aquifer must be based on the geohydrology, water demands, and the aquifer response to many alternative plans of operation. The high-speed digital model of the Edwards aquifer is able to store voluminous complex hydrologic data and rapidly analyze many alternative management plans at a reasonable cost. The use of this model by managers, hydrologists, and others to predict the aquifer response to alternative development patterns and pumpage rates will aid in the selection of the best management or development plan.

The collection of basic hydrologic data pertaining to the Edwards (Balcones Fault Zone) aquifer should be continued and expanded in order to better define the following aquifer parameters: (a) certain physical limits, such as ground-water divides, “bad-water” line, and updip limit of the aquifer; (b) movement and occurrence of ground waters; and (c) complex structural and lithologic composition. A study of water quality should be made to include: (a) determination of the conditions that would allow the transmission of poor quality water across the “bad-water” line barrier into the aquifer; (b) evaluation of current solutioning of the aquifer with respect to saturation of calcite and dolomite; and (c) determination of coefficients necessary to model regional water-quality changes in the aquifer. Many of the details presented in this report eventually will be revised due to the acquisition of additional basic data and because of a better understanding of the Edwards aquifer. However, it is believed that this work provides the foundation for future refinement and revision.

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