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Hydrogeology and groundwater modeling study of the Azua Valley, Dominican Republic

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HYDROGEOLOGY AND GROUND-WATER MODELING STUDY OF THE AZUA VALLEY, DOMINICAN REPUBLIC

by

Odalis Pérez Pérez

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1 9 8 9

STATEMENT BY THE AUTHOR

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This thesis is dedicated to my wife

Nadia S. Batista de Perez

and my sons

Erik, Edwin and Edgar

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ABSTRACT

A two-dimensional finite-difference ground-water flow model has been developed for a portion of the heavily irrigated Azua Aquifer, Dominican Republic. The predictive capabilities of the model are demonstrated by matching measured hydraulic heads for the 1965 steady-state situation and transient simulations for 1971, 1977 and 1988. The behavior of the aquifer and the effects on hydraulic heads derived from the application of seven different pumping scenarios are analyzed.

The model shows the effects of an extensive drainage network on the high ground-water levels which prevailed from 1983 to 1988. A sensitivity analysis also shows the zones of the aquifer which require development of new pumpage in order to overcome the drainage problem in areas still flooded by uncontrolled artesian flow.

The results of the model can be used for enhancing the integrated management of the water resources of the Azua Valley.

CHAPTER 1

INTRODUCTION

The Azua Valley is one of the most important centers of agricultural production in southwestern part of the Dominican Republic. During the last four decades, the Azua Valley together with the San Juan Valley and the Barahona Plains, has been a reliable food producer and a permanent support to the socioeconomic development of the region. By sharing the waters of the Yaque del Sur River basin with those of the San Juan and Barahona, Azua is one of the southwestern regions of the country where investments for developing irrigated agriculture and water resources have been largest. Despite these large investments, little has been done in terms of water-resources planning and management.

Before 1978, irrigation and water supply to Azua city, the largest city of the area, were mainly dependent on ground water. For over 30 years the sustained agricultural growth severely stressed the ground-water resources to a point where alternate sources of water were needed.

Due to expressed concerns of governmental organizations and local water users, a project was proposed which would share surface water from de Sabana Yegua dam with the Barahona-Neyba area. In the fall of 1978 the Yaque del Sur-Azua canal (YSURA) started operation, bringing water from the dam to the valley at a rate of 5-8 cubic meters per second (155-250 MCM/year).

Both stages, before 1978, characterized by intensive and uncontrolled pumpage of ground water (specially between 1975 and 1977), and after 1978, characterized by minimum use of ground water and excessive use of surface water, are of great importance for understanding the behavior of the Azua aquifer and for planning the joint use and management of both resources. During the former period, when most of the wells were constructed, the ground-water levels declined to levels of concern in the whole Azua aquifer, dropping by 20-30 meters in the most heavily pumped area. Consequently, water quality and yield of agricultural lands in the lowest parts of the valley started to deteriorate. During the latter period, fast recovery of the water levels in the aquifer, accelerated by unusual amounts of recharge associated with hurricanes and tropical storms, has caused flooding by uncontrolled flowing wells and serious drainage problems in the lowlands of the valley. The problem now is one of water excess, rather than water scarcity.

In order to alleviate the drainage situation, an extensive network of drainage canals and underground drains has been installed and an on-farm-water-management project is being developed. Although the drainage system is doing its work, still large amounts of surface water are being diverted to the valley and minimum amounts of ground water are being pumped. Nevertheless, the necessity of incorporating ground-water pumpage as a complementary solution to the drainage problem and as an additional resource for satisfying the water requirements of the region, has been recognized. Thus, the first attempt toward the joint use and management of surface water and ground water in the Azua Valley is, possibly, going to start soon.

Purpose and Scope of Investigation

The purpose of this investigation is to develop a ground-water flow model for analyzing the behavior of a portion of the Azua aquifer and estimating the range of the effects on the aquifer derived from the application of proposed and assumed pumping schemes. Another primary objective was to evaluate the sensitivity of water levels in the areas with critical drainage problems to the areal distribution of pumpage and to determine the location of preferential pumping sites for aiding to the solution of the problem. In order to accomplish the first objective, a ground-water flow model, based on MODFLOW, the Three-Dimensional Ground-Water Flow Model of The United States Geological Survey (USGS), is to be developed. The second objective is to be accomplished by using the developed model, once its predictive capabilities are proven, by assuming different pumping schemes derived from actual plans for ground-water extraction. Inasmuch as the application of ground-water modeling has had a very limited development in the Dominican Republic, the model to be developed in this study has a pioneer importance for future applications. It is intended to serve as both an aid to managers for planning the integrated use of the water resources of the Azua Valley and a new tool to be used by the water resources authorities of the Dominican Republic in other areas of the country.

In the short span, the study will contribute to identify those areas where new pumpage should be developed in order to lower ground-water levels in the lowlands and, consequently, aiding with the solution of the drainage problem. On the other hand, the study should serve as a guidance for future investigators regarding the critical areas and field data that are more important for enhancing calibration thus improving the predictive capabilities of the model.

This report has been divided in five chapters. The first chapter contains a general introduction, a description of objectives and a summary of previous studies. The second chapter contains a physical description of the Azua region, including geography, climate, geology and hydrogeology. The third chapter deals with the application of the U.S.G.S. finite-difference ground-water flow model to simulate actual conditions and the effects of future pumpage scenarios on the Azua aquifer. The fourth chapter includes a sensitivity analysis of the aquifer parameters and of the reaction of water levels in the areas with critical drainage problems to the areal location of pumpage. Finally, chapter 5 includes a discussion of the results and the conclusions and recommendations derived from the applications of the model.

Previous Investigations

Several studies have been undertaken on the hydrology and hydrogeology of the Azua Valley. The first study (Gilboa, 1965) was a preliminary evaluation of the conditions of ground water in the valley and produced the first measurements of ground-water levels known to the author. In 1971, the firm Tahal Consulting Engineers and the Instituto Agrario Dominicano (IAD) published a report that contained a thorough evaluation of the general hydrological conditions of the valley. The report included the contour maps of ground-water elevations and most of the data that were used for this thesis. In 1983, the Instituto Nacional de Recursos Hidráulicos (INDRHI) and Tahal delineated a master plan for the development of the ground-water resources of the Dominican Republic. Concerning the Azua Valley, the network of observation wells was reorganized and a new contour map was produced. One year latter, INDRHI and the firm SNC worked on the feasibility study and final designs of the service area of the Sabaneta Dam, the source of surface water to Azua. Information on pumpage and wells constructed after 1971 was generated and a program of aquifer tests was proposed. An electrical resistivity survey, whose results were published in 1967, served as a basis for most of the hydrogeologic studies. Only a map of lines of equal resistivity was available to the author. The rest of the report and other studies were unavailable. In addition, three papers dealing with some hydrogeological features of the artesian zone of the valley and the effects of extraordinary precipitation events were published by INDRHI. In spite of the various studies mentioned above, no attempts have been made to develop a computer model to assess the effects of the applied and planned pumping policies on the behavior of the water levels.

CHAPTER 2

THE STUDY AREA

Geography

The Azua Valley is an alluvial plain within the Yaque del Sur River Irrigation District, southwestern region of the Dominican Republic (Figure 1). The valley covers an area of approximately 400 square kilometers (156 square miles) that trends north to south with an average slope of 10 m/Km (52.8 ft/mi.).

The basin is bounded on the east and north by the Ocoa Mountain range, on the west by the Martin Garcia Mountain Range and on the south by the Caribbean Sea and the La Vigia Hill. The Ocoa and Martin Garcia Mountains range from 1000 m to 1200 m in altitude whereas the mean elevation of the La Vigia Hill is about 300 m. The major surface drainages are the intermittent streams Jura, Via and Tabara (Figure 1). In average, these rivers are dry during 300 days each year. Conversely, exceptional precipitations and streamflows, associated with cyclonic periods, have caused flash flooding and considerable damages to agriculture.

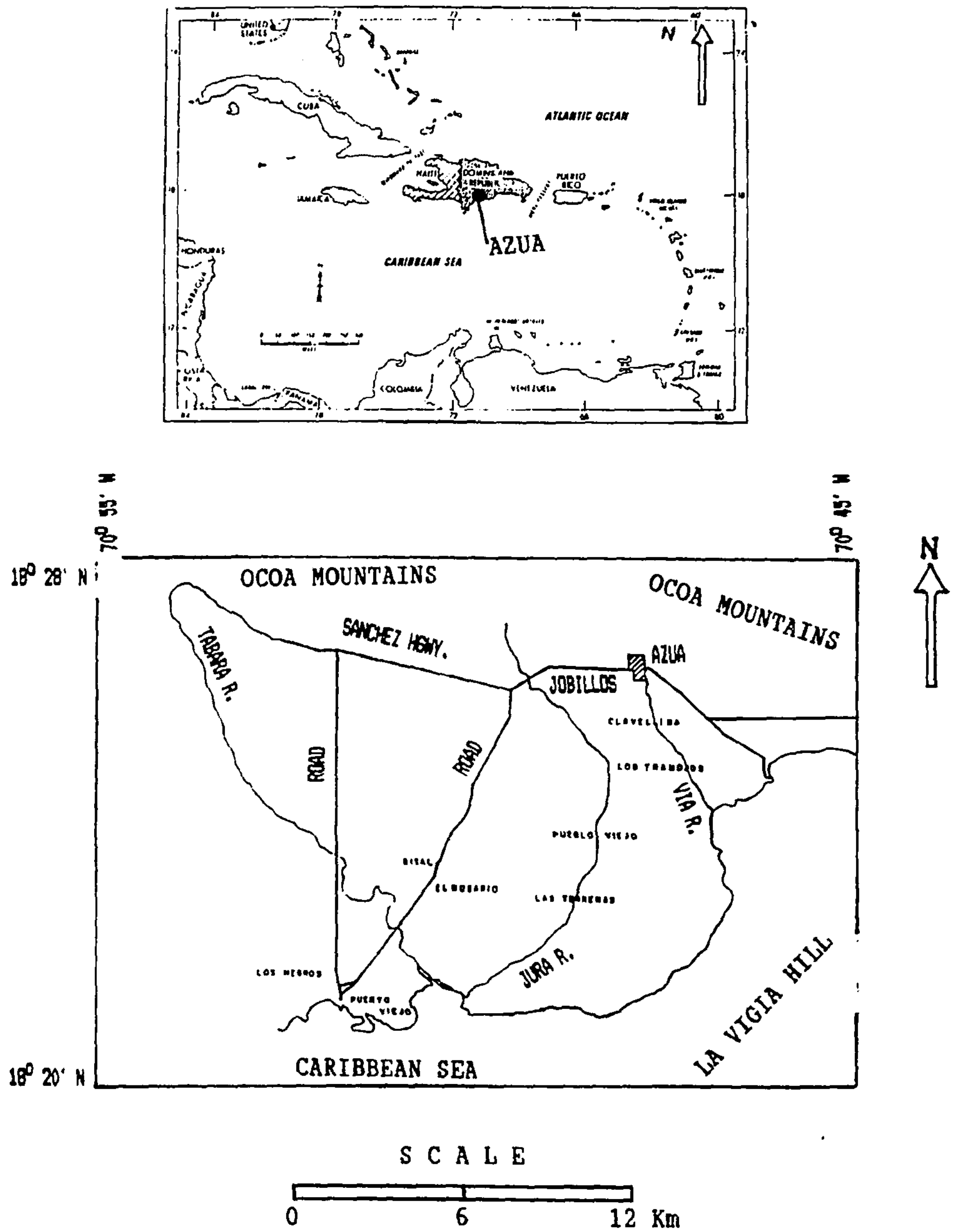

Figure 1 Location of the study area in Azua, Dominican Republic.

The major city within the valley is Azua, the capital of the Azua Province, whose population for 1988 has been estimated in about 20 thousands. Among other towns and villages within the study area the most important are Los Jobillos, Pueblo Viejo, El Sisal, Las Clavellinas y Los Negros.

Climate

The climate of the Azua area is semi-arid with a mean annual temperature of about 27 °C (81 °F) at Azua. During the year, monthly average temperatures are uniform and reach a maximum of 30 °C (86 °F) in July and a minimum of 25 °C (77 °F) in January. The mean annual precipitation recorded around the Azua Valley is about 670 mm (26 in) that falls during 56 days. However, an annual average of 1100 mm (43 inches) falls in the mountains at Peralta at an elevation of 1200 m (3900 ft). For 50 years of records, precipitation has varied from a minimum of 242 mm (9.5 inches) in 1957 to 1700 mm (67 inches) in 1979. Most of the precipitation measured at lower elevations falls from May to June and from August to October, usually in the form of convective showers and storms of short duration, during the former period, and associated with hurricanes and tropical storms during the latter period.

Annual potential evapotranspiration in the valley greatly exceeds the annual precipitation; the average class A pan evaporation measured at El Sisal is about 2900 mm (117 inches) and the annual average relative humidity is about 73%.

Geology

As the result of the work done by Tahal (1971), four major stratigraphic units were identified in and around the Azua valley. The units are (Figure 2):

(a) Eocene-Oligocene-Miocene, mainly limestones, sandstone and clayey shales located to the north, west, and southeast of the valley. This unit is designated as EO on the map (Figure 2). The unit is not well known and because of the large number of impermeable components and the complexity of the geologic structure it does not form a continuous aquifer.

(b) Old alluvial deposits of gravel, sand, conglomerate, and clay. Designated as P on the map. Outcrops are at the piedmont of the Ocoa and Martin Garcia mountain ranges and rest on unit EO. This Pleistocene unit probably grades into the alluvial fans of the valley (Unit AF).

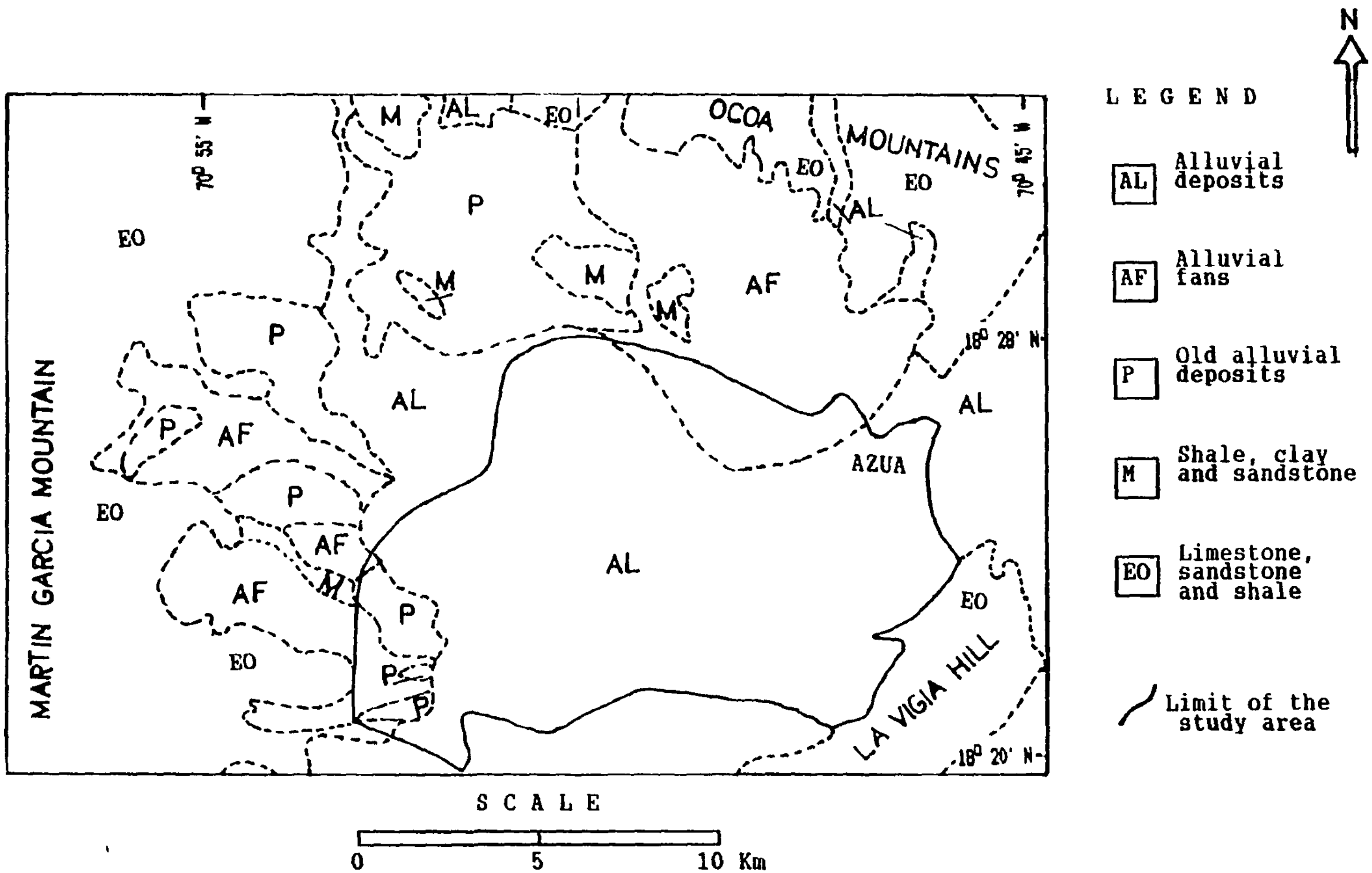

Figure 2 Geologic map of the Azua Valley

(c) Alluvial fans (AF), composed of a mixture of gravel, sand and clay extending from about 2 Km north of the Sanchez highway toward the southern limit of the valley. They have also been deposited over unit EO. Alluvial fan deposits probably range from Pleistocene to Recent in age.

(d) Pleistocene and Recent alluvial deposits of the valley (Al), composed of nonindurated gravel, sand, and clay in mixtures. These deposits constitute the basin-fill sediments of the Azua valley. Its thickness has been estimated to range up to several hundreds of meters. The clay content of the unit increases from north to south, as well as from the lower to the upper layers.

The alluvial deposits of the valley (Al) form the most important aquifer of the region. The old alluvial deposits and the alluvial fans have been identified as the main recharge areas of the aquifer. According to Tahal (1971), the permeable layers of the three units are connected and form what they named the Azua Valley aquifer. Recharge through the surface of the alluvial fill is minimum and is limited to small areas to the northwest and southwest of the valley. Figure 3 shows a generalized geologic cross section of the valley.

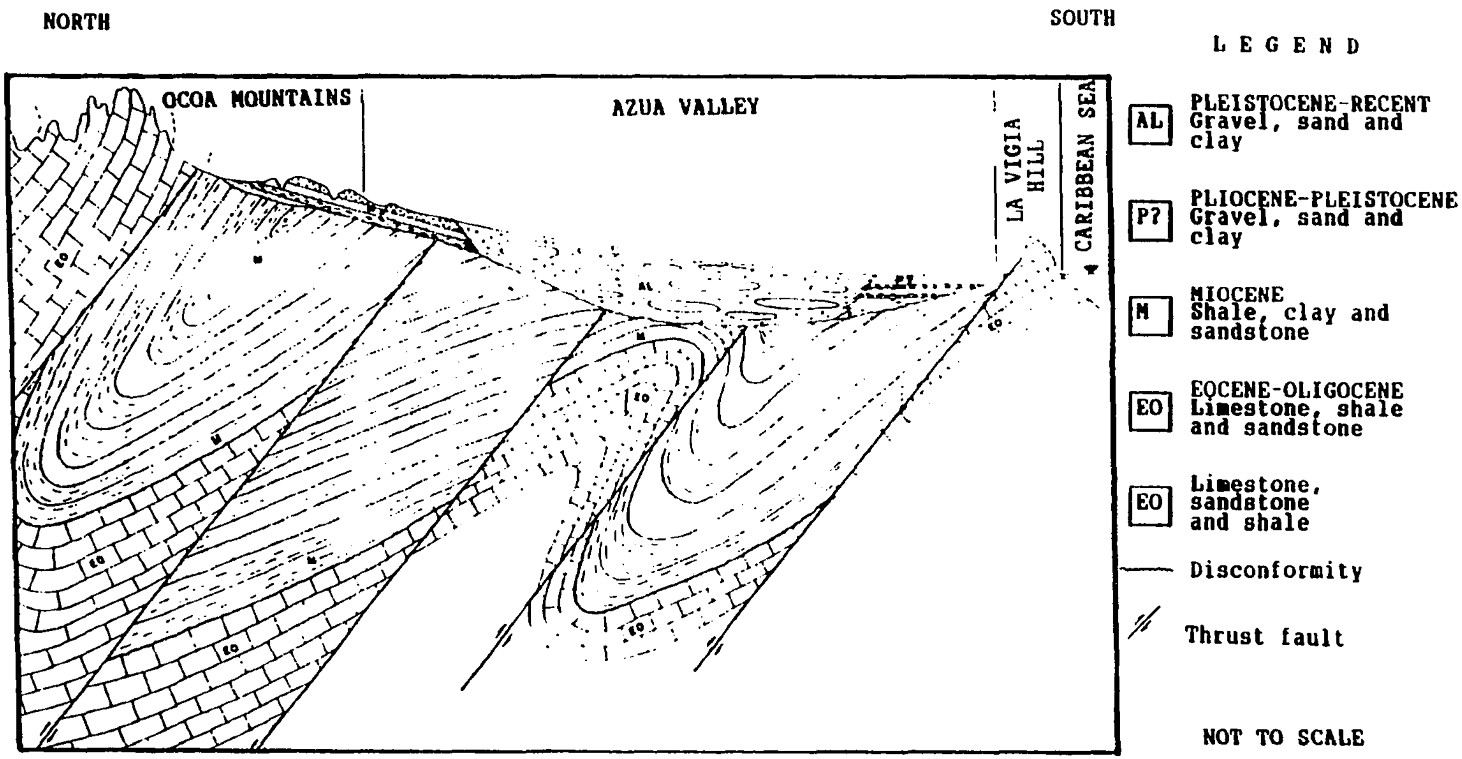

Figure 3 Generalized geologic cross-section through the Azua Valley.

Hydrogeology

Well Characteristics

All of the wells of the valley were drilled in the alluvial fill. Most of them were constructed before 1978 by means of cable-tool rigs. The wells vary in depths from 6 to 232 meters (20 to 760 feet), the deepest ones being exploration wells. The depth of production wells rarely exceeds 100 meters and the shallowest wells, located in the lowest part of the valley, are artesian, most of them flowing wells. None of the wells has commercial screen nor gravel pack. Oxyacetylene-slotted steel pipes serve as both casing and screen. All of the production wells are cased with slotted pipes from top to bottom and most of the exploration wells constructed for the Tahal's study (1971) are cased with individual 2"-slotted pipes for each aquifer which was penetrated. The use of these screening devices reduces significantly the life of the wells and increases maintenance costs. Inasmuch as preservation of casing strength is required (installation by percussion, commonly without auxiliary casing) the open area of the casing is very low and rarely exceeds 2.5%.

Lithology and Aquifer Characteristics

The lithology of the Azua Valley aquifer is the typical of alluvial basins. The sediments deposited by the rivers vary considerably in grain size and consist of very fine silt and clay, fine and coarse sand, and fine and coarse gravel. Layers of silt and clay are greater in thickness and areal extent in the southern parts of the valley, whereas lenses of coarse sand and gravel, normally intermixed with clay, predominate in the northern parts of the valley. Toward the center of the valley, the aquifer is made of layers of fine sand and gravel separated by layers of clay and silt.

Exploration and production wells have shown that the thickness of the aquifer layers varies from 10 m to 35 m whereas the thickness of the impermeable layers varies from 2 m to 40 m. Well #52 penetrated alternating layers of sand and clay and mixtures of both materials. Three aquifer layers were identified and no bedrock was found up to a depth of 232 meters. Exploration well #154 located 7 km west of #52 showed two layers of sand and gravel with approximately the same elevation and thickness of the two upper layers identified in #52. Exploration well #203 (5 Km east of #52, left margin of the Jura River) penetrated 50 m of alluvial material and then the EO Formation (limestone, shale, and sandstone) that outcrops at La Vigia Hill.

In the northern parts of the valley the aquifer layers are of coarser material and greater thickness. Thick layers of gravel, sand, and mixtures with clay were found along the Sanchez highway. These alluvial deposits are particularly important around the river beds of the Jura and Irabon Rivers. Although unconfined layers were penetrated by several wells west and southwest of Azua city, in most of the cases these layers were dry and the water was found under confined conditions in lower layers.

For the purpose of this study, due to the fact that in all of the wells the casings are open from top to bottom and that the hydraulic properties of the individual layers tapped by the wells are unknown, a single confined aquifer was assumed. This is as a realistic approach for analyzing the aquifer system of the valley. Furthermore, all available hydrogeological data (water level elevation, pumpage, transmissivity, water quality, etc.) are representative measurements of the multi-layer wells of the valley and include the individual contributions of the single layers.

Ground Water

Ground-water flow is generally southeastward in the northwest and south parts of the valley and southwestward-southward in the northeast part. Figure 4 is a map showing contours of ground-water levels in 1965. The map was constructed from the map published by Tahal (1971) which in turn was based on water-level measurements made by Gilboa (1965). The map gives a general idea of the ground-water flow pattern of the valley. In 1965, the regional gradient of the potentiometric surface was about 2 meters per kilometer (10.5 feet per mile) and the depth of the static level was from 0 m (artesian zone of the Las Barias-Rosario area) to 83 m (well #111 next to Los Jobillos).

Table 1 shows an inventory of 203 wells. Of these wells only 62 had pumping equipment installed in 1971, 38 were artesian and the rest were not in use, destroyed or abandoned. The distribution of wells which were operating from 1965 to 1985 as well as their corresponding pumping rates are presented in Table 2. During the site visit to the Azua Valley, the wells operating in July 1988 were found to be the same wells that were pumping in 1985. Then, it was assumed that the same pattern continued up to the end of 1988.

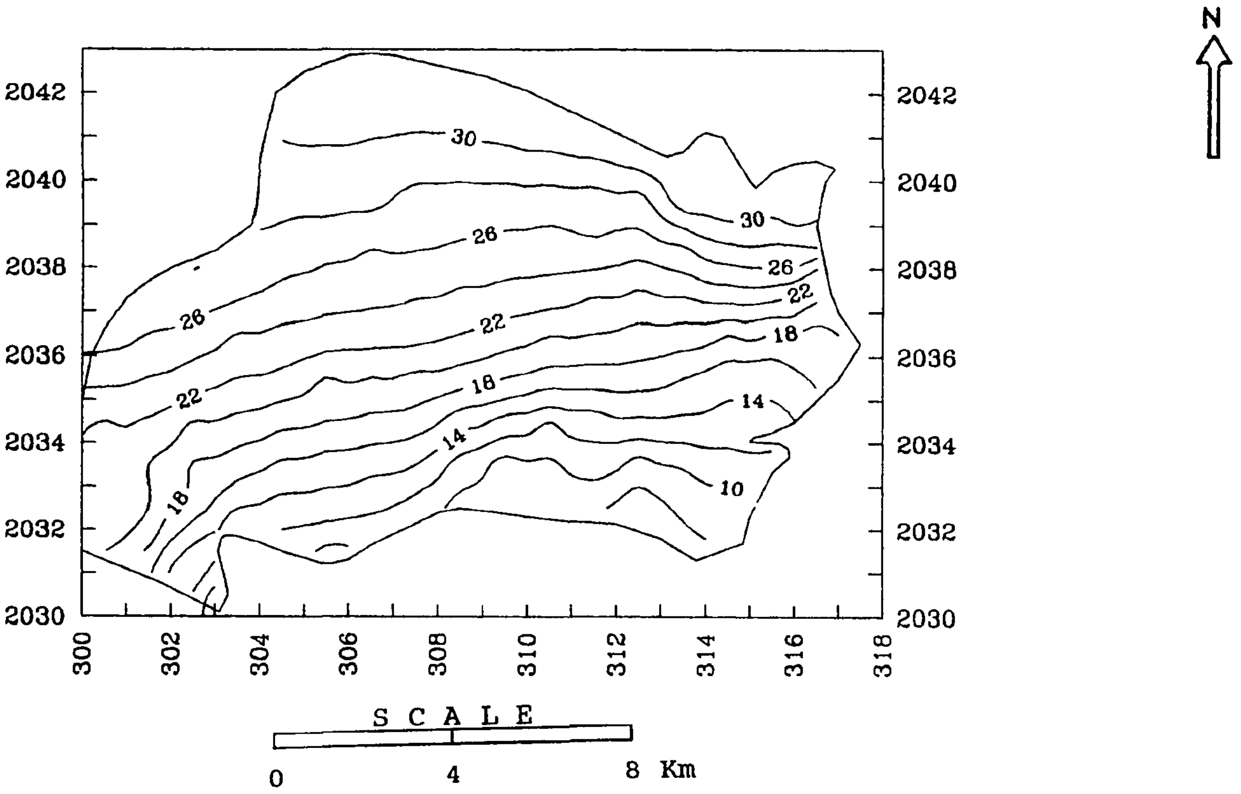

Figure 4 Contour map of measured hydraulic heads in 1965 (m)

Table 1 Inventory of wells in the Azua Valley

WELL 1	NO. 2	COORDINATES	DEPTH (m)	DIAM. (in)	OBSERVATIONS
1		07.9/45.8			aow
2		07.8/45.5			aow
3		07.9/45.6			aow
4		08.2/45.9			aow
5		08.0/45.0			aow
6		08.0/45.3			aow
7		08.0/45.2			aow
8		08.3/45.4			aow
9		08.3/45.5			aow
10	131	11.2/44.7	46	12	D
11	199	09.8/42.4	120	14	D
12	152	14.8/41.9	91	12	O
13		05.7/41.1	91	14	P, L, S:30-90, WSAZ
14	159	13.8/41.1	98	12	P, L, WSAZ
15	161	13.9/41.1	100	12	P, L, WSAZ
16	89	14.1/41.1	85	10	
17	13	14.1/41.0	91	10	P, IRR
18	160	14.0/41.0		12	
19	11	13.7/41.0			P
20	165	13.5/41.0		8	
21	12	13.2/40.9	91	14	P, IRR
22		13.2/40.4	90	14	P, L, S:23-90, WSAZ
23		13.9/39.9	62	14	P, L, S: 9-62, WSAZ
24		14.4/39.4	81	14	P, L, S: 9-81, WSAZ
25	14	14.1/40.4	81	10	
26	120-1	15.1/39.8	55	16	P, IRR
27	81	14.8/40.5	46	14	P, IRR
28		15.2/39.1	84	16	P, L, S: 9-84, WSTF
29		14.9/39.0	90	14	P, L, S: 9-90, WSTF
30	120	14.7/38.8	86	16	P, IRR
31		15.1/38.8	86	10	P, L, S:20-86, WSTF
32		14.9/39.3	89	16	P, L, S: 9-89, WSTF
33	4	13.6/39.2	99	14	P, IRR
34	1	12.4/39.9	97	14	P, IRR
35	2	12.5/39.5	88	14	P, IRR
36	206	12.0/39.1	91	12	L, S:21-91
37	9	11.6/38.3	91	14	
38	9-1	11.7/38.3		16	P, IRR
39	3	12.9/39.6	106	14	P, IRR
40	8	12.2/38.0	85	12	
41	5	13.5/38.4	85	16	P, IRR
42	7	12.8/37.9	91	14	D
43	6	12.9/38.6	85	14	P, IRR

Table 1 - continued

WELL 1	NO. 2	COORDINATES	DEPTH (m)	DIAM. (in)	OBSERVATIONS
44	122	14.6/37.8	45	8	
45	118	14.9/37.9	15	8	P, IRR
46	10	11.5/37.4	85	14	P, IRR
47	17	11.3/37.3	105	16	P, IRR
48	23	13.1/36.7	43	8	F
49	24	13.3/36.7	20	8	F
50	25	13.6/36.6	45	8	P, F, WSPV
51	26	13.7/36.8	15	8	F
52	180A	14.4/37.4	232	12/8	F, L, S1:206-212,2"
	180B	14.4/37.4	232	12/8	F, L, S2: 24- 30,2"
53	169	13.3/36.7		16	F, L
54	18	13.6/37.0	44	8	F
55	19	13.6/36.9	3	10	F
56	20	13.5/37.1	43	10	F
57	21	13.5/37.0	20	10	F
58	22	14.0/36.8	3		F
59	27	13.9/36.8	12		F
60	28	13.9/36.9			F
61	29	13.7/37.0	12	8	F
62	30	13.6/36.8	5	12	F
63	31	13.7/37.1	24	10	F
64	32	13.5/36.9			F
65	33	14.1/37.1			F
66	34	13.6/37.1		10	F
67	35	13.7/37.1	44	10	F
68	48-1	14.1/37.4	36	10	F
69	45	14.1/37.3	52	8	F
70	38	13.7/37.2	23		F
71	39	13.8/37.2		10	F
72	40	13.8/37.3	53	10	F
73	41	13.9/37.3		8	F
74	46	14.1/37.4	21	8	F
75	47	14.2/37.3	18	10	F
76	44	14.0/37.4		6	F
77	48	14.2/37.4	43	10	F
78	175	13.5/34.0	15	6	F, A, WM
79		08.5/31.9	28	10	F
80	168	14.8/33.9		16	F
81	123	14.2/36.4	49	10	F
82	51	09.8/38.1	100	14	F
83	55	10.2/38.2	90	12	F
84		11.2/35.6		8	F
85		11.1/35.5		8	F
86	16	11.0/34.8	61		F

Table 1 - continued

WELL 1	NO. 2	COORDINATES	DEPTH (m)	DIAM. (in)	OBSERVATIONS
87	130	10.5/36.3	10	6	F
88	184	12.1/31.3	125	10	F, L
89	183A	08.6/31.2	107	12/8/6	F, L, S1: 36-42, 2"
	183B	08.6/31.2	107	12/8/6	F, L, S2: 3- 9, 2"
90	69	09.9/34.4		10	F
91	68	10.1/34.3	4	12	F
93	56	10.2/38.3	90	14	
94	57	10.4/38.8	85	14	
95	70	09.4/33.8	2	10	F
96	71	09.5/33.7	1	10	F
97	73	09.4/33.6			F
98	72	09.4/33.5			F
99	74	09.4/33.7	1	10	F
100	75	09.1/33.5	1	23	F
101	125	09.6/33.0			F
102	84	09.1/32.7			F
103		08.6/32.8	20	10	P, IRR
104		08.1/37.1	36	12	P, IRR
105		08.0/31.8	33	8	
106		07.7/31.7	20	10	
107		07.6/31.5	4	36	DW
108		07.4/31.3			
109		07.3/31.1			
110	110	07.1/31.1	21	10	D
111		07.1/30.8	34	10	
112		06.7/30.3			
113	188A	06.5/30.9	207	12/10/8/6	L, S1: 85-103, 2"
	188B				L, S2: 10- 16, 2"
114	54	10.5/38.3	91	14	
115	107	05.9/30.3	45	12	
116	148	05.9/30.4	33	8	
117	53	11.1/38.8	91	14	
118	86	05.8/30.7	19		
119	85	05.6/30.7	50	8	
120	127	11.3/38.2	40		P, IRR
121	52	10.8/39.4	91	14	
122		06.8/32.7	81	14	L
123	58-1	11.4/39.7	91	16	
124	126	11.4/40.2	70	10	P, IRR
125	144	05.6/32.4	24	10	D
126	67	04.6/31.8	50		P, IRR
127	181A	03.4/31.2	150	12/10	L, S1:109-116, 2"
	181B				L, S2: 79- 85, 2"
	181C				L, S3: 65- 71, 2"

Table 1 - continued

WELL 1	NO. 2	COORDINATES	DEPTH (m)	DIAM. (in)	OBSERVATIONS
128		03.2/31.7			
129	63	03.8/32.8	45	12	
130	65	04.9/32.4	50	10	P, IRR
131	143	04.4/32.5	18	4	
132	62	03.9/32.8	45	12	P, IRR
133	66	04.7/32.8	50	10	P, IRR
134	64	04.2/32.8	48	16	
135	142	35.5/32.8	28		P, IRR
136	60	05.1/33.1	45	14	P, IRR
137	59	04.7/33.1	55	14	P, IRR
138	58	04.1/33.1	60	12	P, IRR
139		03.5/33.5	95	14	L
140	190A	05.1/33.7	192	12/8	L, S1:185-192, 2"
	190B				L, S2: 19-25, 2"
141		04.9/34.7	84	14	P, L, S: 19-84, IRR
142		04.3/34.2	95	14	P, L, IRR
143	195	03.9/34.2	49	12	L, S: 46-49, 2"
144		03.3/34.6			P, IRR
145	207	03.7/35.0	98	20/14	L, S: 15-98, 14"
146		03.5/35.7	90	14	
147		02.6/35.7	95	14	L, S: 24-95
148		02.6/36.3	88	14	L
149	205	07.7/34.9			
150	187	07.1/35.8	122	18/14	L, S: 39-122, GP, D
151	156	07.0/36.4		14	L, D
152	157	06.2/36.4	96	14	P, L, S: 26-93, D
153	116	07.9/36.5	94	14	L, S: 26-93, D
154	189	08.2/36.1	91	14	L, S: 25-91, D
155	185	08.9/39.3	91	14	L, S: 10-89
156		09.2/36.8	100	14	L, S: 21-100
157		08.9/37.3	95	14	P, L, S: 29-95, IRR
158	117	08.1/37.1	103	14	P, L, S: 27-102,IRR
159		08.5/37.8	109	14	P, L, S: 33-109,IRR
160		08.0/38.0	91	14	P, L, S: 20-91, IRR
161	114	07.1/37.8	95	14	P, L, S: 34-94, IRR
162	115	06.6/38.1	127	14	P, L, S: 32-94, IRR
163	121	05.9/37.8	35	16	D
164		04.3/37.4	107	14	L, S: 41-107
165		03.7/38.2	104	14	L, S: 29-104, D
166	196	04.3/28.3	117	12	
167		05.5/38.2	101	14	L
168	113	07.7/38.4	97	14	P, L, S: 34-95, IRR
169		07.2/38.8	83	12	
170	194	05.9/39.3			D

Table 1 - continued

WELL 1	NO. 2	COORDINATES	DEPTH (m)	DIAM. (in)	OBSERVATIONS
171		06.7/39.2	115	14	L, S: 48-115
172	163	07.8/39.1	111	14	P, L, S: 33-111,IRR
173		08.4/39.2	93	14	L, S: 27-93
174		08.2/39.6	107	14	P, IRR
175	162	07.8/39.1	112	14	P, L, S: 35-111,IRR
176		08.4/39.9		14	P, IRR
177	164	06.2/39.9	112	14	L, S: 35-111
178	198	07.8/41.1	120	14	
179	193	06.7/41.2			D
180	111	05.9/41.3	142	14	P, L, S: 14-61, D
181	197	04.3/41.6			D
182	192A	06.7/42.9	186		L, S1:162-168, 2"
182	192B				L, S2:143-149, 2"
	192C				L, S3:113-119, 2"
183	191A	99.6/35.7	176	20/12/10/8	L, S1:150-156, 2"
	191B				L, S2: 57-63, 2"
	191C				L, S3: 5-11, 2"
184		15.6/40.9	70	14	P, IRR
185		16.0/40.6	89	14	
186		16.0/40.9			D
187		16.1/40.5	91	14	P, IRR
188		15.1/39.5	72	12	
189		16.3/39.1	73	10	P, IRR
190		17.2/39.6	70	10	L, D
191		15.9/38.6	70	16	
192		15.9/38.4		16	P, IRR
193		15.6/37.9	42	12	P, IRR
194		16.1/38.6	70	16	
195		16.3/38.1	40	6	
196		17.2/37.7			D
197		16.4/36.9	15	8	D
198		16.4/36.7			D
199		17.2/36.9	85	10	
200	182	15.5/34.7	88	12	L, S: 68-74, 2"
201		18.1/40.3	103	8	P, WS church
202		18.9/39.9			
203	186A	18.4/37.8	110	12/8	L, S1: 75-105, 2"
	186B				L, S2: 35-47, 2"

Table 1 - continued

EXPLANATIONS

WELL NUMBERS	1.	Assigned in this report according to numbers used by SNC (1984).
	2.	Assigned by Tahal (1971). Wells constructed after 1971 are not numbered.
COORDINATES		Referred to topographic map AMS Scale 1:50000 (1969).

LIST OF ABBREVIATIONS

A	abandoned well
aow	abandoned oil well
D	destroyed well
DW	dug well
F	flowing well
IRR	well used for irrigation
L	known well log
O	obstructed well
P	pumping well
S:60-70, 2"	slotted-pipe interval (m) and diameter
WM	windmill
WS	well used for water supply
WSAZ	well used for water supply to Azua city
WSPV	well used for water supply to Pueblo Viejo
WSTF	well used by tomato-past factory

History of Ground-Water Development

Prior to 1978, the agricultural and urban development that took place in the Azua Valley was based on pumpage of its ground-water resources.

As shown in Table 2 and Figure 5 pumpage increased from about 20 MCM/year in 1965 to 70 MCM/year in 1977. From 1972 to 1977, pumpage increased by nearly 25 MCM/year and water levels declined 20-30 m in the Azua-Las Clavellinas area. In addition, flow from all of the artesian wells ceased in 1977.

Inasmuch as the Yaque del Sur-Azua canal started operation in 1978, pumpage was reduced to about 30 MCM/year. The only wells that continued pumping were those supplying water to Azua City and to some agricultural areas out of the service area of the canal. Water levels rose more than 30 m in some areas of the northern part of the valley and artesian wells (Pérez, 1979) started to flow again thus flooding about 3500 hectares of the lowlands.

In 1979, a certain number of wells were destroyed by hurricane David which caused a 3-day precipitation which was 26% larger than the mean annual precipitation (Febrillet, 1979). There are 52 of these wells whose rehabilitation has been proposed (Gurevitz, 1985) with the purpose of irrigation and mitigation of drainage problems.

Table 2 Nodal distribution of pumpage in the study area in MCM/year

ROW	COL	NW	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985
2	6	1														0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
2	14	3			0.54	0.54	0.54	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62
2	15	1			0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
2	16	1			0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67								
3	8	1								0.82	0.82	0.82	0.82	0.82	0.82								
3	12	1						0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04								
3	14	2			0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07
3	15	1					1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65
3	16	1	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54								
3	17	2	0.54	0.54	0.54	0.54	0.54	0.54	0.59	0.59	0.59	0.59	0.59	0.59	0.59								
4	8	1								1.27	1.27	1.27	1.27	1.27	1.27								
4	9	4								0.27	4.05	4.05	4.05	4.59	4.59								
4	11	1							0.07	0.07	0.07	0.07	0.07	0.07	0.07								
4	12	1	0.30	0.30		0.30	0.30	0.30															
4	13	3	3.55	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	3.50	3.50	3.50	3.50	3.50	3.50	3.50	3.50
4	14	2		1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	1.55	3.12	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57
4	15	3								2.62	4.58	4.58	4.58	4.58	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15	6.15
4	16	2	0.25					0.25		1.18	1.18	1.18	1.18	1.18	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.43
4	17	1									0.18	0.18	0.18	0.18	0.18								
5	7	1					1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32								
5	8	1						0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27								
5	9	1												0.82	0.82								
5	10	1			0.55	0.55	0.55																
5	11	4			1.22	2.02	2.02	1.67	1.67	2.02	2.02	2.02	2.02	2.02	2.02	0.32	0.32	0.32	0.32	0.32	0.32	0.32	0.32
5	12	3	1.35	1.35	1.35	2.39	2.39	2.46	2.46	2.46	2.46	2.46	2.46	2.46	2.46	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35
5	13	2	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57	1.57								
5	14	1	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50								
5	15	1		3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94
5	16	2	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	1.59	1.59	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
5	17	2	0.14	0.14	0.14	0.14	0.14	0.14	0.64	0.64	0.64	0.64	0.64	0.64	0.64								
6	8	1						0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88								
6	9	4					0.21	1.36	1.36	1.36	1.36	1.36	1.36	2.91	2.91								
6	12	2	0.50	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60								
6	13	1	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25								
6	14	11	1.68	1.68	1.68	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.26	1.26	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
6	15	8	0.44	0.38	0.38	0.38	0.38	0.38	0.37	0.37	0.37	0.37	0.37	0.22	0.22	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38
6	16	2	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.36	0.36								

Table 2 - continued

ROW	COL	NW	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985
7	7	1						0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44								
7	8	1						0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41								
7	9	1								0.27	0.27	0.27	0.27	0.27	0.27								
7	11	1	0.01	0.01	0.01	0.01	0.01	0.01															
7	14	10	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.34	1.34	1.43	1.43	1.43	1.43	1.43	1.43	1.43	1.43
7	15	2	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.31	0.31	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34
8	12	2	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
9	5	2												1.78	1.78								
9	10	1	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
9	11	1	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
9	12	1	0.09	0.09	0.09	0.09	0.09	0.09															
10	5	2				2.18	2.18	0.71	2.18	0.71	0.71	0.71	0.71	2.18	2.18								
10	6	1				1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13								
10	10	8	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	0.94	0.94	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86
11	4	2	0.64	0.64	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23								
11	5	3			0.39	2.61	2.61	2.61	2.61	2.61	2.61	2.61	2.61	2.61	2.61								
11	9	1				0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27								
11	10	1	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
12	5	1				0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89								
12	8	3										0.24	0.24	0.24	0.24								
13	6	2	0.04	0.04	0.04	0.04	0.04	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16								
13	7	1										0.08	0.08	0.08	0.08								
13	8	1										0.08	0.08	0.08	0.08								

EXPLANATIONS

1 MCM/YEAR = 10^6 m^3 /year

NW = number of pumping wells in the cell

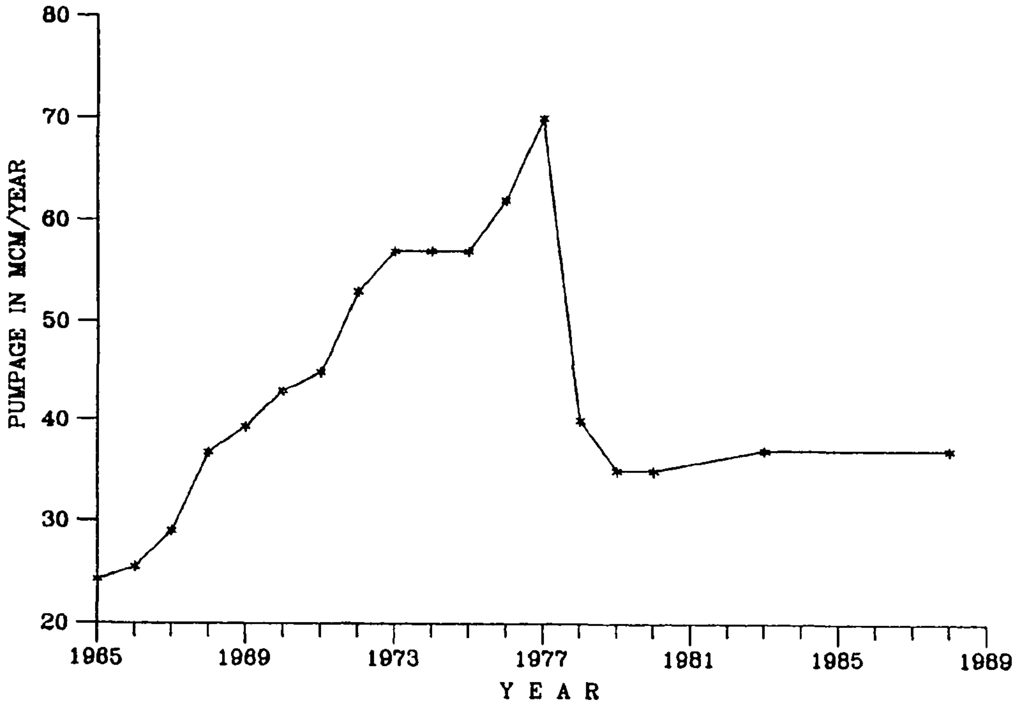

Figure 5 Evolution of pumpage in the Azua Valley

CHAPTER 3

GROUND-WATER MODELING

A ground-water flow model was developed to simulate the behavior of hydraulic heads in the Azua aquifer under the assumed pumping stresses. The effects of irrigation return water, evapotranspiration, drainage, and river-canals on hydraulic heads were analyzed. The behavior of drawdown at selected nodes within critical areas of the aquifer was analyzed for seven pumping scenarios. The analyses were projected 22 years into the future.

The USGS Model

The computer model program used was MODFLOW, the three-dimensional ground-water flow model of the United States Geological Survey (McDonald and Harbaugh, 1988). This model is capable of simulating three dimensional flow with a heterogeneous set of parameters (transmissivity, hydraulic conductivity, storage coefficient, specific yield, and conductance), irregularly shaped boundaries and heterogeneous boundary conditions. MODFLOW has a modular structure consisting of a set of relatively independent subroutines that may simulate river flow, drain flow, recharge, and evapotranspiration (Maddock, 1987, 1988).

MODFLOW requires the area of interest to be divided into a rectangular finite-difference mesh. The elements of the mesh are commonly described as blocks or cells and their centers are called nodes. Within each block the hydraulic properties of the aquifer layers are assigned to the nodes. The model solves the ground-water flow equation numerically by computing hydraulic heads at the nodes. Resultant flows between blocks due to the imposed stresses are also computed. The flow equation used in MODFLOW for representing the three-dimensional movement of ground water of constant density through the porous material has the form (McDonald and Harbaugh, 1988)

$$\frac{\partial}{\partial x}\left[K_{xx}(\hat{x},z)\frac{\partial h(\hat{x},z,t)}{\partial x}\right] + \frac{\partial}{\partial y}\left[K_{yy}(\hat{x},z)\frac{\partial h(\hat{x},z,t)}{\partial y}\right] + \frac{\partial}{\partial z}\left[K_{zz}(\hat{x},z)\frac{\partial h(\hat{x},z,t)}{\partial z}\right] - W(\hat{x},z,t) = S_s(\hat{x},z)\left[\frac{\partial h(\hat{x},z,t)}{\partial t}\right] \quad (1)$$

where

$K_{xx}(\hat{x},z)$, $K_{yy}(\hat{x},z)$ and $K_{zz}(\hat{x},z)$ are hydraulic conductivities in the x, y and z principal coordinate directions, respectively, at point $\hat{x}$; dimensions Lt^{-1};

$h(\hat{x},z,t)$ is the hydraulic head at time t at point $(\hat{x},z)$; dimension L;

$W(\hat{x},z,t)$ is a source/sink term at point $(\hat{x},z)$ and time t, expressed as a volumetric flux per unit volume; dimension t^{-1};

$S_s(\hat{x},z)$ is the specific storage of the porous medium at point $(\hat{x},z)$; dimension L^{-1};

$\hat{x}$, z represent the spatial coordinates of x, y and z, with $x=(\hat{x},y)$ horizontal and z vertical; and

t is the time; dimension t.

The Modeled Area

The modeled area encompasses a portion of the Azua valley within the service area of the Yaque del Sur-Azua canal (Figure 6). The La Vigia Hill and Martin Garcia Mountains form the eastern and western boundaries, respectively. The alluvial fans of the recharge area form the northern boundary, whereas the southern boundary coincides with the southern limit of the Azua Irrigation Zone.

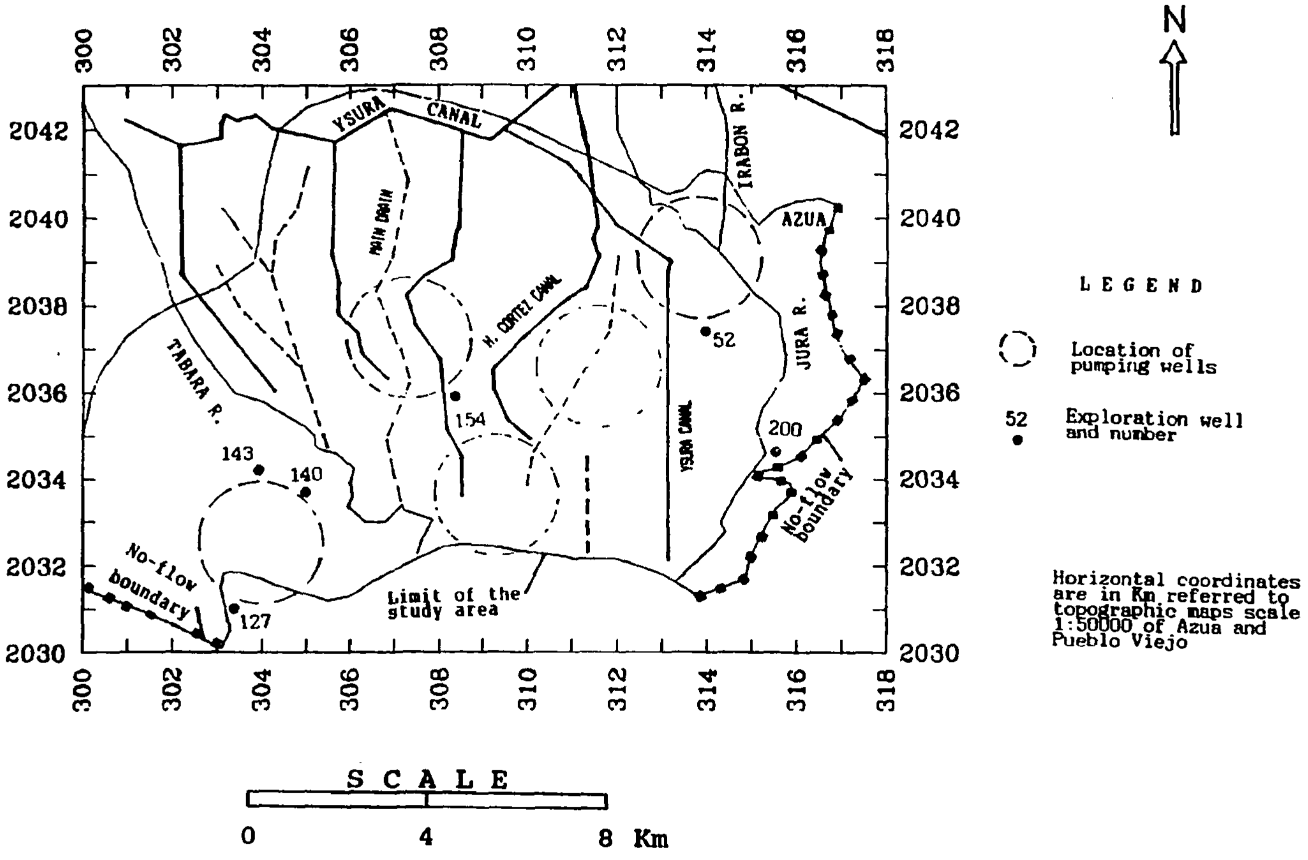

Figure 6 Reference map of the study area.

Ground-Water Flow Equation for the Modeled Area

A quasi three-dimensional flow option of MODFLOW was used for modeling the alluvial aquifer of Azua. The aquifer was assumed as a single confined layer whose parameters such as transmissivity (T) and storage coefficient (S) were assumed to represent the more complex aquifer system described in Chapter 2.

The flow of ground water through the aquifer was assumed to be horizontal only. Thus, the equation used (Freeze and Cherry, 1979) was

$$\frac{\partial}{\partial x}\left[T_{xx}(\hat{x})\frac{\partial h(\hat{x},t)}{\partial x}\right] + \frac{\partial}{\partial y}\left[T_{yy}(\hat{x})\frac{\partial h(\hat{x},t)}{\partial y}\right] - W(\hat{x},t) = S_s(\hat{x})\left[\frac{\partial h(\hat{x},t)}{\partial t}\right] \qquad (2)$$

Data Requirements

The data required by the model include the following:

1.- Ground-water elevation (hydraulic head)

2.- Pumpage

3.- Recharge

4.- Aquifer transmissivity

5.- Aquifer storage coefficient

6.- Optional data:

a) Rivers-canals

b) Drains

c) Evapotranspiration

d) Recharge

7.- Geographic coordinates

To supply the data, all known sources of information available at the Division of Hydrogeology of the Instituto Nacional de Recursos Hidraulicos (INDRHI) of the Dominican Republic, were reviewed. In addition, previous studies of the area, especially the ones performed by Tahal (1971, 1983) and INDRHI-SNC (1984), were also reviewed. Very valuable data were supplied by the Azua Irrigation District of INDRHI and the USAID's project "On Farm Water Management" during the site visit of July 1988.

Data Availability

Although the Azua valley is the region of the Dominican Republic where a number of hydrogeological studies have been undertaken, data for developing reliable estimates of aquifer parameters are scarce.

Aquifer test analyses performed before 1983 were revised by INDRHI-SNC (1984). Due to very large errors the computed parameters were considered invalid. The parameters were not corrected and, in turn, a program of aquifer tests covering the most important areas of the aquifer was proposed.

Measurements of ground-water levels have been published by Tahal (1971), INDRHI-Tahal (1983), and INDRHI-SNC (1984). The simulations of this study rely entirely on the published data and on the measurements made by the author in 1988.

Data on pumpage, irrigation recharge, and river infiltration were based on estimates published in the reports mentioned above. Data on drains, canals, and evapotranspiration were based on the information collected during the site visit.

Development of the Model

The development of the model implied the design of a finite-difference mesh, the selection of initial input data, and the calibration of the model under steady state and transient conditions.

Finite-Difference Grid

The area to be modeled was divided into a uniform grid with each square representing one square kilometer (Figure 7). The grid contained 13 rows and 18 columns. The nodes of the grid were classified as either active or inactive. Active nodes represent blocks within the modeled area whereas inactive nodes represent blocks out of the modeled areas. This finite-difference grid consisted of 159 active block-centered nodes.

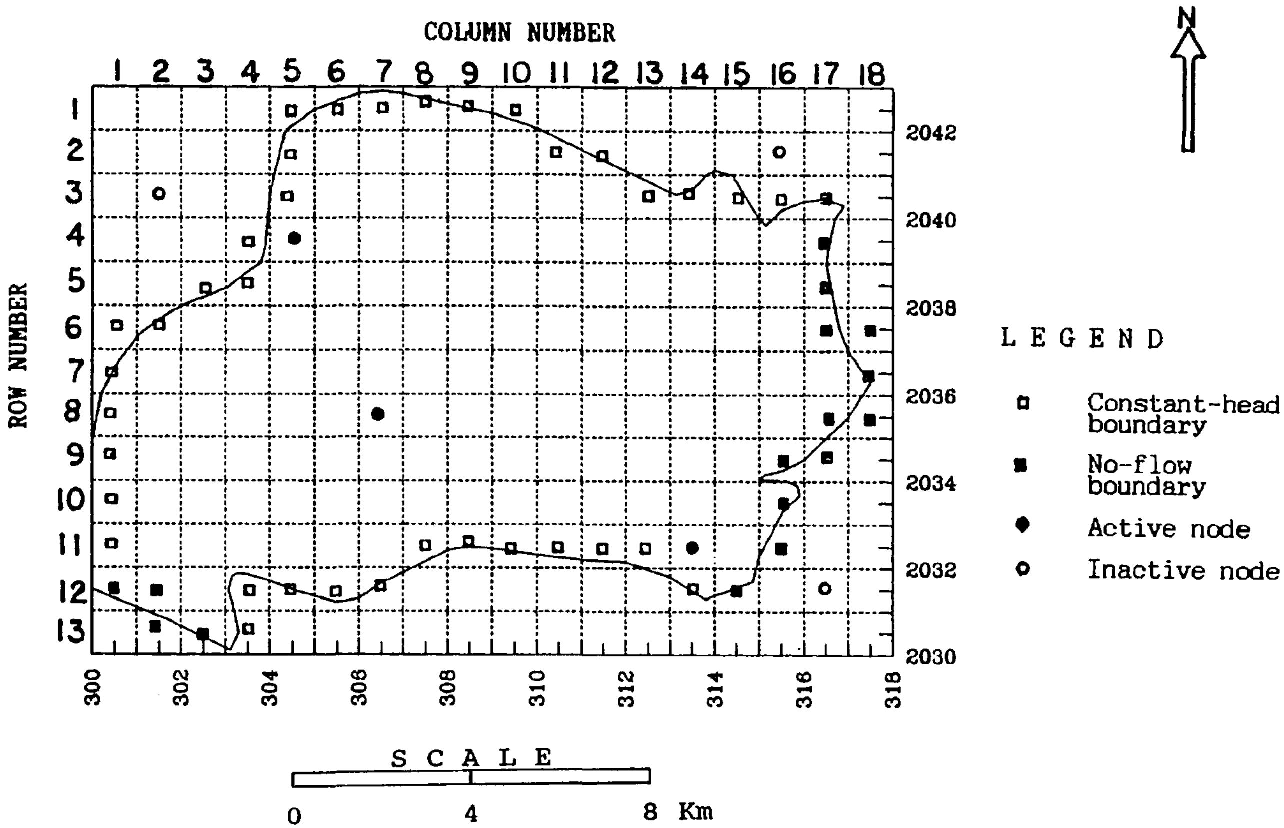

Figure 7 Finite-difference grid and boundary conditions.

Initial Input Data

Hydraulic Head

Initial head values relied heavily on those measured by Gilboa (1965) and published by Tahal (1971) as a contour map (Figure 4). Inasmuch as the measurements made in 1965 did not cover the entire aquifer, the contour lines were extrapolated, especially in the areas northwest of Los Jobillos and in the vicinity of Los Negros and Galindo, at the western boundary of the modeled area. Averages of the contour lines within each block were taken as initial heads for 1965 and assigned to the nodes.

Transmissivity

Transmissivity measures the ability of an aquifer to transmit water. This property is defined as the rate at which water is transmitted through a unit width of aquifer under a unit hydraulic gradient. Numerically, transmissivity is the product of the hydraulic conductivity of the aquifer and its saturated thickness.

The initial values of transmissivity used in the model were based on the values reported by Tahal (1971) and on two aquifer tests run in 1988 (Table 3). Corrections to three of the pumping-test analyses of 1971 and one of 1988 were made. The transmissivity values computed for wells 47, 88 and 156 were corrected for casing storage effects (Driscoll,

1987) whereas the pumping test for well 33 was corrected for computational errors. In the first case, the resulting transmissivities increased by 30% to 100%. In the second case, the value reported for well 33, at node (4,14), decreased by about 30%.

Notice that the transmissivity values reported by Inapa are the lowest. This is due to the characteristics of the wells and to the type and capacity of the pump used for the tests. Inasmuch as these wells were constructed for supplying modest amounts of water to small villages, their diameters are small (6 inches) and penetrate not more than 3 meters (10 feet) into the aquifer. Due to the fact that the pumping tests run by Inapa were of very short duration (up to 30 minutes) and that low-capacity pumps (30-50 GPM) were used for the tests, the time-drawdown values used for computing transmissivity were highly affected by casing storage effects. Corrections to the transmissivity values of wells at nodes (8, 14), (4, 17) and (5, 15) were not made and these data were not used.

The average nodal values ranged from 100 m^2/day at the eastern edge of the valley to 7500 m^2/day near the Jura River at Las Clavellinas (Table 4).

For areas where aquifer tests were not available, reasonable estimates were made based on the transmissivities of surrounding nodes, the resistivity map of 1966 and well logs. Estimates were also made by using a flow net for the steady-state head configuration (Davis and DeWiest, 1966) and comparing these results with the values obtained by the other methods.

Table 3 Transmissivity data obtained from pumping tests in the study area

WELL NO. 1	WELL NO. 2	GRID COORDINATES ROW	GRID COORDINATES COLUMN	TRANSMISSIVITY m^2/day	SOURCE
13		2	6	4,250	Indrhi, 1988
26	120-1	4	16	11,000	Tahal, 1971
33	4	4	14	15,200	Gilboa, 1965
33	4	4	14	10,900	Indrhi, 1988
38	9-1	5	12	4,200	Gilboa, 1965
47	17	6	12	2,200	Tahal, 1971
50	25	7	14	226	Inapa
85	55	5	11	2,300	Gilboa, 1965
88	16	9	12	100	Tahal, 1971
120	86	13	6	160	Tahal, 1971
	122-1	5	15	32	Inapa
123	52	4	11	6,700	Gilboa, 1965
	124-1	11	9	465	Tahal, 1971
125	58-1	4	12	760	Gilboa, 1965
126	126	3	12	120	Inapa, 1971
134	62	11	4	7,300	Tahal, 1971
	137	8	14	7	Inapa
156	189	7	9	1,180	Tahal, 1971
	158	2	14	485	Unknown
170	113	5	8	2,600	Tahal, 1971
191	94	4	17	60	Inapa

EXPLANATIONS

WELL NO. 1 : used in this study
2 : assigned by Tahal (1971)

INAPA Instituto Nacional de Agua Potable

INDRHI Instituto Nacional de Recursos Hidraulicos

Table 4 Initial nodal transmissivity values

Row	Column	Transmissivity (m^2/day)
1	5	2400.
1	6	2400.
1	7	2400.
1	8	2400.
1	9	2400.
1	10	2400.
2	5	2400.
2	6	2400.
2	7	3200.
2	8	3200.
2	9	3500.
2	10	5000.
2	11	2000.
2	12	2400.
2	13	3800.
2	14	4000.
2	15	1500.
3	5	2400.
3	6	2400.
3	7	3200.
3	8	3200.
3	9	3500.
3	10	4500.
3	11	3200.
3	12	3500.
3	13	4500.
3	14	4500.
3	15	5500.
3	16	3000.
3	17	1100.
4	4	2000.
4	5	2400.
4	6	2400.
4	7	4200.
4	8	4200.
4	9	4200.
4	10	4000.
4	11	3500.
4	12	3500.
4	13	3500.
4	14	7500.
4	15	7500.

Table 4 - continued

Row	Column	Transmissivity (m^2/day)
4	16	3000.
5	3	2000.
5	4	2000.
5	5	2400.
5	6	2800.
5	7	4200.
5	8	4200.
5	9	4200.
5	10	4000.
5	11	4000.
5	12	3500.
5	13	3200.
5	14	3200.
5	15	3000.
5	16	1000.
6	1	2500.
6	2	2500.
6	3	2500.
6	4	2500.
6	5	2500.
6	6	2500.
6	7	3000.
6	8	3000.
6	9	3000.
6	10	4000.
6	11	4000.
6	12	2000.
6	13	2000.
6	14	2000.
6	15	950.
6	16	700.
7	1	2500.
7	2	2500.
7	3	2550.
7	4	2500.
7	5	2500.
7	6	2500.
7	7	2500.
7	8	2500.
7	9	2500.
7	10	2500.
7	11	2500.
7	12	2500.

Table 4 - continued

Row	Column	Transmissivity (m^2/day)
7	13	1500.
7	14	1000.
7	15	500.
7	16	300.
7	17	100.
8	1	1500.
8	2	1500.
8	3	1500.
8	4	2000.
8	5	2000.
8	6	2000.
8	7	2000.
8	8	2000.
8	9	2000.
8	10	2000.
8	11	2000.
8	12	2000.
8	13	2000.
8	14	500.
8	15	500.
8	16	300.
9	1	1500.
9	2	1500.
9	3	1500.
9	4	2000.
9	5	2000.
9	6	2000.
9	7	2000.
9	8	1000.
9	9	1000.
9	10	2000.
9	11	2000.
9	12	2000.
9	13	400.
9	14	400.
9	15	400.
10	1	1000.
10	2	1000.
10	3	1000.
10	4	2500.
10	5	2500.
10	6	2500.

Table 4 - continued

Row	Column	Transmissivity (m^2/day)
10	7	2000.
10	8	1000.
10	9	1000.
10	10	1500.
10	11	1500.
10	12	1500.
10	13	500.
10	14	400.
10	15	300.
11	1	1000.
11	2	1000.
11	3	1000.
11	4	2500.
11	5	2500.
11	6	2500.
11	7	1500.
11	8	1500.
11	9	1000.
11	10	1000.
11	11	1000.
11	12	1000.
11	13	400.
11	14	300.
11	15	300.
12	3	1000.
12	4	2000.
12	5	2000.
12	6	1500.
12	7	1000.
12	13	800.
12	14	300.
13	4	1000.

Storage Coefficient

The storage coefficient measures the ability of a confined aquifer to release water from or take water into storage.

This property is defined as the volume of water that an aquifer releases or takes into storage per unit surface area of aquifer per unit change in head. Numerically, the storage coefficient is the product of the specific storage of the aquifer and its saturated thickness. Tahal (1971), INDRHI-Tahal (1983) and INDRHI-SNC (1984) analyzed the Azua aquifer as confined and reported average values of the storage coefficient ranging from 0.10 to 0.15. Although these values may look high when compared to the typical values for confined aquifers, they reflect the fact that the aquifer system is a combination of layers of granular materials with high specific storage and large thickness. In addition, since these average values were used for the entire aquifer, they take into account the estimated specific yield for the areas where the aquifer is partially unconfined. Based on these assumptions, a storage coefficient of 0.08 was initially used for the transient calibration.

Rivers and Canals

MODFLOW simulates the effects of flow between surface-water bodies and ground-water systems in terms of the head gradient between the streams and the aquifer. Leakage to or from rivers and canals was simulated for the subset of river reaches corresponding to the Jura River, the Irabon River, and the network of irrigation canals. Inasmuch as the interaction between the Tabara River and the aquifer is negligible within the modeled area, leakage was not simulated for this river.

Each river reach is entirely contained in a cell and the parameters used for computing leakage are assigned to the node. Leakage is calculated by means of the equation

$$Q_{riv} = C_{riv}\,(H_{riv} - h_{i,j}) \qquad (3)$$

where

Q_{riv} is the flow between the stream and the aquifer, taken as positive if the river recharges the aquifer and

C_{riv} is the hydraulic conductance of the riverbed, expressed as

$$C_{riv} = \frac{K_{rb}\,L_r\,W_r}{M_{rb}} \qquad (4)$$

where

K_{rb} is the hydraulic conductivity of the riverbed;

M_{rb} is the thickness of the riverbed;

L_r is the length of the reach;

W_r is the width of the river;

H_{riv} is the river stage; and

$h_{i,j}$ is the head at the cell containing the reach.

When the head in the aquifer falls below the bottom of the riverbed MODFLOW modifies equation 3 to

$$Q_{riv} = C_{riv} (H_{riv} - H_{bot}) \tag{5}$$

where

H_{bot} is the elevation of the bottom of the river bed.

Leakage from the rivers to the aquifer was computed based on the measurements made during the site visit in 1988 and the streamflow values reported by Tahal (1971). The average leakage computed for the Jura River and the Irabon River was about 3650 m^3/km/day. For an average river width and riverbed thickness of 20 meters, a length of one kilometer and a difference of 1.22 m between river stage and head at node (2,13), the conductance given by equation (3) is 3000 m^2/day. Then, the vertical hydraulic conductivity of the river bed obtained from equation (4) is 3 m/day. This value is one order of magnitude less than the average horizontal hydraulic conductivity of the aquifer and decreases to very small values downstream the confluence of both rivers.

Average river stage varies from about 55 m above sea level north to the Sanchez highway to 8 meters above sea level at Las Terreras. Conductance varies from 3000 m^2/day north of Las Clavellinas to 50 m^2/day at Las Terreras.

Drains

The simulation of the effects of existing agricultural drains was made by using the Drain option of MODFLOW. The model computes the flow to the drain by multiplying a conductance factor by the head difference between the aquifer and the median drain elevation. When the head in the aquifer is below the median drain elevation, the flow to the drain is set to zero. The conductance factor describes all kinds of head losses between the drain and the cell and can be estimated from measured values of drain flow and head differences (McDonald and Harbaugh, 1988).

The number of active drain nodes increased from 7 for the steady state simulation to 38 for the transient simulation. Initial conductance values for the main agricultural drains were assumed based on field data supplied by the project On Farm Water Management in 1988. Drain elevations were taken from the maps supplied by the project.

Irrigation Recharge

Irrigation water has been applied in the study area since the 1950's. The irrigated area increased from less than 2000 hectares in 1965 to about 6500 hectares in 1988. The sustained agricultural development demanded increasing amounts of irrigation water that were supplied by both ground water and surface water. Before 1979 irrigation was mainly based on ground water and surface water supplied by the Hernan Cortez canal. Inasmuch as the Yaque del Sur-Azua canal started operation at the end of 1978, thus bringing large amounts of water to the irrigation district, surface water became the main source of irrigation.

Local irrigation practices and deficiencies of the irrigation network produce irrigation returns that percolate through the soil and recharge the aquifer. Areal recharge was estimated for 105 active nodes based on INDRHI-SNC

(1984) and unpublished data supplied by the Azua Irrigation District. The net recharge rate per unit area varied from 1.25×10^{-5} m/day to 1.25×10^{-4} m/day for the period 1965-1977 and from 3.00×10^{-5} m/day to 3.00×10^{-4} m/day after 1978.

Evapotranspiration

The effects of plant transpiration and evaporation of ground water is simulated by MODFLOW based on three main assumptions (McDonald and Harbaugh, 1988):

a) The evapotranspiration loss occurs at a maximum rate, considered constant, when the hydraulic head in the aquifer is above a certain elevation. The maximum evapotranspiration loss, expressed as volume of water per unit surface area per unit time, is called EVTR and the elevation above which it applies is called SURF;

b) The evapotranspiration loss is zero when the hydraulic head in the aquifer is below a certain elevation called extinction depth (EXDP); and

c) The evapotranspiration loss varies linearly with the hydraulic head when it is between EXDP and SURF.

An evapotranspiration rate of 4.93×10^{-3} m/day, equivalent to the average evapotranspiration measured at El Sisal station (INDRHI, 1988), was assumed constant for the modeled area. The evapotranspiration surface (SURF) was also assumed constant and approximately with the same nodal elevations as the land surface (taken from the topographic map scale 1:50000). The extinction depth was taken as 1.50 m for the whole modeled area.

Pumpage

Steady-State Pumpage

Pumping data reported by Tahal (1971) were used for the steady state simulation. As shown in Tables 3, sixty six wells contained in 24 active model cells were operating in 1965. Total pumpage was about 20×10^{6} cubic meters. The wells were concentrated in the following areas:

a) Las Clavellinas-La Estancia (rows 2-7, columns 12-16), with 75% of the pumpage;

b) El Rosario (rows 8-11, columns 10-11), with 16% of the pumpage; and

c) Finca-Escuela (row 12, column 4) with 9% of the pumpage.

Transient Pumpage

The transient simulation was divided in two stages:

1. Verification, which was based on reported pumpage and head measurements. Three different periods, each subdivided in stress periods of one year, were used:
   a) From 1966 to 1971. Pumpage increased from 27×10^6 m^3 in 1966 to 47×10^6 m^3 in 1971 (Tahal, 1971).
   b) From 1972 to 1977. Pumpage was increased from 52×10^6 m^3 per year to 70×10^6 m^3 per year and extended to 55 active grid cells, the most severe stress imposed to the Azua aquifer. INDRHI-Tahal (1983) and INDRHI-SNC (1984).
   c) From 1978 to 1988. Pumpage was reduced to about 30×10^6 m^3 per year in 1983 and increased up to about 37×10^6 in 1988. The Yaque del Sur-Azua canal started operation in 1978 and extensive lowland areas were flooded by uncontrolled flowing wells. Hydraulic heads were reported by INDRHI-Tahal (1983) and INDRHI-SNC (1984) and measured by the author in 1988.

2. Prediction, which was based on the seven pumping scenarios described in the section regarding transient simulation.

STEADY STATE CALIBRATION

The steady-state calibration was based on the data developed from the work of Gilboa (1965) and Tahal (1971). The calibration process was conducted according to the following steps:

Step 1: The boundary conditions were defined. The hydraulic heads were set to constant values at the boundaries where inflow to and outflow from the aquifer was assumed to occur (Tahal, 1971). These hydraulic head values were taken from the ground-water contour map of 1965. There were 38 constant head nodes, 159 active nodes, and 75 inactive nodes.

Step 2: An initial run was made and the hydraulic head values generated by MODFLOW were compared to the nodal values of 1965. As expected, the results were not satisfactory and so began the task of adjusting the initial input data, especially transmissivity, until the differences between the water-level elevations of 1965 and the water-level elevations generated were minimized. A value of 2 m was adopted as an acceptable difference because it was assumed to represent the magnitude of the errors when initial heads were averaged for the model blocks. Five additional runs were necessary to reproduce the initial head values.

Step 3: The constant head nodes were replaced by constant flux nodes for verifying the results of step 2. Minor adjustments to the initial parameters were required, especially along the outflow boundary.

Results of the Steady State Calibration

Figures 8 and 9 show, respectively, the steady state hydraulic head configuration and the differences between measured and generated water-level elevations for 1965. The hydraulic heads replicate the hydraulic head map of 1965 and the differences comply with the 2 meter-error criterion. The largest errors occurred in the area northwest of Los Jobillos, where average hydraulic heads were obtained by extrapolation. Figure 10 shows a contour map of calibrated transmissivities. The flow rates through the constant head nodes are presented in Table 5.

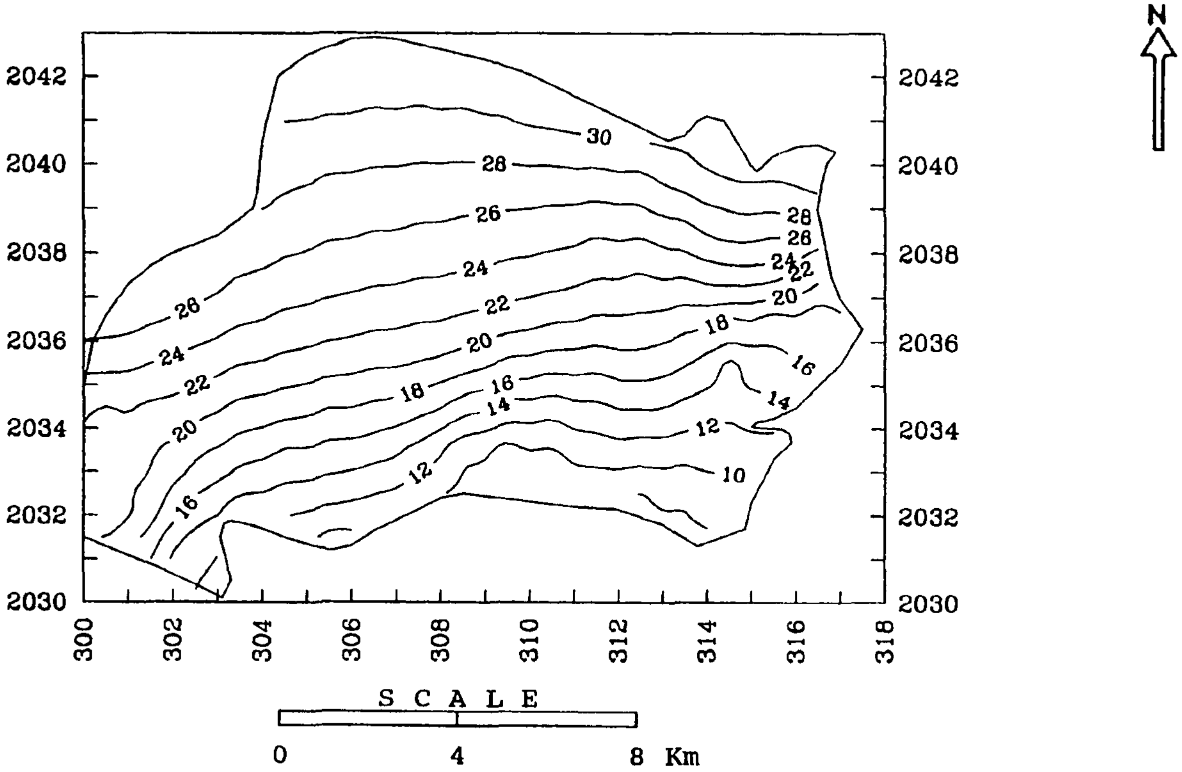

Figure 8 Contour map of simulated steady-state hydraulic heads (m).

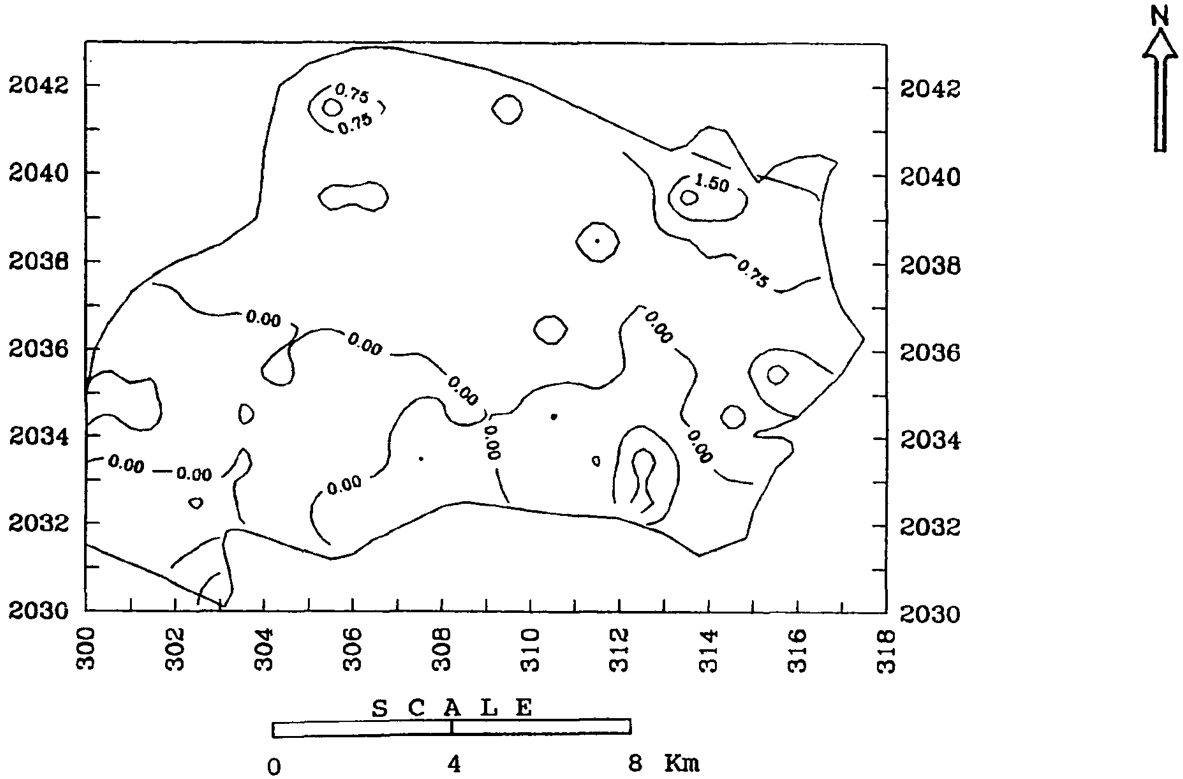

Figure 9 Differences between measured and simulated steady-state heads (m).

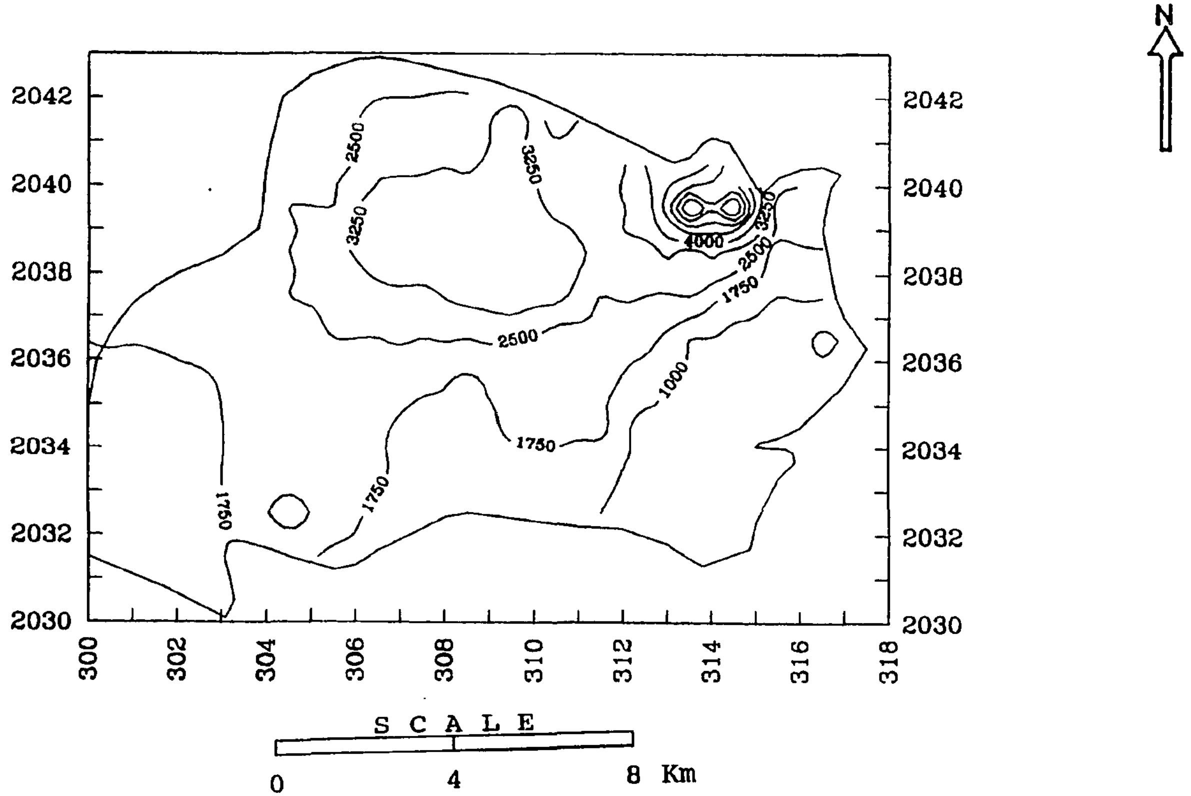

Figure 10 Contour map of steady-state calibrated transmissivities (m**2/day)

The water balance indicated a discrepancy of approximately 0.005 % between total inflow and total outflow. This indicates that the numerical errors associated with the finite-difference solution have only a minor effect and the model is simulating equilibrium conditions with good precision. The volumes of inflow and outflow were as follows:

INFLOW (in millions of cubic meters per year)	
Inflow through constant head boundary	38.049
Local recharge (mainly from irrigation)	2.938
River leakage	3.411

Total in	44.398
OUTFLOW (in millions of cubic meters per year)	
Outflow through constant head boundary	13.686
Pumpage	19.090
Drainage	0.258
Evapotranspiration	11.225
River leakage	0.137

Total out	44.396
Percent discrepancy	0.0045

Transient Solution

The transient solution was divided in two stages: verification and prediction. The verification stage consisted of three sets of runs, six years long each, designed for testing the parameters and assumptions used in the calibrated model. The prediction stage consisted of seven runs based on the calibrated parameters and the pumping projections of seven different scenarios.

Boundary Conditions

For all of the transient simulation runs the constant head boundary conditions used for the steady-state simulation were replaced by constant flux boundaries. The procedure was performed by introducing artificial recharging wells along the inflow boundary nodes and artificial pumping wells along the outflow boundary nodes. The rates of inflow to and outflow from the aquifer were considered constant and equal to the flow rates computed in the steady-state calibration (Table 5). Notice that 75% of the total inflow occurs through the northern boundary of the aquifer, with 55% coming from the alluvial fans of the Jura and Irabon rivers and 20% from the old alluvial deposits around the highway to San Juan City. The remaining 25% gets into the aquifer through the western boundary, especially from alluvial deposits and alluvial fans of Lajas, Galindo, Yarey and Copey.

Table 5 Computed flow rates through the constant head nodes at the end of the steady-state calibration

BOUNDARY NODE		INFLOW		OUTFLOW		% OF
ROW	COLUMN	m^3/day	MCM/year	m^3/day	MCM/year	TOTAL
1	5	0	0.00			0.00
1	6	2870	1.05			2.76
1	7	4086	1.49			3.93
1	8	4224	1.54			4.05
1	9	3990	1.46			3.84
1	10	4852	1.77			4.65
2	5	870	0.32			0.85
2	11	9873	3.60			9.47
2	12	6326	2.31			6.07
2	13	8550	3.12			8.20
2	14	4233	1.55			4.08
2	15	0	0.00			0.00
3	5	1908	0.70			1.84
3	15	24010	8.76			23.03
3	16	4783	1.75			4.59
3	17	0	0.00			0.00
4	4	2928	1.07			2.84
5	3	4795	1.75			4.60
5	4	4478	1.63			4.30
6	1	0	0.00			0.00
6	2	5368	1.96			5.15
7	1	1462	0.53			1.45
8	1	482	0.18			0.46
9	1	322	0.12			0.34
10	1	1285	0.47			1.25
11	1	2221	0.81			2.13
11	8			4302	1.57	11.55
11	9			1294	0.47	3.53
11	10			888	0.32	2.45
11	11			1582	0.58	4.30
11	12			2603	0.95	7.02
12	4			8404	3.07	22.49
12	5			6755	2.47	18.09
12	6			6076	2.22	16.29
12	7			4298	1.57	11.55
12	13			496	0.18	1.45
12	14			473	0.17	1.38
13	4			0	0.00	0.00
TOTALS (*)		**104244**	**38.05**	**37497**	**13.69**	

1MCM/year = 10^6 m^3/year

Pumping Scenarios

Pumping projections were based on the recommendations of INDRHI-SNC (1984) and Gurevitz (1985) for increasing irrigated agriculture in the Azua valley in about 4500 additional hectares. The development of the additional lands will be based on pumpage from existing wells and wells to be constructed. Presently, 15-20 wells are in operation and 52 wells are proposed for rehabilitation. The new wells will be in the vicinity of the areas to be developed.

In addition to the 3 pumping scenarios used for the verification process, 7 different scenarios were projected up to 22 years into the future. These 7 scenarios were as follows:

Scenario 1A: pumping pattern of 1988 was maintained at 37 MCM/year and at the same 22 model nodes; no new wells were allowed;

Scenario 1: assumed that all of the existing wells were rehabilitated and operating 24 hours a day; total pumpage was 169 MCM/year at 43 model nodes;

Scenario 2: same as scenario 1 with a volume of artificial recharge of 30 MCM/year through the northern and northwestern boundary nodes;

Scenario 3: same pumping sites as scenario 1, with rehabilitated wells operating 12 hours a day; total pumpage was 103 MCM/year;

Scenario 4: assumed that rehabilitated wells were added to the pumping pattern of 1988 only in the area between rows 2 to 9 and columns 11 to 16; total pumpage was 104 MCM/year at 26 model nodes;

Scenario 5: assumed pumping pattern of 1988 with new and rehabilitated wells in the western area limited by rows 4 to 12 and columns 1 to 10; total pumpage was 102 MCM/year at 39 model nodes; and

Scenario 6: assumed pumping pattern of 1988 with the addition of new and rehabilitated wells in the northwestern area between rows 3 to 8 and columns 6 to 12; total pumpage was 102 MCM/year in 35 model nodes.

Results of the Transient Simulations

A. Verification

Three sets of transient runs were performed for calibrating the storage coefficient and for testing the response of the calibrated model to the stresses imposed on the aquifer during the periods 1966-1971, 1972-1977, and 1978-1988. The nodal transmissivity values for the three periods and the starting head values for the first period were the calibrated values of the steady-state simulation. Final heads at the end of each period were used as starting heads for the next period.

The first set of runs corresponded to the period 1966-1971 and was used for calibrating the storage coefficient of the aquifer. Four runs were necessary until the absolute difference between observed and simulated hydraulic heads in 1971 was less than 2 meters for all of the active nodes. The value of the storage coefficient giving the best fit to this condition was 0.10 and this was the value used for all of the transient runs. The average drawdown of the aquifer was 5.45 m for the entire period (0.90 m per year). The final hydraulic head configuration for 1971 is shown in Figure 11.

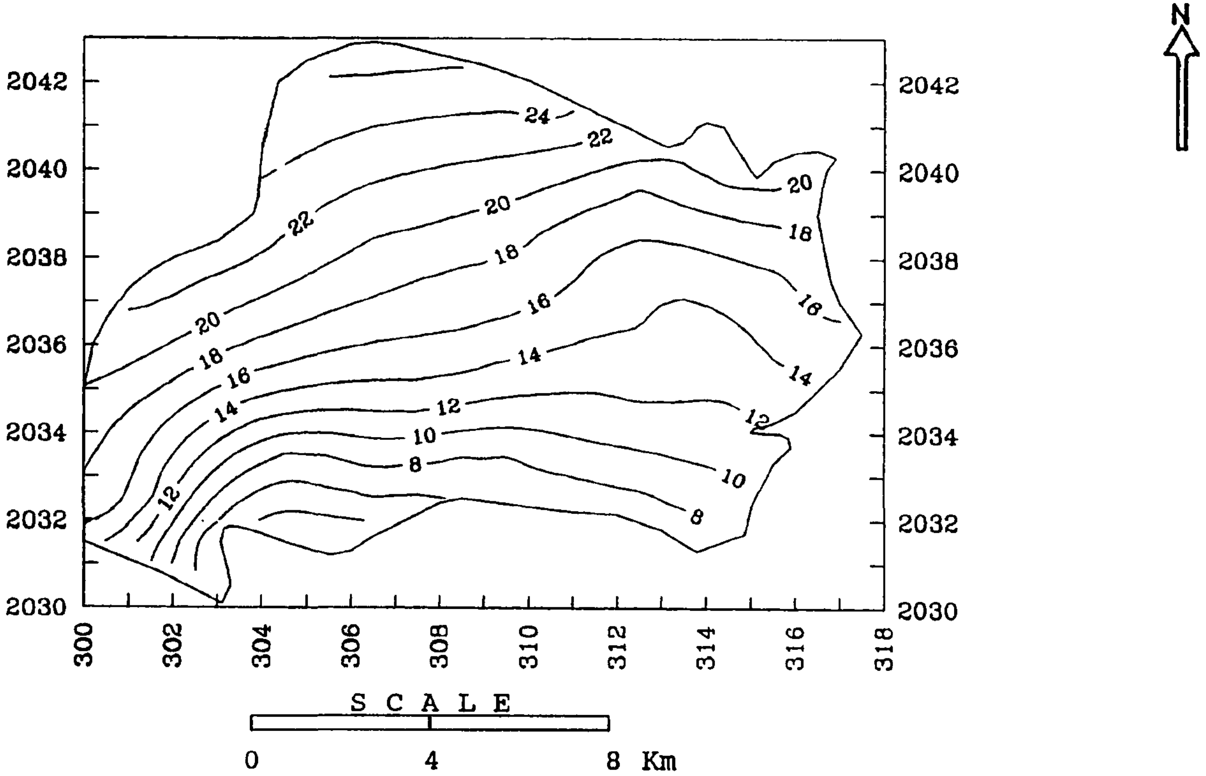

Figure 11 Contour map of simulated hydraulic heads in 1971 (m).

Two runs were required for the period 1972-1977. The only difference between the two runs was the distribution and amount of pumpage for each year between 1975 and 1977. The first run was performed by assigning the new pumping data reported by INDRHI-SNC (1984) to the model nodes (Table 2). The difference between observed and simulated head values was up to 6 m in the Azua-Las Clavellinas area. After a careful revision of the reported pumping data and the pumpage assigned to the nodes in that area, two main errors were found. The first error was in the computation of pumpage and the second error was in the location of wells. By assigning the corrected pumpage to the right nodes, the second run was performed and a reasonable fit between measured and simulated hydraulic heads (difference less than 2 m) was attained. The average drawdown for the period was 7.76 m or 1.30 m per year. An additional 12-year run, starting in 1966 and ending in 1977, resulted in the same head configuration at the end of 1977. The average drawdown was about 1.10 m per year and the maximum drawdown at the end of 1977 was 22.50 m (2 km south of Azua City). As shown in Figure 12, hydraulic heads in 1977 varied from about sea level in the vicinity of Los Negros to 13 m above sea level at the northwestern boundary of the modeled area.

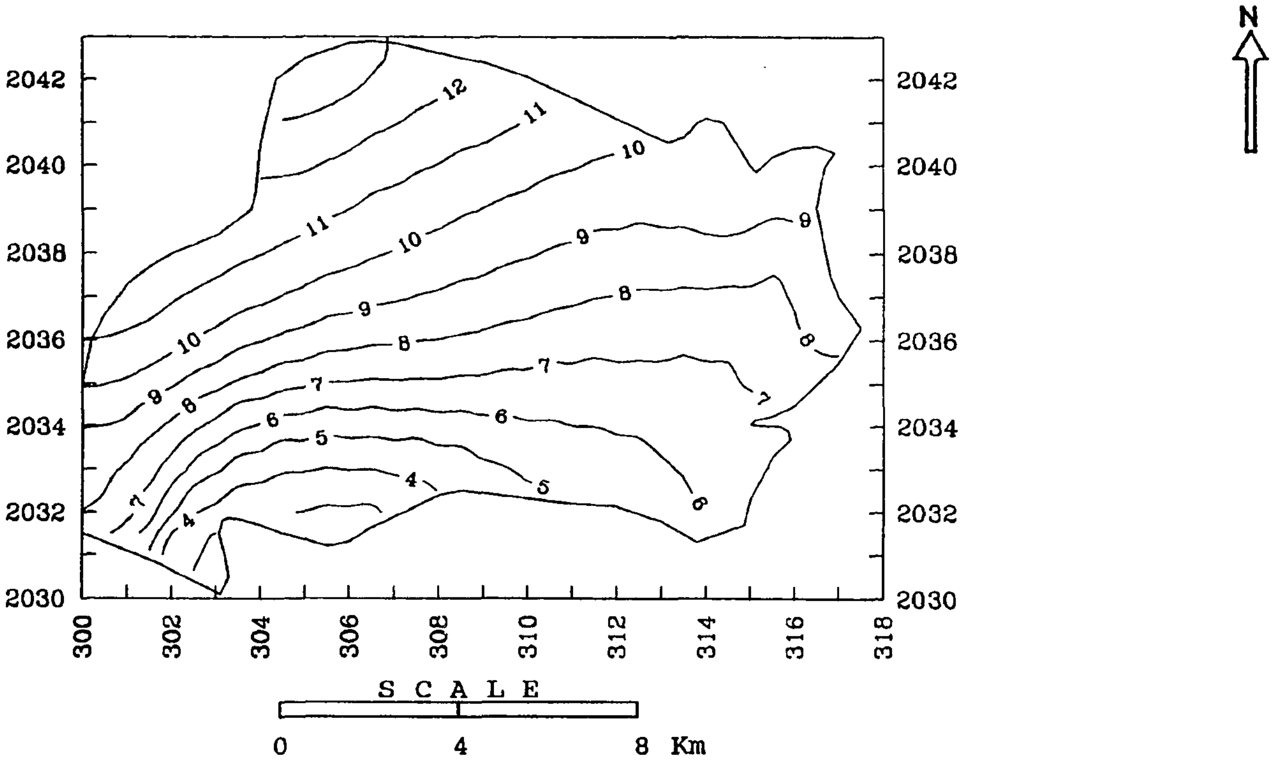

Figure 12 Contour map of simulated hydraulic heads in 1977 (m).

Even though the orientation of the equipotential lines was more or less the same in 1971 and in 1977, a cone of depression started to develop in the area between Azua City and Las Clavellinas at the end of 1977. This is the result of the high concentration of pumpage in the area during the year of maximum stress imposed on the aquifer. Cumulative drawdowns from 1965 to 1977 are shown in Figure 13.

For the period 1978-1988 three runs were needed. With the exception of the Well Package, all of the packages of MODFLOW were maintained as in 1977 for the first run. Pumpage decreased from 70 MCM in 1977 to about 30 MCM in 1988. Inasmuch as the Yaque del Sur Azua canal started operation in 1978 and the network of drainage canals was not sufficient for controlling very large amounts of irrigation returns, local recharge increased to larger amounts than in 1977. In the same way, the number of river-canal reaches increased from 15 in 1977 to 31 in 1988 and the number of drains increased from 7 in 1977 to 33 in 1988. Figures 14 and 15 show, respectively, the hydraulic head configuration for 1983 and 1988. The ground-water contours for both years are almost the same and the slight difference between them (average 0.12 m, maximum 2.14 m) is, mainly, the result of the drainage canals whose construction started in 1983 and continued in 1988 (Figure 16). The effects of extraordinary rainfalls, associated with hurricanes and tropical storms in 1979 and in 1987, were accounted for through the recharge package and the river package. Figures 17 and 18 present, respectively, the behavior of hydraulic heads for the 17 nodes along row number 7 and for the 11 nodes along column 10. Curves are for years 1965, 1971, 1977, and 1988. Maps showing the rise of the potentiometric surface for 1977-1983 and depth-to-water in 1988 are presented in Figures 19 and 20, respectively.

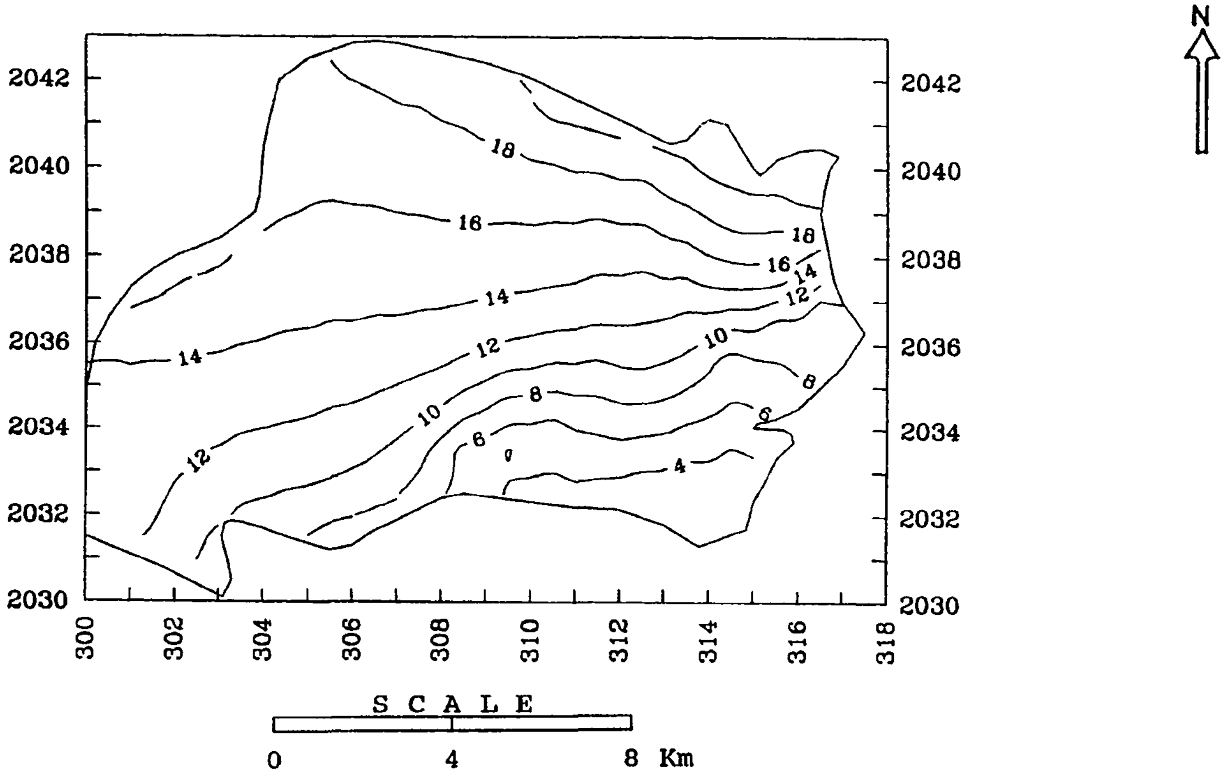

Figure 13 Contour map of cumulative drawdown from 1965 to 1977 (m).

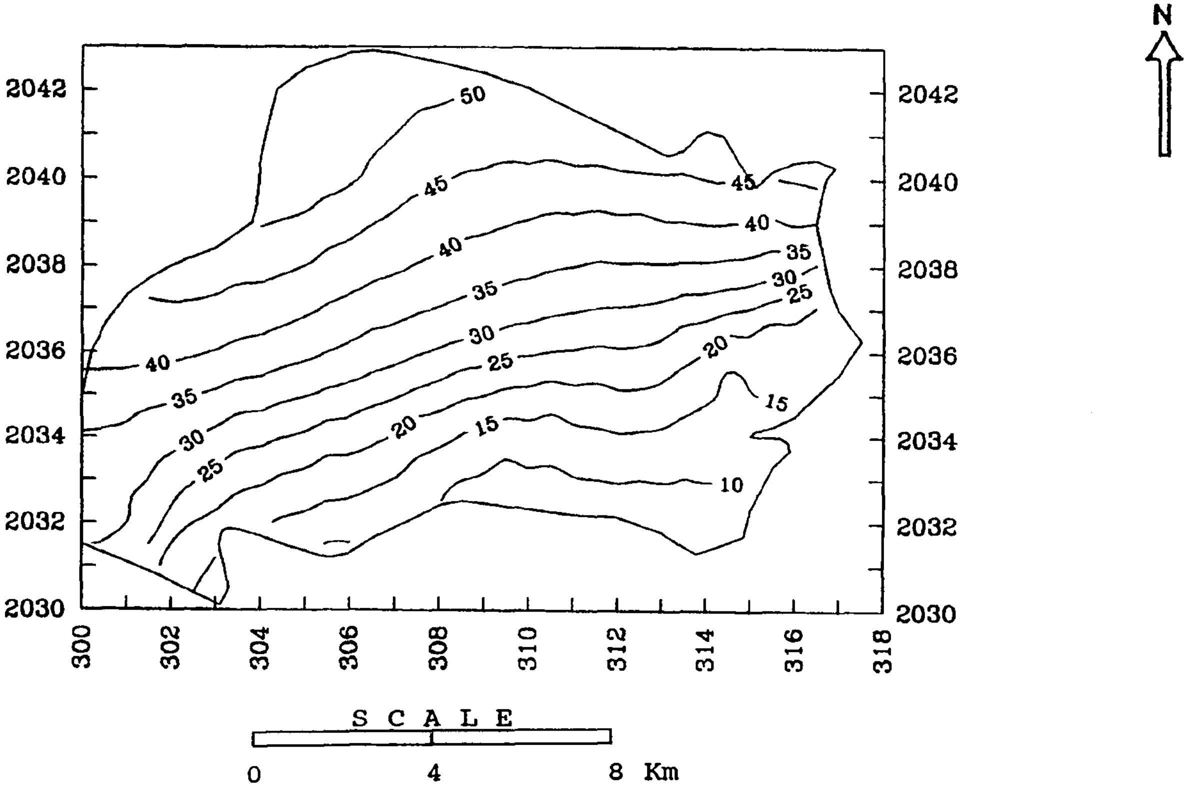

Figure 14 Contour map of simulated hydraulic heads in 1983 (m).

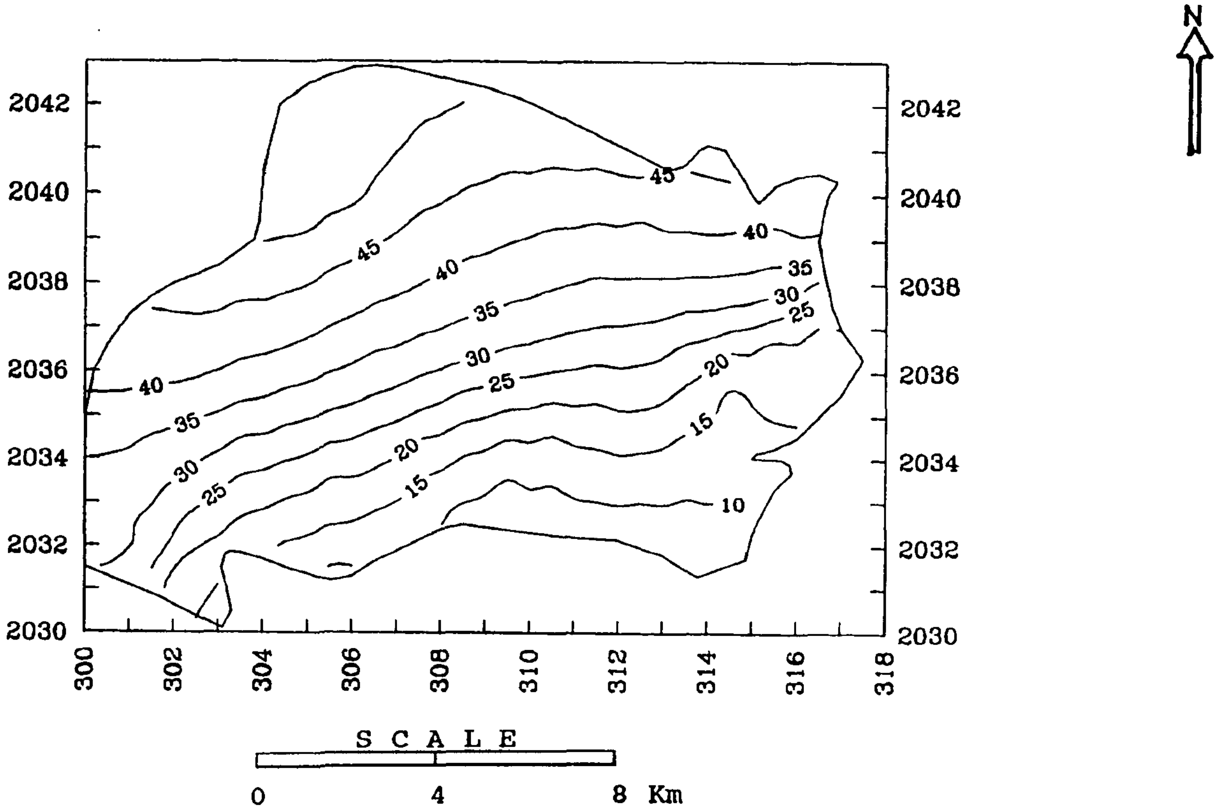

Figure 15 Contour map of simulated hydraulic heads in 1988 (m).

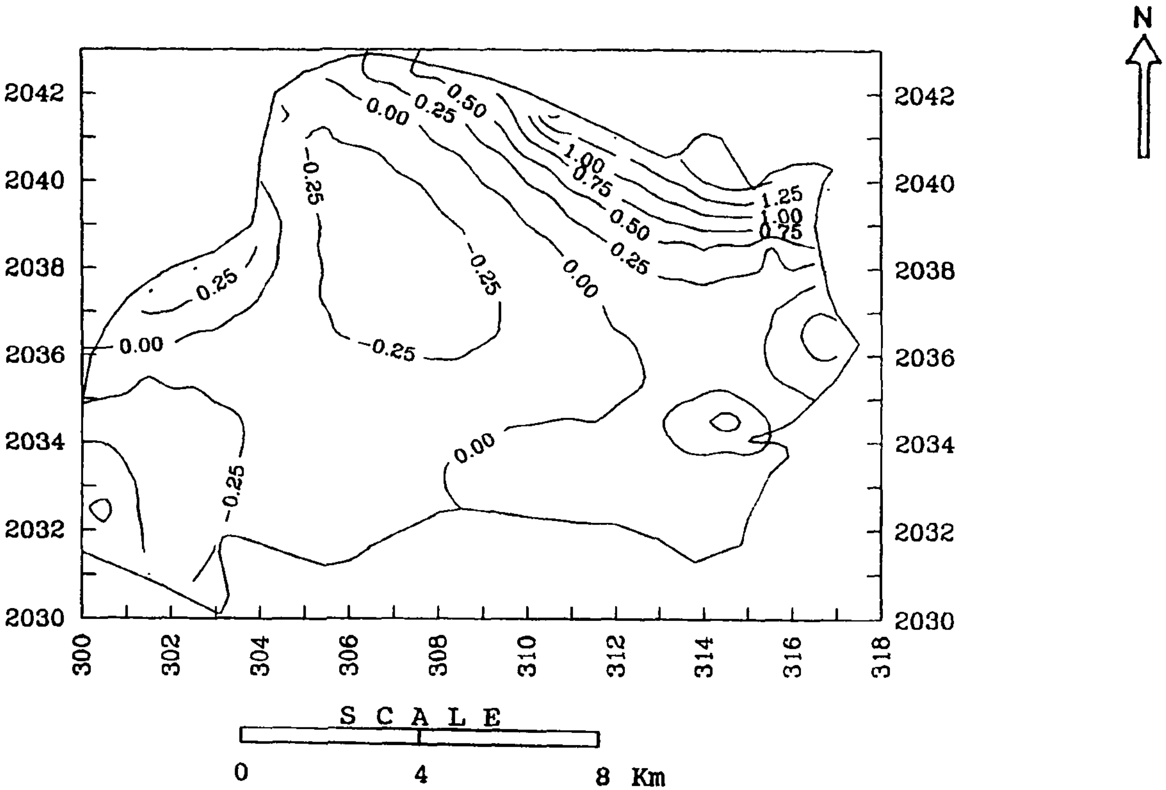

Figure 16 Contour map of water-level change due to drains in 1983-1988 (m).

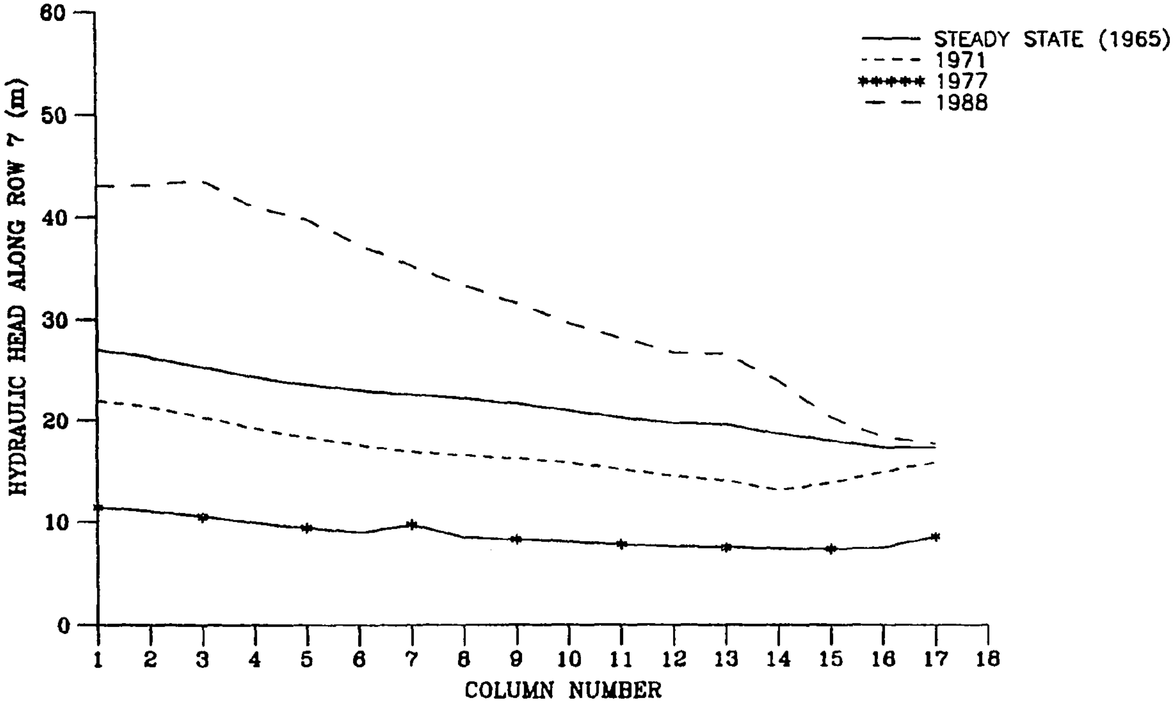

Figure 17 Variation of measured hydraulic heads along row number 7 for the period 1965–1988

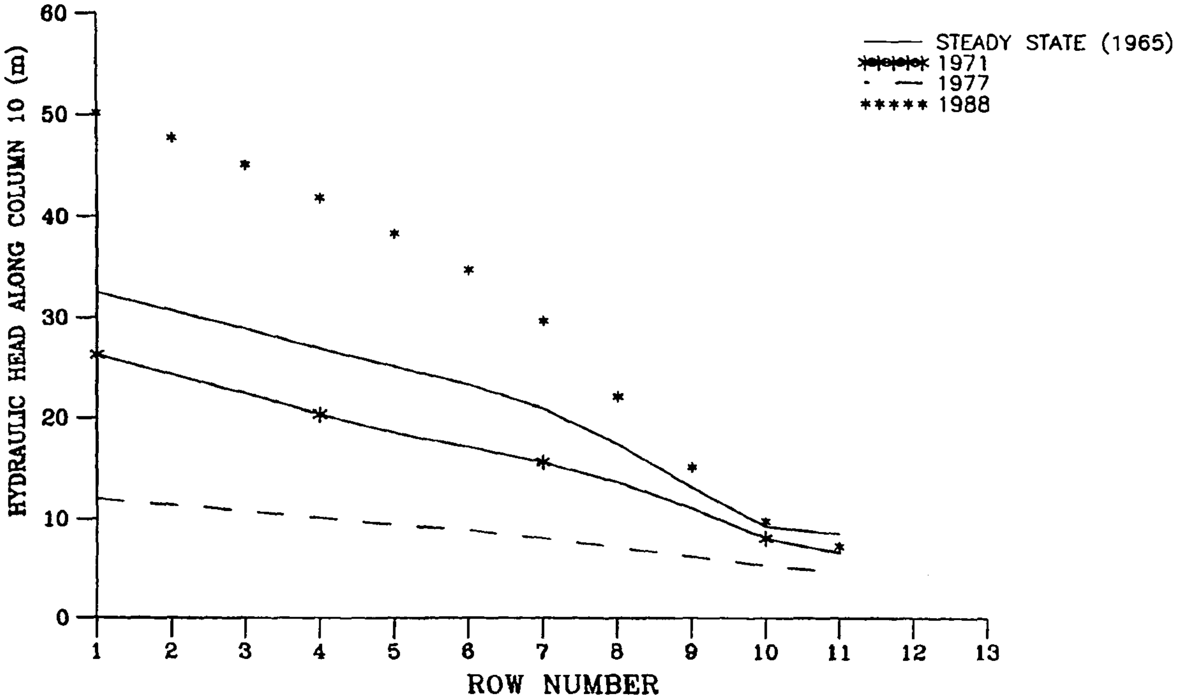

Figure 18 Variation of measured hydraulic heads along column number 10 for the period 1965–1988

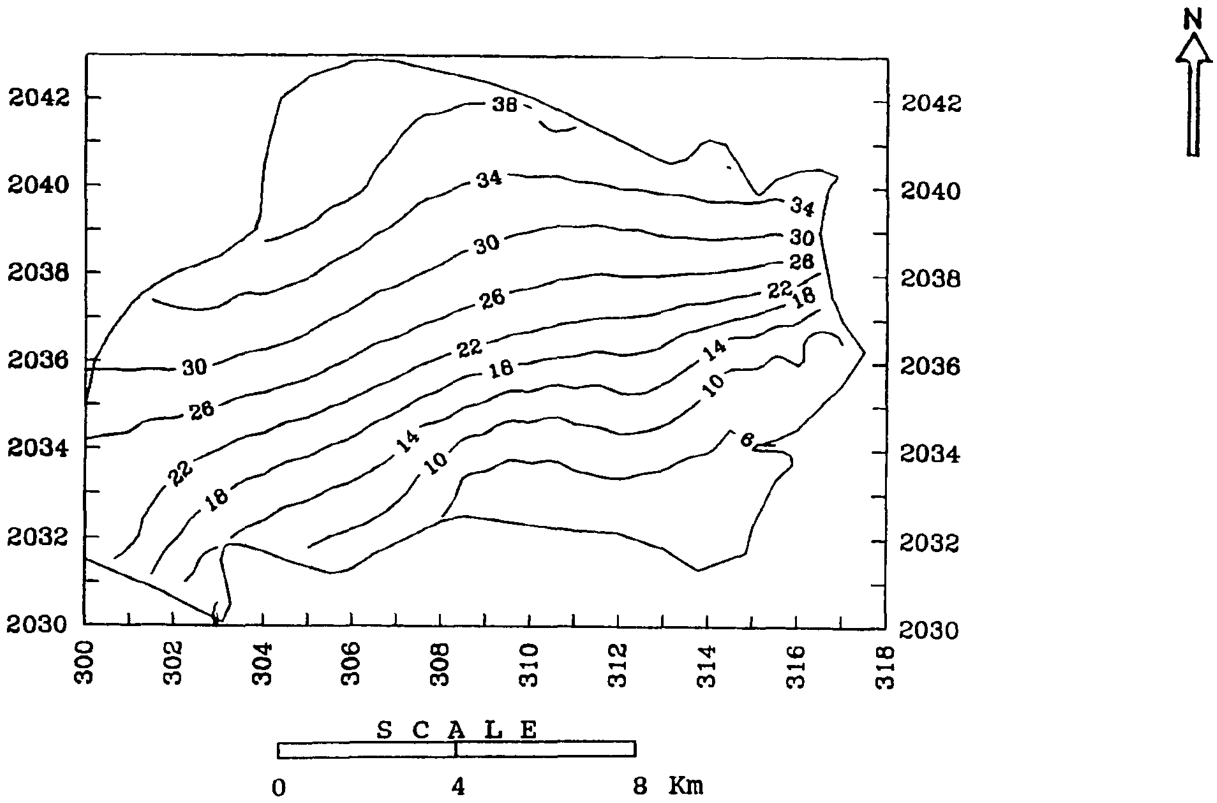

Figure 19 Contour map of water–level rise between 1977 and 1983 (m).

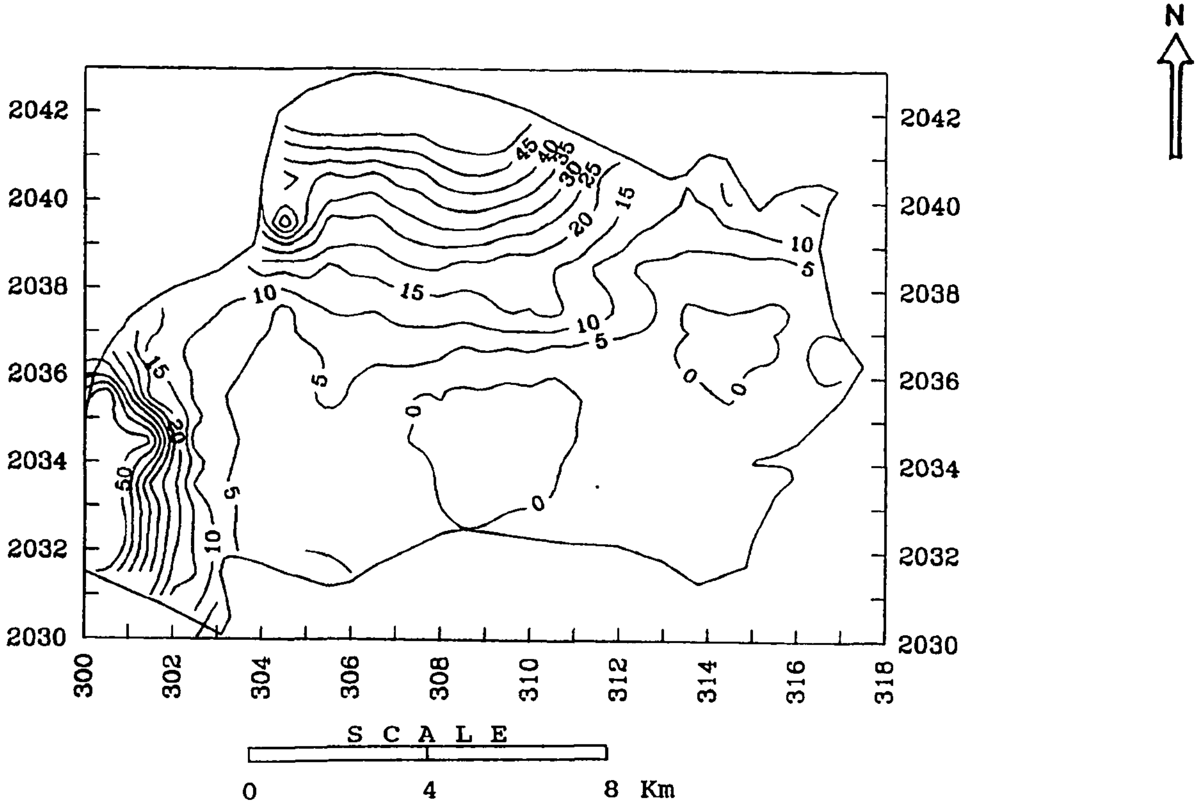

Figure 20 Contour map showing depth to water in 1988 (m).

B. Prediction

Based on the final head values of 1988 and the seven pumping scenarios described above, transient simulations were performed for 12 years and 22 years. The outputs of these simulations consisted of distributions of heads and drawdowns and water budgets for years 1990, 1995, 2000, 2005 and 2010. Contours of hydraulic head distributions and graphical representations of drawdown and hydraulic head along selected rows and columns were produced.

Scenario 1A:

This scenario modeled the pumpage of 37 MCM/year for a period of 22 years, assuming that the situation of 1988 is maintained without change.

As shown in Figure 21, the hydraulic head distribution for year 2010 has little change with respect to 1988. This means that if the situation of 1988 is maintained, without improving the efficiency of the irrigation system and without using larger amounts of ground water for irrigation, the drainage situation in the lowland areas will continue the same.

Scenario 1:

This scenario modeled a constant pumpage of 169 MCM/year up to year 2000, under the assumption that all of the existing wells entered in operation in 1989 (Gurevitz, 1985). Recharge from irrigation returns was reduced in 17% and, conversely, the number of drains was increased in 15%. The number of river-canals was maintained as in 1988, but their contributions to the aquifer decreased about 19%.

As shown in Figure 22, the equipotential lines of the central and eastern portions of the aquifer have changed; their orientation is almost perpendicular to the equipotential lines of 1988. A large cone of depression, with heads up to 8 m.b.s.l. has started to develop in the area between Azua City and Los Tramojos. The Figure shows that the main component of the direction of flow, at year 2000, would be from the central and southeastern parts of the aquifer toward the Azua-Los Tramojos area.

Figures 23 and 24 show, respectively, the variations of hydraulic heads for the 17 nodes along row number 7 and for the 11 nodes along column 10. Curves are presented for years 1990, 1995, 2000, 2005, and 2010.

Scenario 2

Scenario 2 modeled a constant pumpage of 169 MCM/year up to year 2000. The only difference with scenario 1 is the application of a constant rate of artificial recharge equivalent to 80% of the natural inflow to the aquifer through its northern and northwestern boundaries.

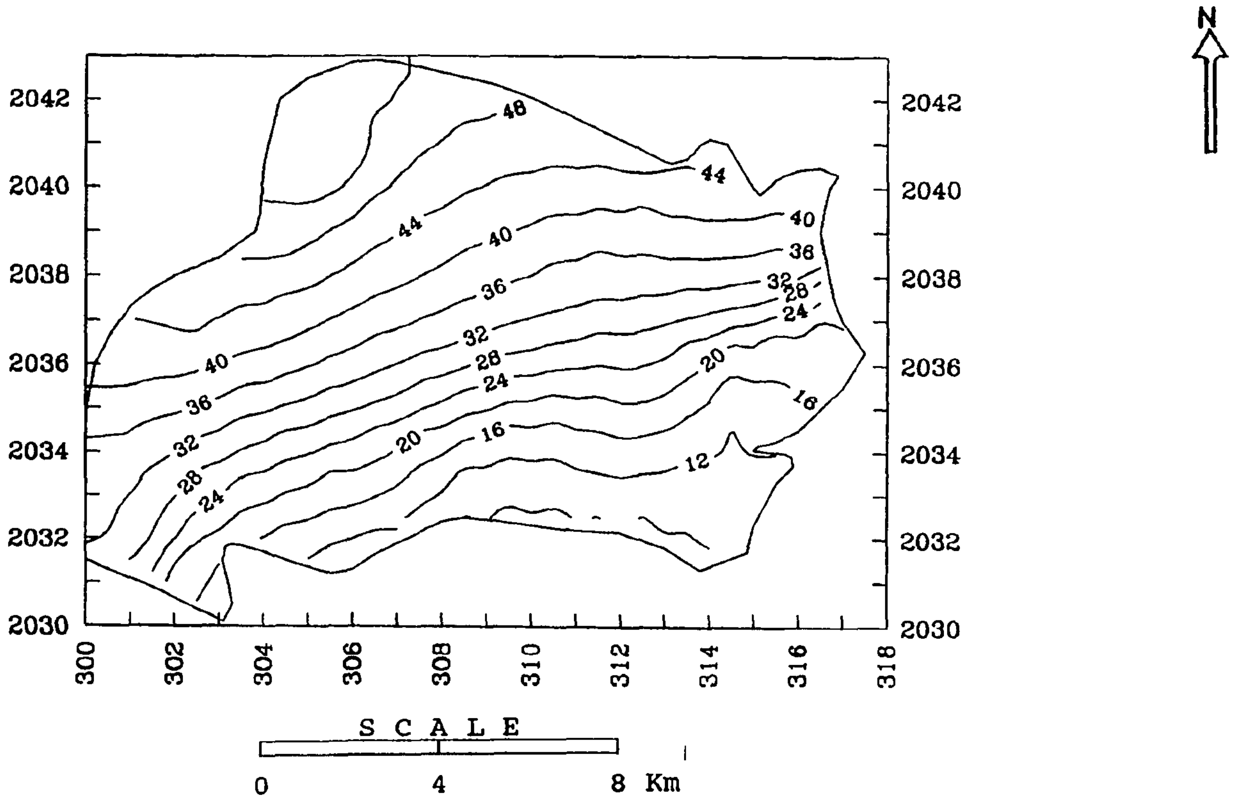

Figure 21 Contour map of hydraulic heads for scenario 1A – year 2010 (m).

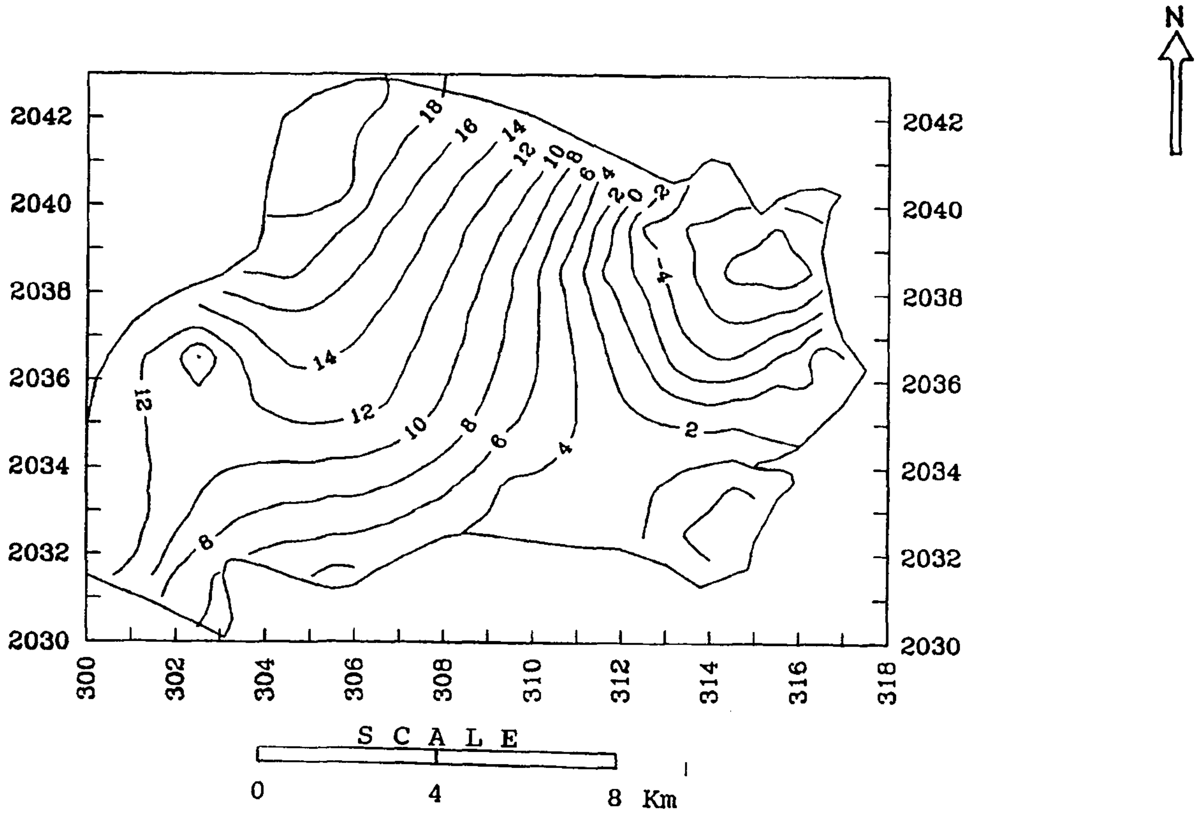

Figure 22 Contour map of simulated heads for scenario 1 by year 2000 (m).

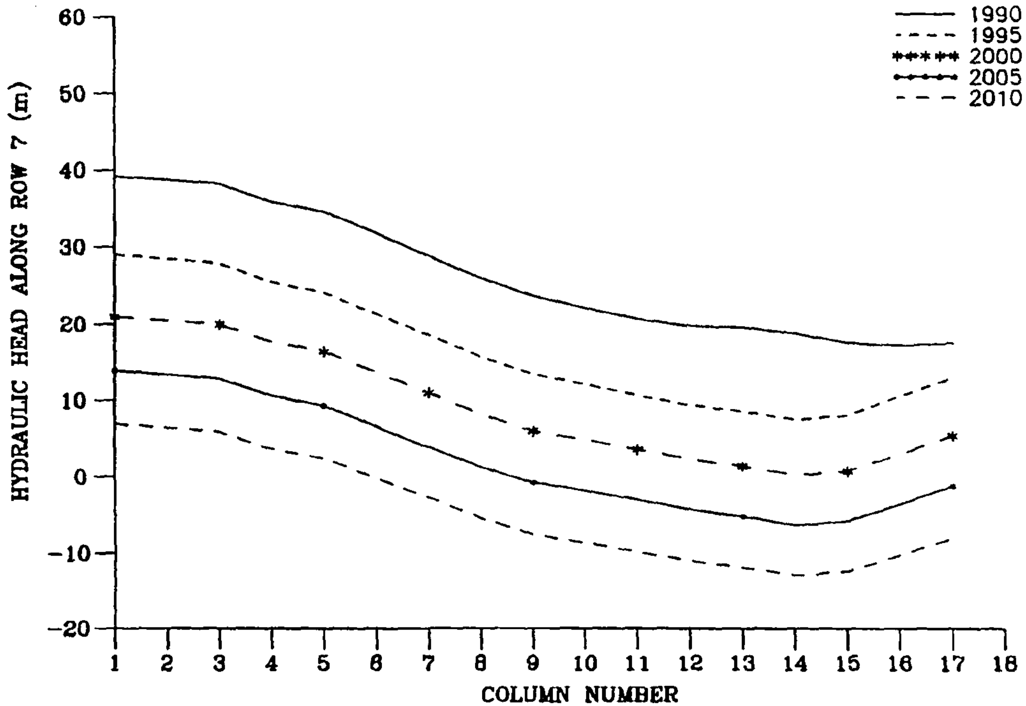

Figure 23 Variation of simulated hydraulic heads along row number 7 for scenario 1 (1990–2010)

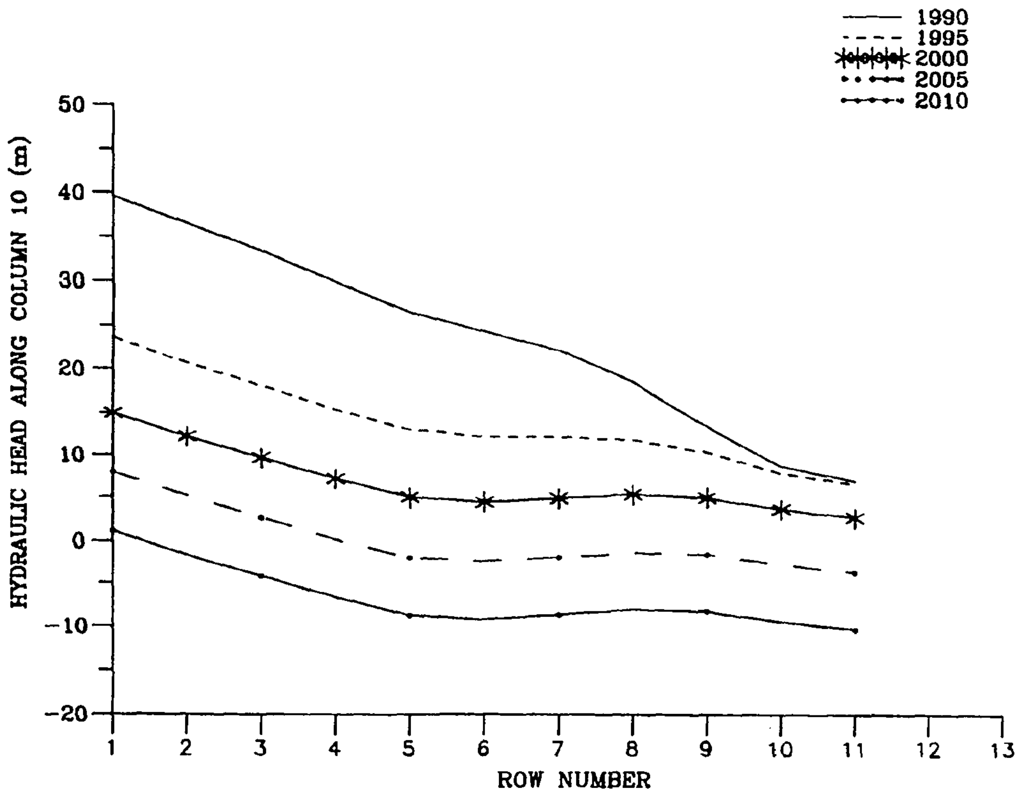

Figure 24 Variation of simulated hydraulic heads along column number 10 for scenario 1 (1990–2010)

As shown in Figure 25, the addition of 30 MCM/year as artificial recharge does not avoid the enlargement of the huge cone of depression identified in scenario 1 by year 2000. The cone of depression now extends to the entire aquifer and heads are below the mean sea level in all but 18 nodes of its northwestern boundary. All of the aquifer flow is now directed to the northeastern area of the aquifer between Azua and Los Tramojos. The reversal of the natural direction of ground-water flow, now from south to north, will, undoubtedly, allow large amounts of sea water to intrude into the aquifer thus causing extensive damage.

Scenario 3:

This scenario modeled a constant pumpage of 103 MCM/year up to year 2010. Wells were in same locations as in scenario 1, but rehabilitated wells were considered to be pumping at half of the rate reported by Gurevitz (1985). Contour maps of simulated hydraulic heads for 1990 and 1995 are presented in figures 26 and 27, respectively.

In Figure 26 the equipotential lines start to change their direction around the area Los Tramojos-Azua, and by year 1995 (Figure 27) a cone of depression is clearly defined. From year 2000 to 2010 the cone of depression is so large that most of the ground-water flow is directed toward the northeast corner of the modeled area. By year 2010 hydraulic heads are below the mean sea level in more than two thirds of the modeled area. By year 2005, the potentiometric surface reaches the lowest elevation of scenario 1 in year 2000 in the Azua-Los Tramojos area and along the southern boundary of the study area. At those locations, a potential risk of sea-water intrusion develops by year 2005.

The response of river leakage to the pumping in this scenario is to decrease in about 0.7 MCM/year with respect to scenario 1. Conversely, drain discharge increases by about 2 MCM/year.

Scenario 4:

Scenario 4 modeled a constant pumpage of 104 MCM/year composed of the pumping pattern of 1988 and the pumping from wells proposed by Gurevitz in the area between rows 2 to 9 and columns 11 to 16. Figure 28 shows the hydraulic head configuration at year 2010. The ground-water flow is toward the Azua-Los Tramojos area and the potentiometric surface at the eastern two thirds of the aquifer is generally below sea level.

Although the amount of pumpage for this scenario is about the same of scenario 3, the amount of drainage discharge was reduced in about 55% and the volume of river-canals leakage was approximately the same.

Scenario 5:

Scenario 5 modeled a constant pumpage of 102 MCM/year made of the pumping pattern of 1988 and pumping from wells proposed by Gurevitz (1985) in the area between rows 4 to 12 and columns 1 to 10. Hydraulic heads for year 2010 are presented in Figure 29. The equipotential lines are now below the mean sea level in the central and southwestern areas of the aquifer, where new pumpage has been developed. Head varies from a minimum of 32 m.b.s.l. at Los Negros, where ground-water flow is directed, to 4 m.a.s.l. at the northern boundary of the aquifer. In terms of risk of sea water intrusion, the situation depicted by Figure 29 is as critical as the situations created by scenarios 1 to 4.

The amount of river leakage for this scenario was about the same as in scenario 4, whereas the volume of discharge to the drainage system increased in 70%.

Scenario 6:

In this scenario a constant pumping of about 102 MCM/year was modeled. Pumping centers were located at the same nodes as in 1988 and in the area between rows 3 to 8 and columns 6 to 12 (Gurevitz, 1985). The simulated hydraulic heads for year 2010 are presented in Figure 30. As in the other scenarios, a large cone of depression is developed. The potentiometric surface is lowest in the area of La Estancia, where it reaches 16 m.b.s.l. and where most of the ground-water flow is directed.

The amount of river leakage and drain discharge were, respectively, 3 times and 1.5 times larger than in scenario 5.

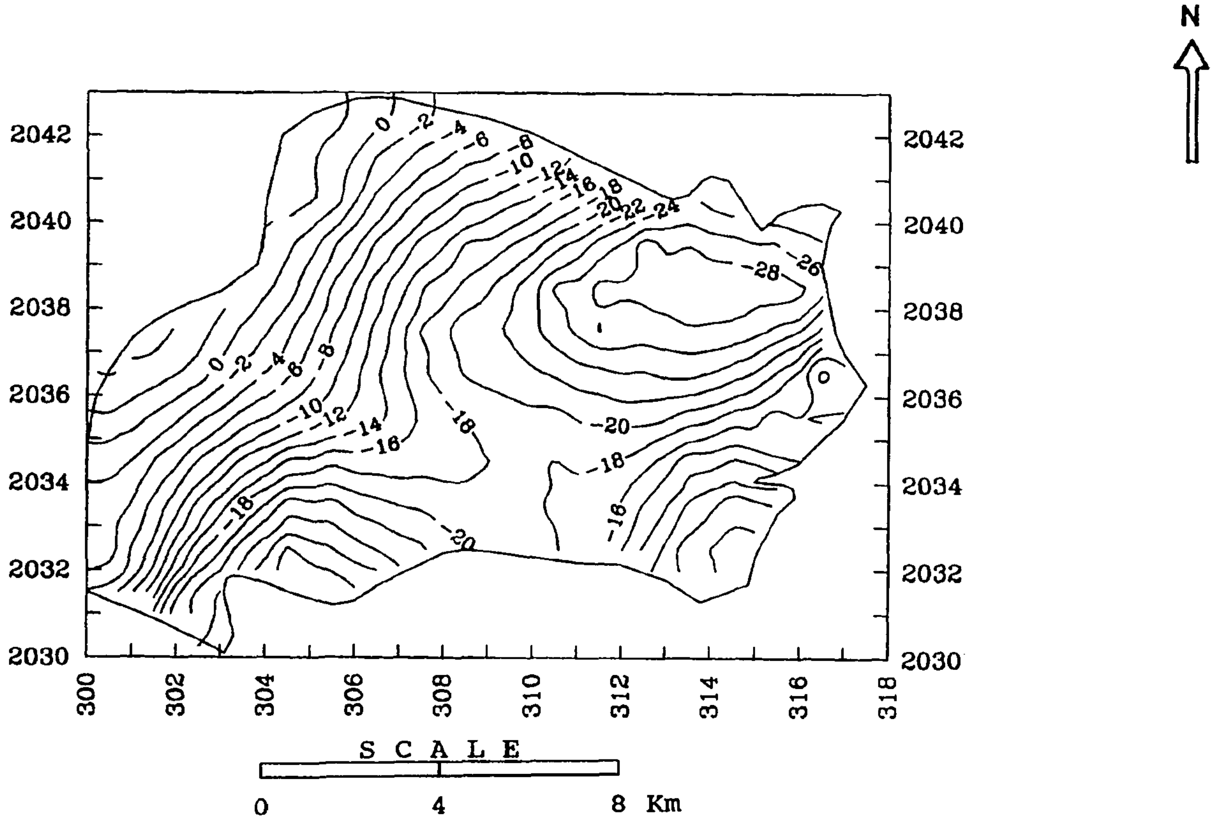

Figure 25 Contour map of simulated heads for scenario 2 by year 2010 (m).

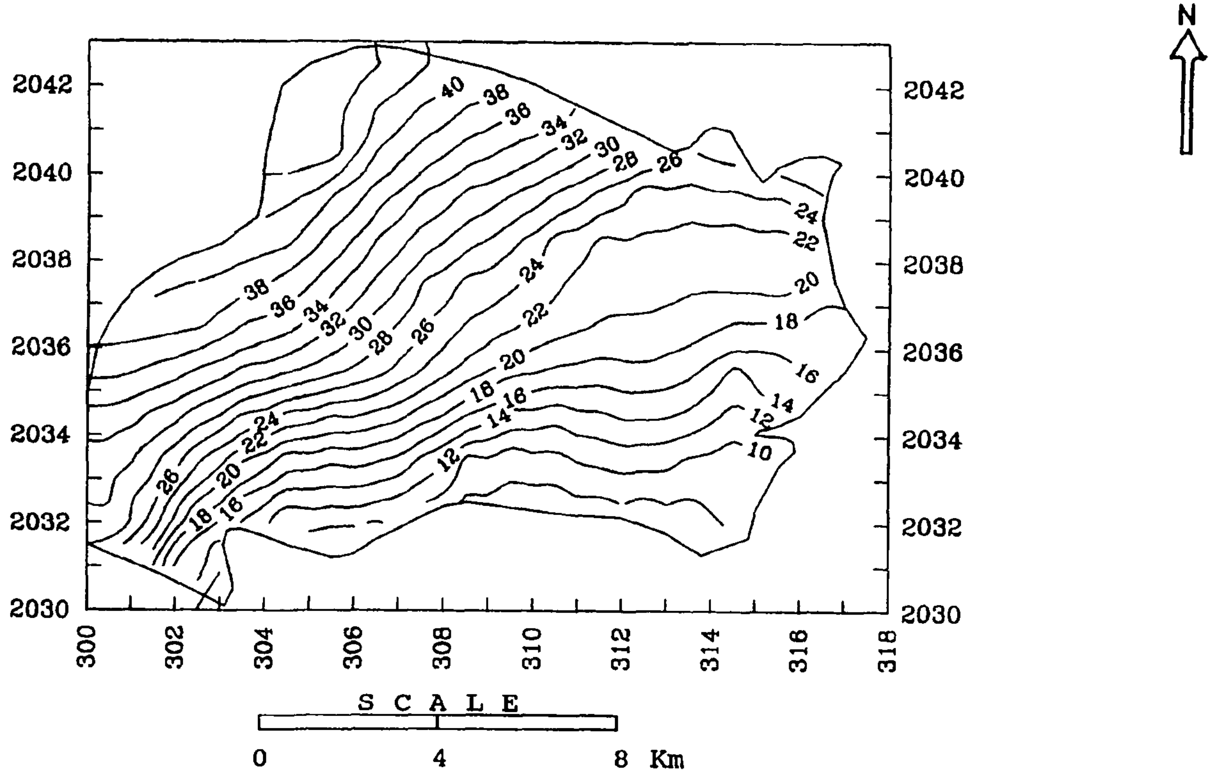

Figure 26 Contour map of heads for scenario 3 by year 1990 (m).

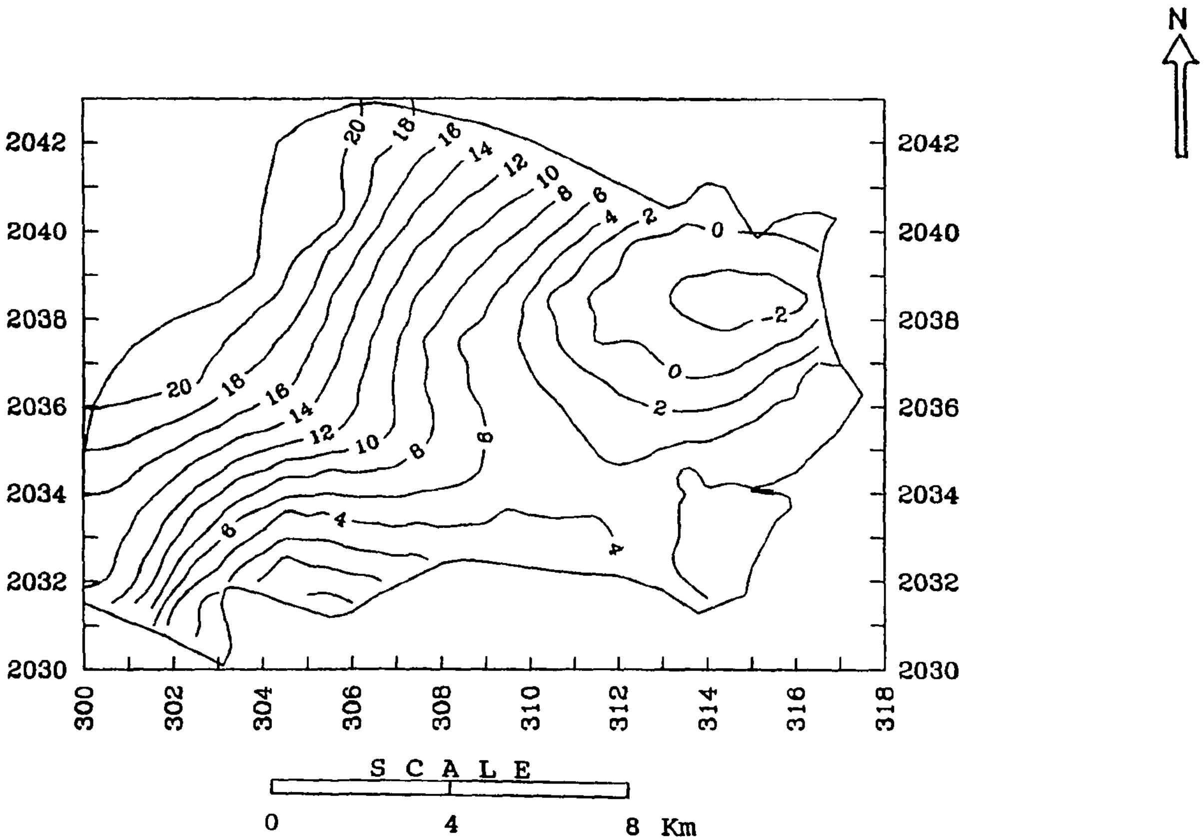

Figure 27 Contour map of heads for scenario 3 by year 2000 (m).

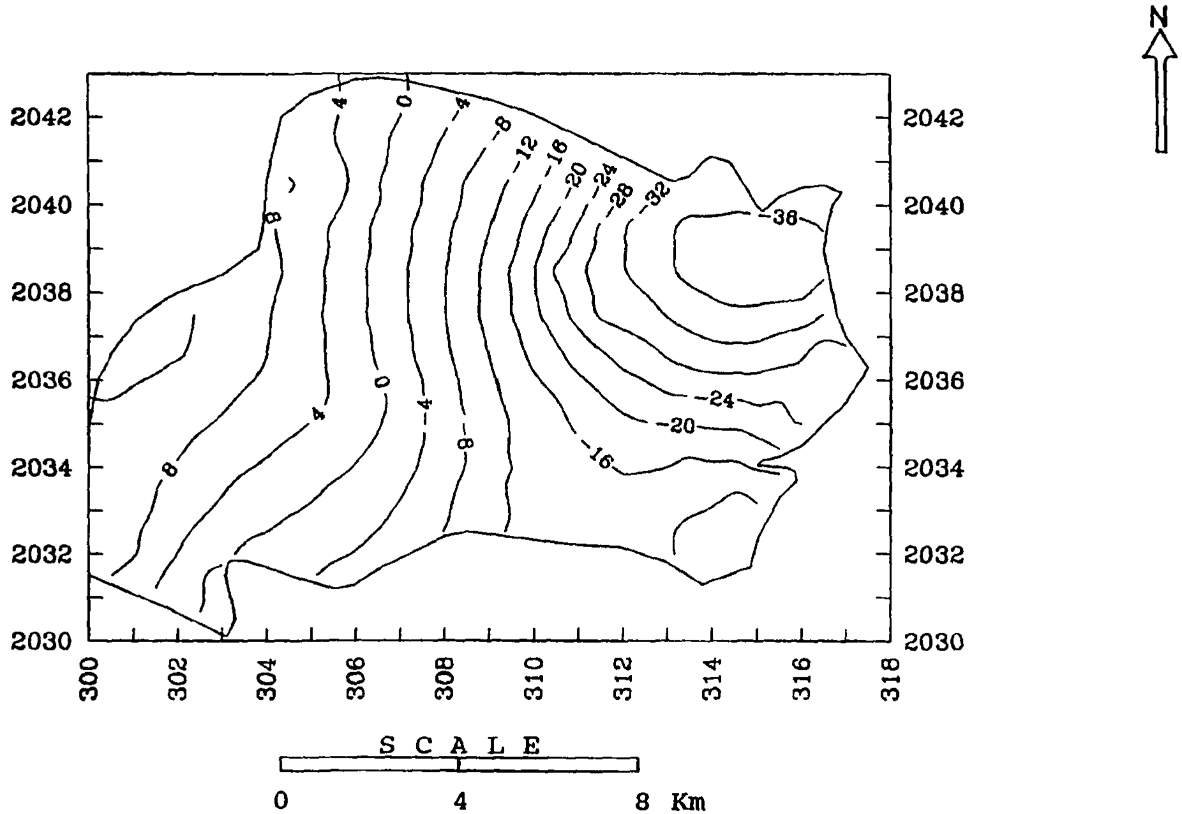

Figure 28 Contour map of simulated heads for scenario 4 by year 2010 (m).

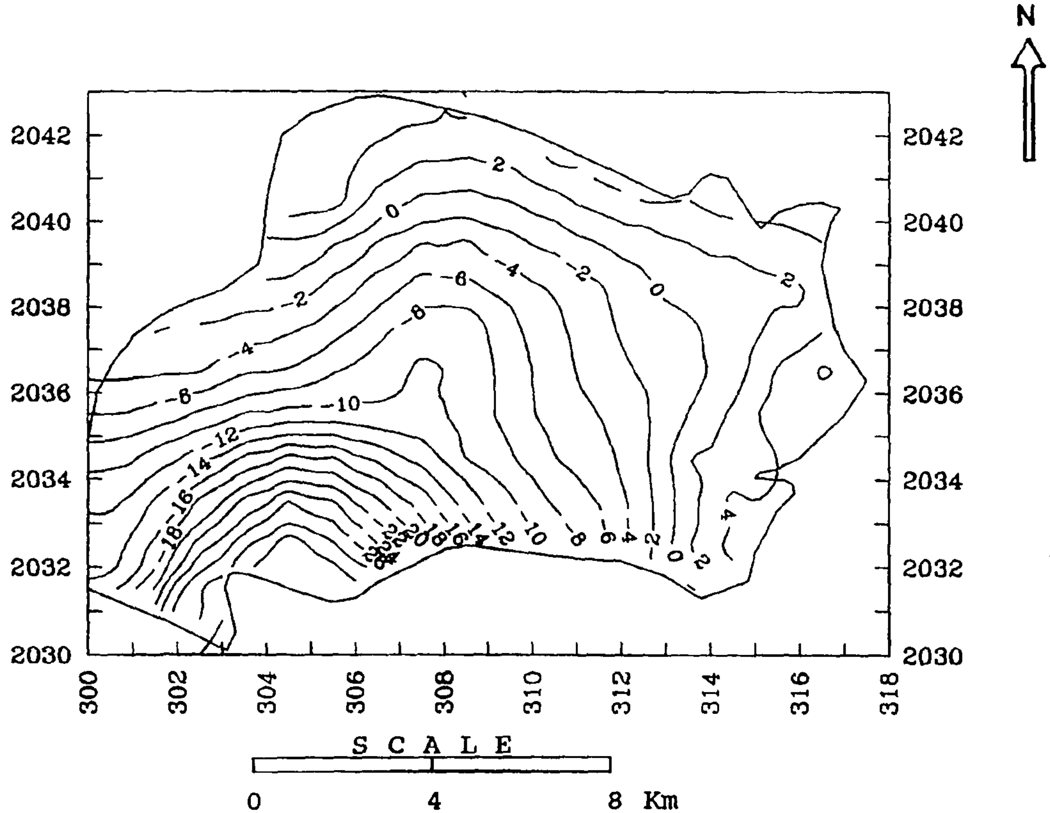

Figure 29 Contour map of simulated heads for scenario 5 by year 2010 (m).

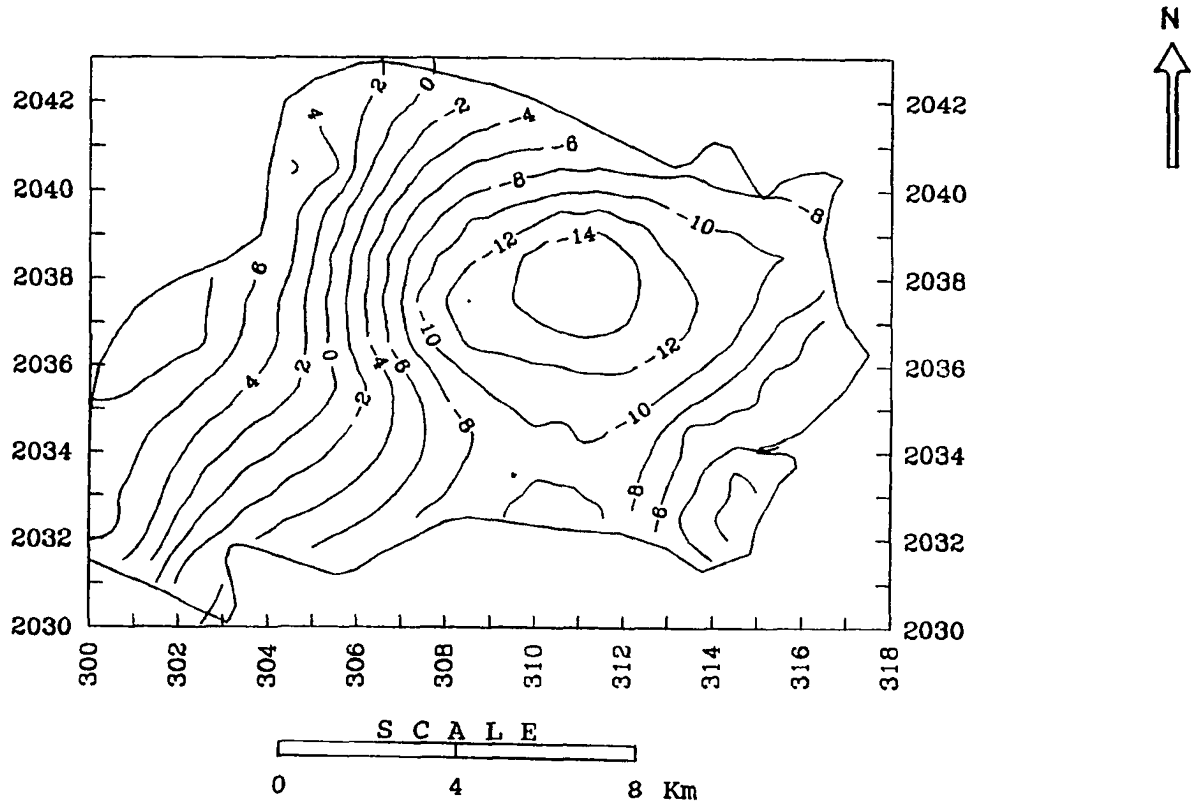

Figure 30 Contour map of simulated heads for scenario 6 by year 2010 (m).

CHAPTER 4

SENSITIVITY ANALYSIS

The purpose of the senstivity analysis was to determine the relative sensitivity of computed head and drawdown values to changes in the calibrated aquifer parameters and to the location of pumpage. The results of the sensitivity analysis provide useful information with respect to the reliability of calibrated aquifer transmissivity and storage coefficient and for planning future field investigations. Moreover, the sensitivity of head changes in a given area to changes in the location of pumpage will guide exploitation strategies or the solution of local problems, such as the drainage of excess of ground water.

The Azua aquifer model was analyzed for sensitivity in two steps:

- sensitivity to changes in aquifer parameters and
- sensitivity to changes in the location of pumpage.

Sensitivity to Changes in Aquifer Parameters

The sensitivity of the model to changes in the aquifer parameters (transmissivity and storage coefficient) was analyzed by allowing each parameter to vary for the entire aquifer or for specified areas within a certain range. The new values of the parameters were chosen in such a way that the percentage of discrepancy between inflow and outflow computed by MODFLOW was always less than or equal to 0.5%. Thus, convergency problems associated with numerical errors were minimized.

The effects of changes of hydraulic heads or drawdowns at given locations were compared to the model-calibrated values. Comparison was made for rows, columns and zones where the effects of changing the parameters were larger and accounting for other factors such as intensity of pumpage, lack of reliability of input parameters, and existence of drainage problems due to high water levels.

Sensitivity to Changes in Transmissivity

The effects produced by changes of transmissivity on hydraulic heads were computed in two different ways. One was to modify the calibrated nodal transmissivity values by the same percentage and observe how these changes affected the computed hydraulic heads. The other way was to modify nodal transmissivity values in a given area, while holding the other parameters constant, including transmissivity in other areas, and measure the effects on hydraulic heads (Mattlock, 1979; Boggs, 1980, and Hernandez, 1986).

To apply the first method, the calibrated nodal transmissivity values were modified by reducing or increasing them by a certain percent. The percents used were 25 for the steady-state solution and 50 for the transient solution of 1972-1977. The resulting heads were then compared with the final head values previously obtained for each case. Figures 31 and 32 show, respectively, the effects of varying the aquifer transmissivity by +/-25% on computed steady-state hydraulic heads along row 7 and column 10. The sensitivity is generally small and dependent on location within the model. The maximum head differences were 2.00 m for row 7, at columns 14 and 16, and 1.35 m for column 10 at rows 5 and 10. The effects of varying aquifer transmissivity by +/-50% on the computed drawdowns along column 14 for 1972-1977 are presented in Figure 33. The differences were generally small and varied from about 0 m at the aquifer borders to about 2 m at the center. For all of the cases, the nodes where changes in the aquifer transmissivities produced the largest effects on hydraulic heads are within the areas designated by zone 1, zone 4, zone 5 and zone 7 (figure 34). Additional field data in these areas would improve calibration.

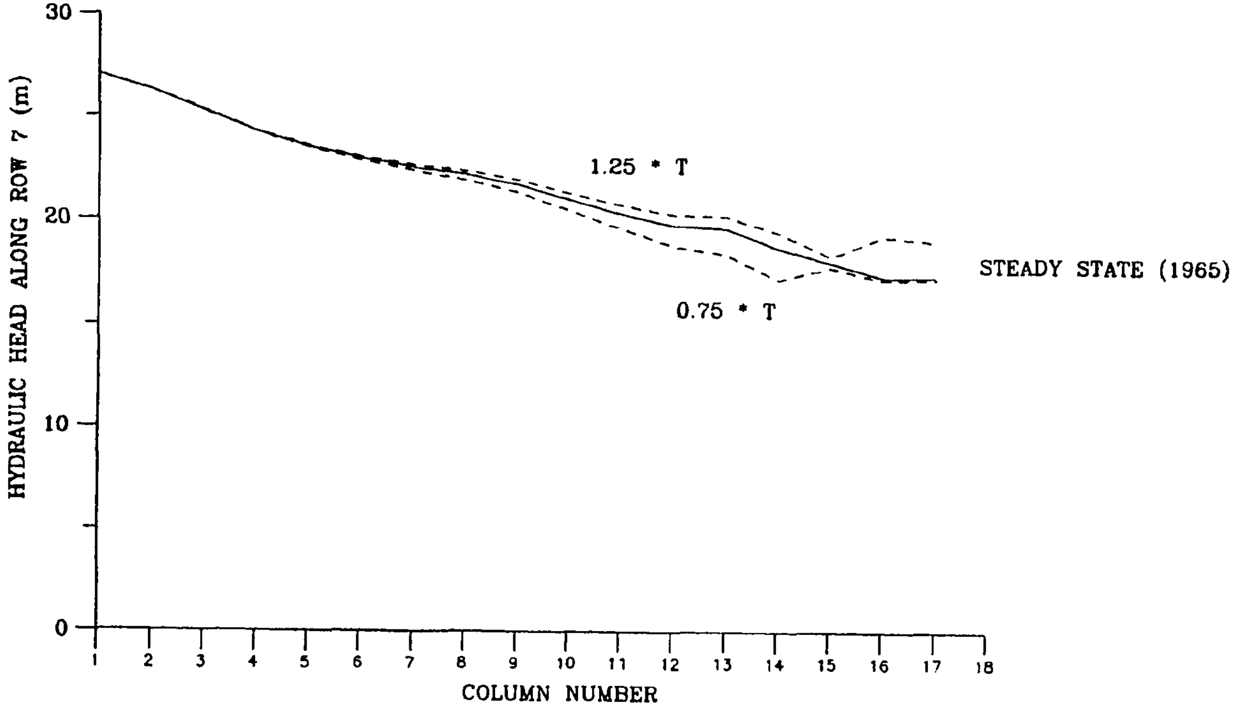

Figure 31 Effects of varying the transmissivity of the aquifer by +/−25% on the model-calibrated state-state hydraulic heads along row 7

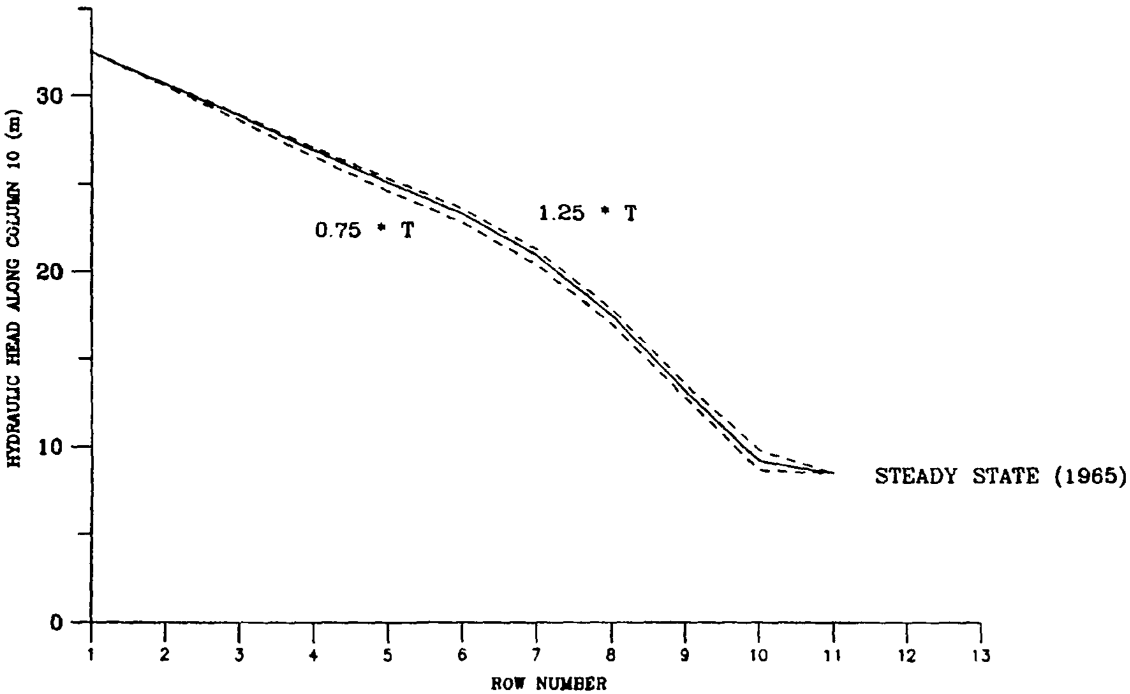

Figure 32: Effects of varying the transmissivity of the aquifer by +/-25% on the model-calibrated steady-state hydraulic heads along column 10

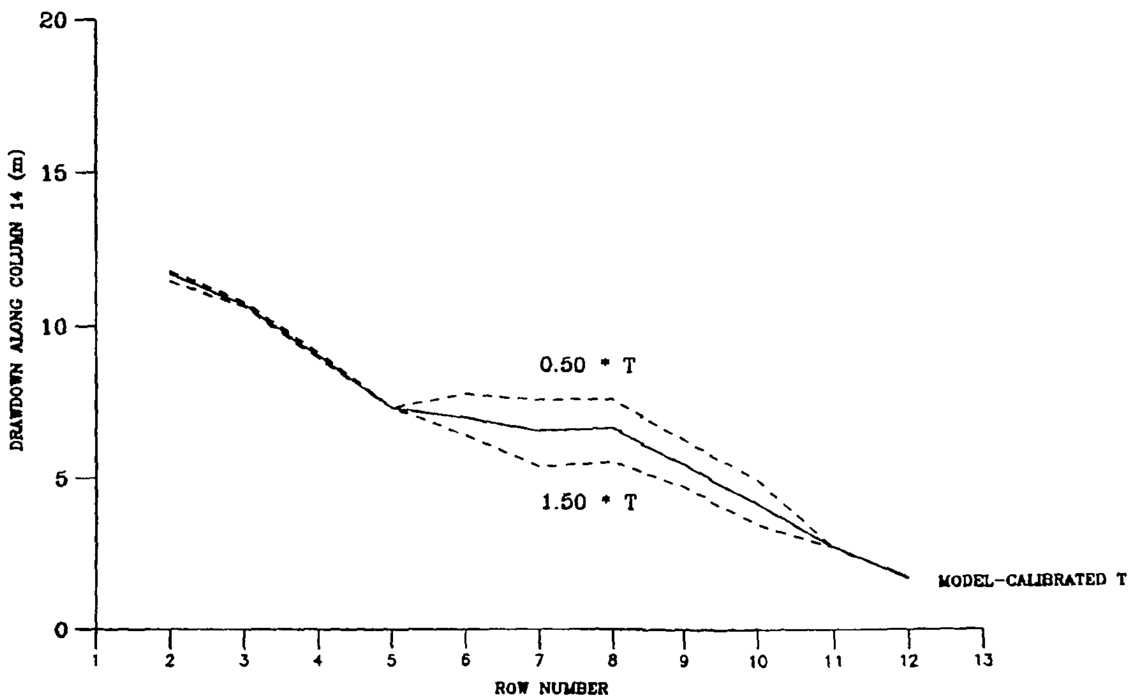

Figure 33 Effects of varying the transmissivity of the aquifer by +/-50% on the model-calibrated drawdowns along column 14 for period 1972-1977

To apply the second method, eight zones of equal size and shape were selected within the modeled area (figure 34). These sensitivity zones were chosen taking into account the results of the first method and providing an adequate representation of the hydrologic conditions of the entire aquifer. The zones were squares of 4 Km by 4 Km with the following limits:

Zone 1: rows 2 to 5, columns 12 to 15;
Zone 2: rows 1 to 4, columns 5 to 8;
Zone 3: rows 6 to 9, columns 1 to 4;
Zone 4: rows 5 to 8, columns 8 to 11;
Zone 5: rows 6 to 9, columns 13 to 16;
Zone 6: rows 9 to 12, columns 3 to 6;
Zone 7: rows 8 to 11, columns 7 to 10; and
Zone 8: rows 8 to 11, columns 12 to 15.

Transmissivities at all nodes within each zone were increased by 100% while the other parameters, including boundary conditions and transmissivities for the other zones, were held constant at their initially calibrated values. Eight transient simulations were performed for the period 1972-1977.

Boggs (1980) and Mattlock (1983) showed that the sensitivity of hydraulic head gradients to variations of transmissivity is proportional to the specific discharge and inversely proportional to transmissivity. These relations are quantified, for each sensitivity zone, by means of a weighted average sensitivity coefficient (B_k) defined as

$$B_k = \frac{\sum_{j=1}^{M} h_j * A_j}{S_k}, \quad \text{for all} \quad k = 1,2,. . .,K \qquad (6)$$

where

h_j is the absolute value of the resulting head change at node j,

A_j is the area of the cell associated with node i,

S_k is the area of the subdomain associated with zone k,

M is the number of nodes in zone k, and

K is the total number of zones.

Each time transmissivity is changed, K sensitivity coefficients are produced. Thus, by repeating this procedure for the K zones, a set of K by K sensitivity coefficients, forming a sensitivity matrix, is obtained. Inasmuch as the sensitivity zones are of the same size and transmissivities are modified by the same percentage, the relative sensitivity of heads observed in a particular zone to changes of transmissivities in other zones can be obtained by comparison of the sensitivity coefficients.

Those zones where a change in transmissivity produces the largest variations of hydraulic heads throughout the model, are the areas where more reliable data are required for a proper calibration of the model. Furthermore, additional field data in these areas would help to improve the estimates of transmissivity.

The sensitivity matrix for the transient simulation of the period 1972-1977 is presented in Table 6. In general, the magnitudes of the sensitivity coefficients are small and only 38% of them are greater than or equal to 0.25 m. The largest coefficient, 1.14 m, was obtained for zone 6, when transmissivity was modified in the nodes of that zone. If we adopt a size of the sensitivity coefficient of 0.25 m as a lower limit for comparison, zone number 7 is affected the most by variations of transmissivity. Zones 6, 3, and 4 are second in importance. Zones 1, 8, 2, and 5 affect hydraulic heads in other zones to a lesser degree. Zone 7 affects zones 6, 4, 2, and 1 the most. Zone 6 produces a large local effect and a moderate effect on zones 3 and 7. Zone 3 affects more zone 6; zone 4 affects more zone 2. Changing transmissivities in zone 2 produces only a local effect whereas zone 1, the area of largest pumpage and largest transmissivity values, has only moderate effects on zones 5, 8, and 2.

Although the sensitivity coefficients do not exhibit large contrasts, the sensitivity analysis indicates that useful information for future calibration could be obtained from aquifer tests performed in areas where heads are most sensitive to changes in transmissivity, such as zones 3, 4, 6, and 7. Improved estimates of transmissivity in these areas would enhance calibration.

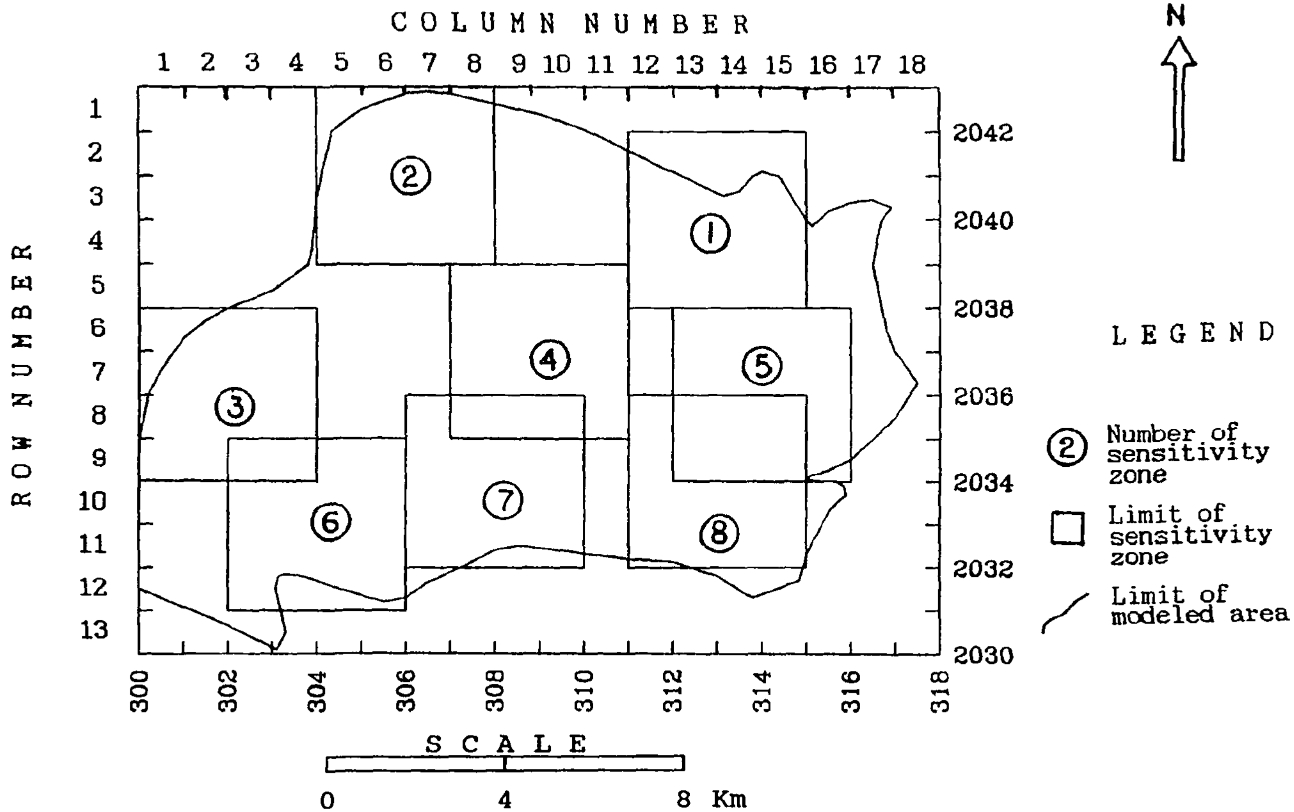

Figure 34 Map of sensitivity zones

Table 6 Sensitivity matrix for the transient simulation of the period 1972-1977

Zone	Sensitivity Coefficient in meters							
No.	B1	B2	B3	B4	B5	B6	B7	B8
1	0.24	0.25	0.09	0.08	0.49	0.05	0.05	0.33
2	0.07	0.69	0.03	0.13	0.10	0.08	0.13	0.11
3	0.07	0.25	0.63	0.06	0.03	0.75	0.25	0.03
4	0.15	0.57	0.21	0.30	0.15	0.05	0.34	0.35
5	0.08	0.02	0.01	0.01	0.10	0.01	0.01	0.09
6	0.14	0.30	0.70	0.09	0.11	1.14	0.40	0.03
7	0.40	0.42	0.03	0.55	0.32	0.60	0.52	0.17
8	0.15	0.08	0.03	0.10	0.29	0.02	0.02	0.24

<u>**Sensitivity to Changes in the Storage Coefficient**</u>

The effects of changes in the storage coefficient on drawdowns were computed by applying the procedure described for the first method of transmissivity sensitivity analysis. The storage coefficient was modified by +/-25% and by +/-50% for the transient simulation of 1972-1977. Thus, four additional runs were made. Results for row 4 and column 14 are presented in Figures 35 to 38.

Figure 35 shows that when the storage coefficient was increased from 0.10 to 0.125, drawdown decreased by a maximum of 1.80 m (15% of computed value) at node (4,4). Conversely, when the storage coefficient was reduced from 0.10 to 0.075, drawdown increased by a maximum of 3.06 m (25% of computed value) at node (4,4). Whereas, along column 14 (Figure 36) drawdown decreased by a maximum of 1.52 m (13% of computed value), at node (2,14), when the storage coefficient was increased from 0.10 to 0.125. Drawdown increased by 20% at nodes 2-8, when the storage coefficient was reduced from 0.10 to 0.075.

Figures 37 and 38 show the changes of computed drawdowns along row 4 and column 14, respectively, due to changes in +/-50% in the storage coefficient. For both cases, the maximum changes of drawdown occurred at the same locations as described above. Along row 4, an increase of the storage coefficient from 0.10 to 0.15 reduced drawdown at node (4,4) by 3.30 m (28% of computed value) whereas a decrease from 0.10 to 0.05 produced an additional drawdown of 8.49 m (72% of computed value). Along column 14, the maximum reduction of drawdown at node (2,14) was 2.78 m (24% of computed value) when the storage coefficient was increased from 0.10 to 0.15. Drawdown increased by 7.88 m (68% of computed value) at node (2,14) when the storage coefficient was reduced from 0.10 to 0.05.

These results show that the model is highly sensitive to changes in the storage coefficient, especially when the assigned values were smaller than the calibrated value of 0.10. Notice that the drawdown decreases more rapidly for a storage coefficient in the range 0.10 to 0.125 than in the range 0.125 to 0.15. Inasmuch as the largest observed differences occurred for areas of intensive pumpage, these are the areas where more reliable estimates of the storage coefficient are required for enhancing future transient calibrations of the model. Thus, aquifer tests, with observation wells, are needed for the entire aquifer and particularly for the northeastern, central and southwestern parts of the modeled area.

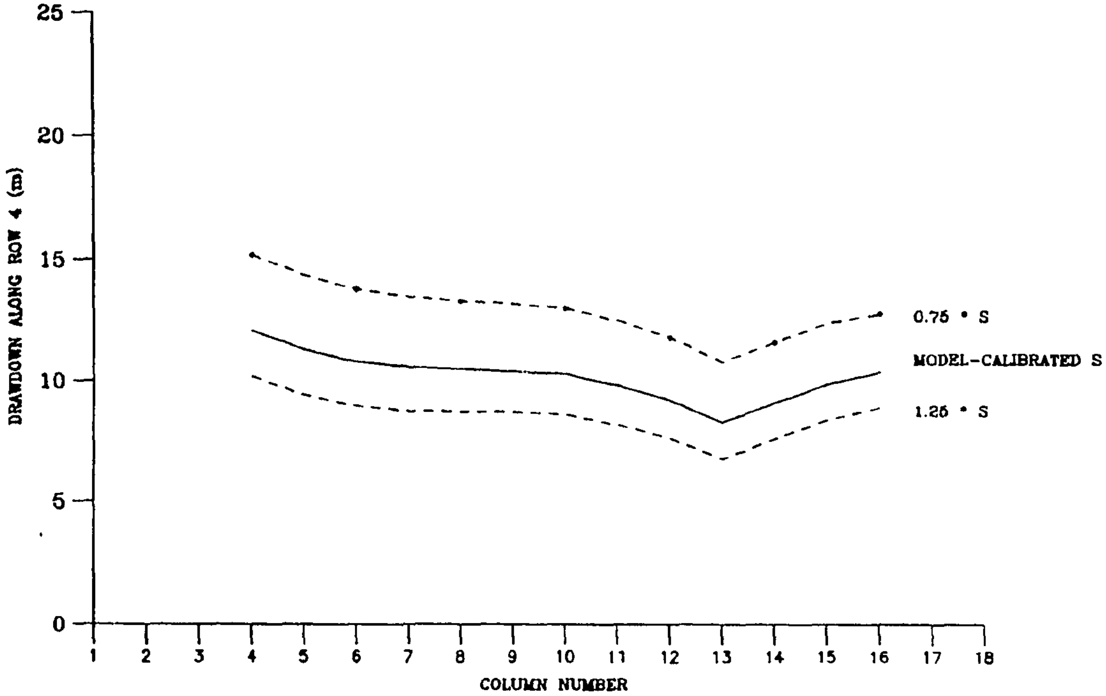

Figure 35 Effects of varying the storage coefficient of the aquifer by +/-25% on the model-calibrated drawdowns along row 4 for period 1972-1977

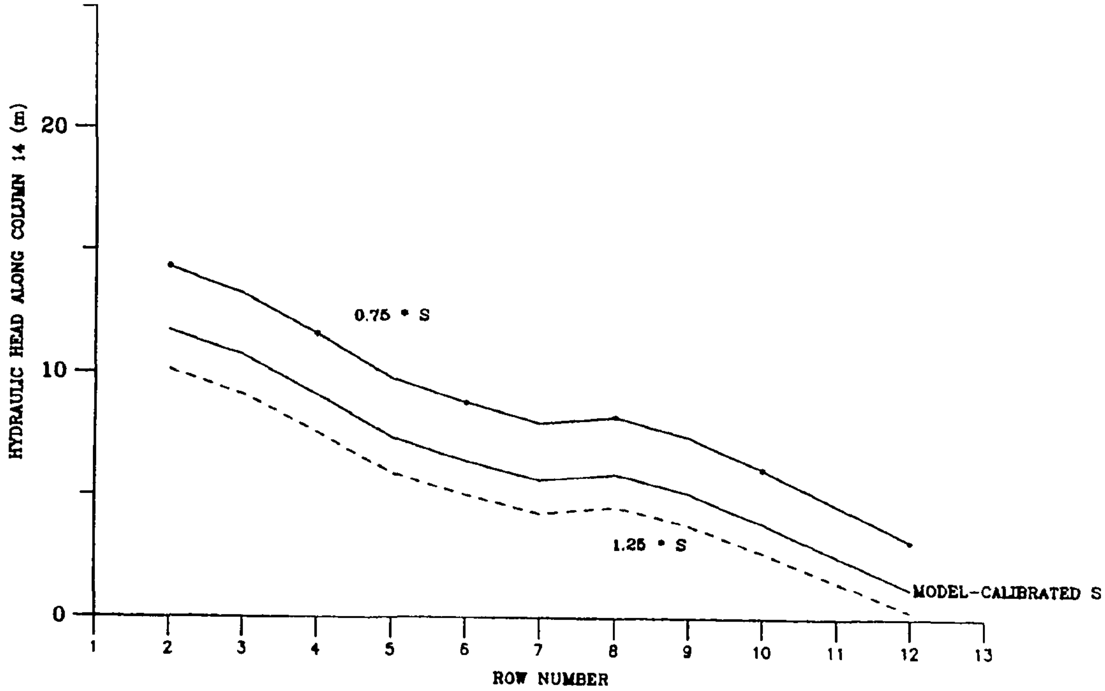

Figure 36 Effects of varying the storage coefficient of the aquifer by+/-25% on the model-calibrated drawdowns along column 14 for period 1972-1977

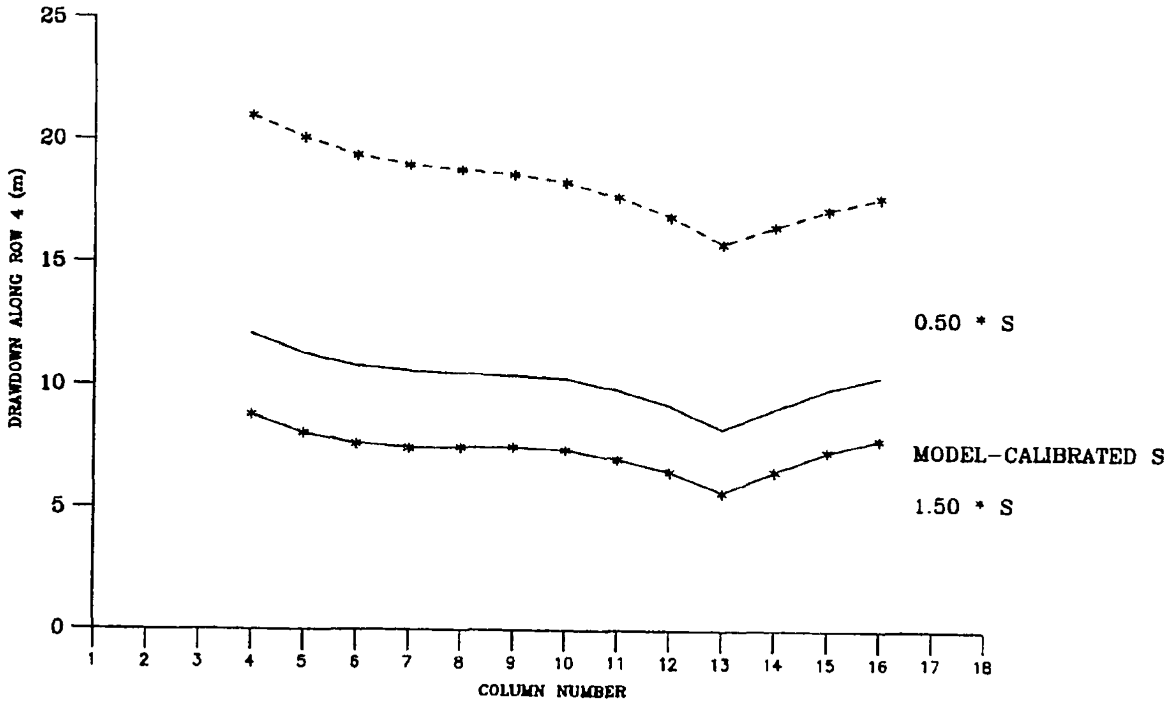

Figure 37 Effects of varying the storage coefficient of the aquifer by +/-50% on the model-calibrated drawdowns along row 4 for the period 1972-1977

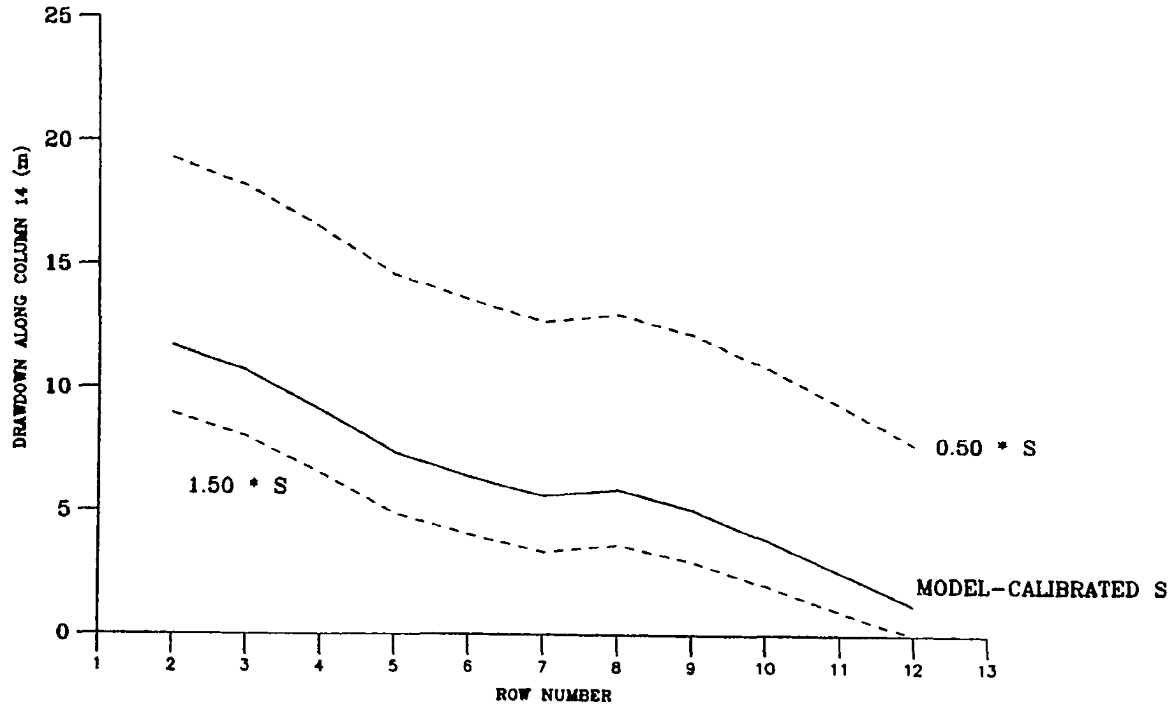

Figure 38 Effects of varying the storage coefficient of the aquifer by +/-50% on the model-calibrated drawdowns along column 14 for period 1972-1977

<u>**Sensitivity to Changes in the Location of Pumpage**</u>

This sensitivity analysis was performed for determining the locations where pumpage should be concentrated for reducing hydraulic heads in the areas where drainage problems exist. Six areas with critical drainage problems due to high water levels are presented in Figure 39 (numbered I to VI). They all are within the sensitivity zones 4, 5, 6, 7 and 8 and enclose subareas where the depth-to-water in 1988 (Figure 40) was 2.00 m or less.

The second method used for the sensitivity analysis of transmissivity showed that the sensitivity zones that produce the largest head changes in the areas with critical drainage problems were zones 1, 4 and 6. Thus, pumpage within these zones, for six different scenarios, was used for evaluating the effects on hydraulic heads in subareas I to VI. The pumping scenarios consisted of all of the possible combinations of pumpage at zones 1, 4 and 6, each pumping 20 MCM/year in addition to pumpage of 1988. This volume of pumpage is equivalent to 8 wells pumping at 1200 GPM, 24 hours a day, 365 days a year.

Seven transient runs were performed for the period 1989-2000. Taking heads at the end of 1988 as initial values, drawdown for each scenario was computed for 10 nodes at the end of year 2000. The scenarios were as follows:

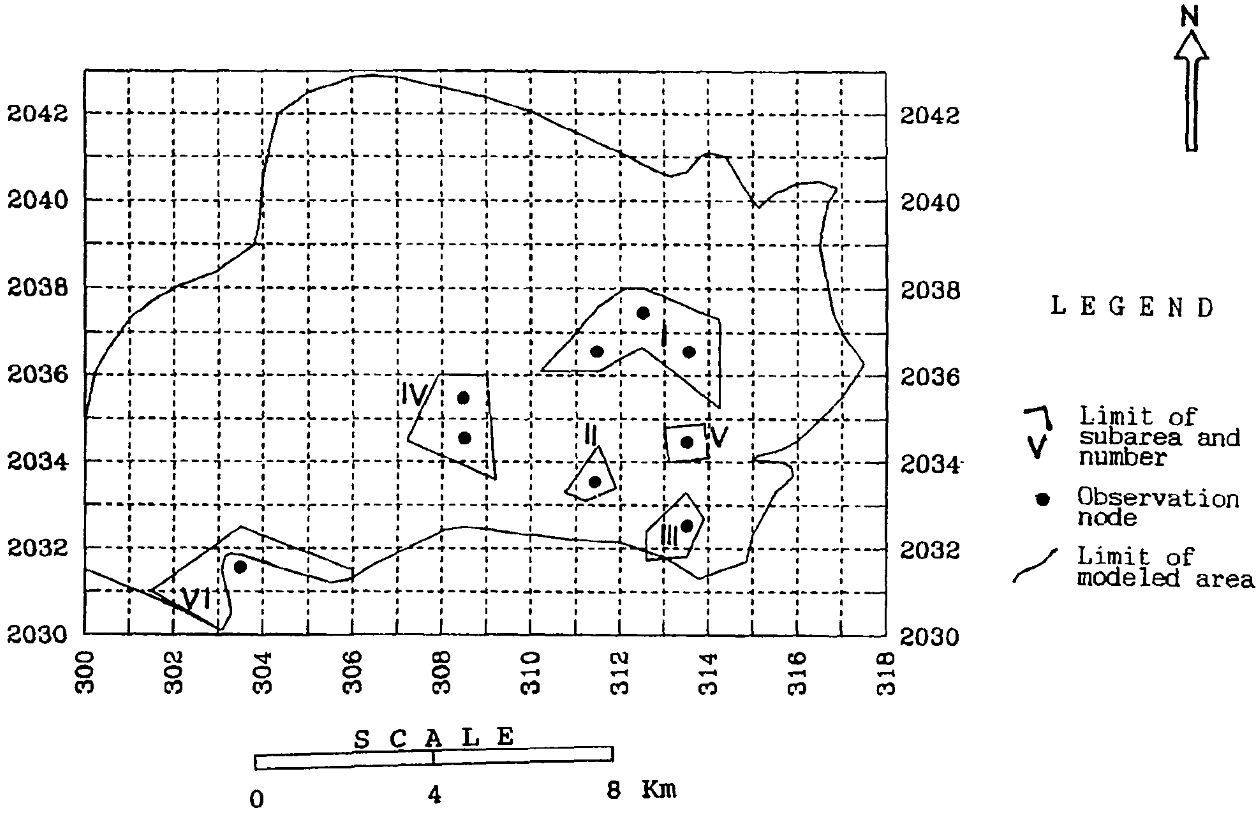

Figure 39 Subareas and nodes used for sensitivity analysis of heads to pumpage

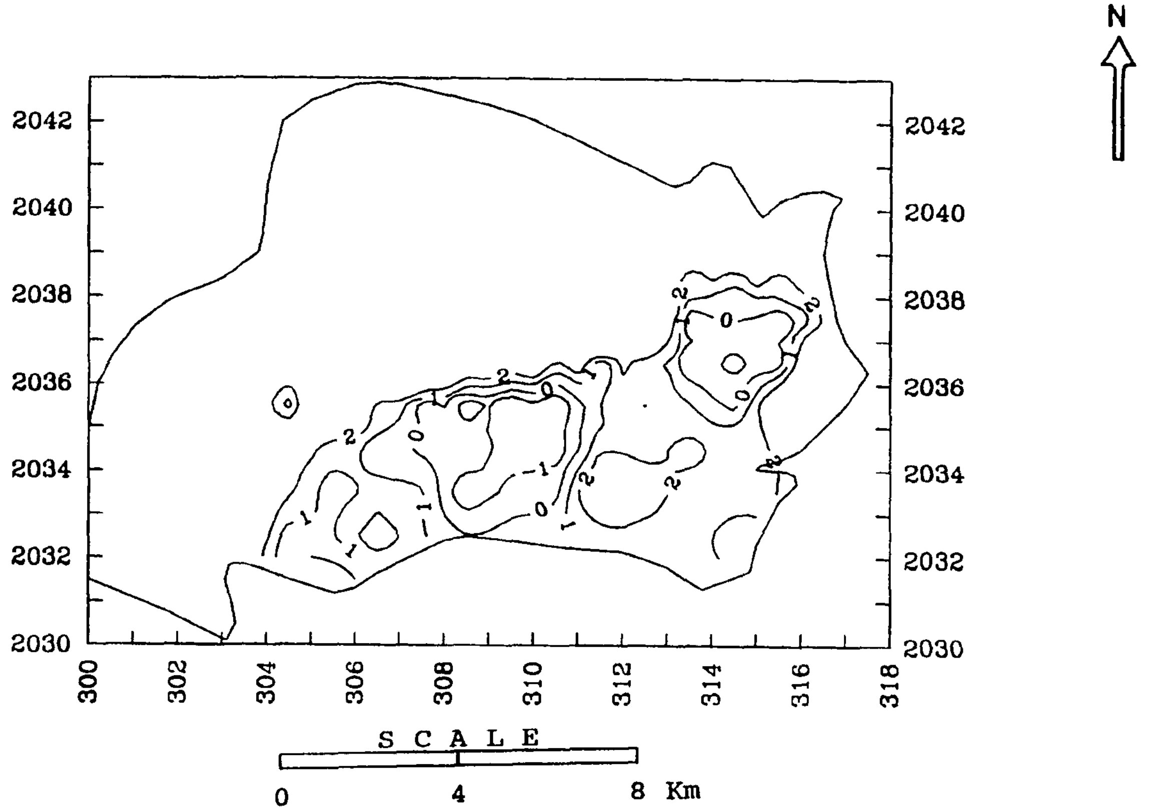

Figure 40 Depth-to-water map in area with critical drainage problems (m).

Scenario D1 : pumpage of 50 MCM/year in zone 1,

Scenario D4 : pumpage of 50 MCM/year in zone 4,

Scenario D6 : pumpage of 50 MCM/year in zone 6,

Scenario D14 : combination of D1 and D4,

Scenario D16 : combination of D1 and D6,

Scenario D46 : combination of D4 and D6 and

Scenario D146: combination of D1, D4 and D6.

The results of the sensitivity analysis are summarized in Table 7 and in Figures 41 to 43. The scenarios producing the largest drawdowns are D1 and D14. Scenario D146 increases drawdown by only 10% for all of the nodes but node (12,4), which is in zone 6, subarea VI. This confirms that changing conditions in zone 6 (scenarios D146 and D6) has a large local effect and a small effect on the other zones. Thus, any effort toward the solution of the drainage problem around Los Negros (zone 6) should be accompanied by pumpage within the Los Negros area itself.

Water levels in subareas I, II and IV are directly affected by pumpage in zone I (scenario D1) and by the combination of this pumpage with pumpage in zone 4. The average drawdowns produced by scenario D1 were 2.99 m for subarea I, 0.86 m for subarea IV and 0.31 m for subarea II. Scenario D14 is the combination of two other scenarios producing the largest effects on drawdowns over the entire critical area. The average drawdown produced for all of the critical area was 2.20 m and the average for subarea I was 5.31 m.

Subareas III and V are practically unaffected by the pumping scenarios described herein. Thus, drainage of these areas can be improved by developing new pumping sites within zones 5 and 8. Even though subarea V is closer than node (10,13) to the pumping centers, the subarea showed a lesser sensitivity (1.5 to 2 times smaller) than the node. This is due to the contributions of the Jura river to subarea V.

In conclusion, this sensitivity analysis has shown how pumpage can be used as and aid for solving the drainage problems of the critical areas, in combination with the existing network of drainage canals. With the exception of zones 6 and 8, no new wells need to be constructed for a short-term solution. If pumpage were increased within zones 1 and 4, a general reduction of water levels within the critical area would be, soon, observed.

The discussions on the results of applying different pumping scenarios for predicting the behavior of hydraulic heads showed that any solution to the drainage problem should be accompanied by an increase in pumpage.

Nevertheless, special care should be taken for those areas, like zones 1, 7 and 8, where sensitivity to local changes in pumpage is very high and where the intrusion of sea water could be a risk.

Table 7 Sensitivity of areas with critical drainage problems to seven pumping scenarios expressed as drawdown in year 2000 with respect to hydraulic heads in 1988 (m).

SUBAREA No.	NODES (R,C)	SENSITIVITY ZONE	SCENARIO D1	D14	D146	D16	D4	D46	D6
I	6,13	5	3.96	6.44	6.68	4.14	1.72	1.86	0.33
	7,12	4-5	1.55	4.12	4.34	1.69	1.29	1.42	0.14
	7,14	5	3.46	5.39	5.60	3.61	1.17	1.24	0.32
II	10,12	8	0.31	0.84	0.89	0.35	0.44	0.47	0.05
III	11,14	8	0.01	0.01	0.01	0.01	0.01	0.01	0.00
IV	8,9	4-7	1.11	2.65	2.90	1.38	1.97	2.18	0.32
	9,9	7	0.61	1.43	1.50	0.72	1.11	1.20	0.14
V	9,14	5-8	0.18	0.41	0.43	0.19	0.16	0.16	0.07
VI	12,4	6	-0.04	0.10	4.89	4.53	0.00	4.67	4.35
	10,13	8	0.23	0.65	0.69	0.26	0.27	0.30	0.03

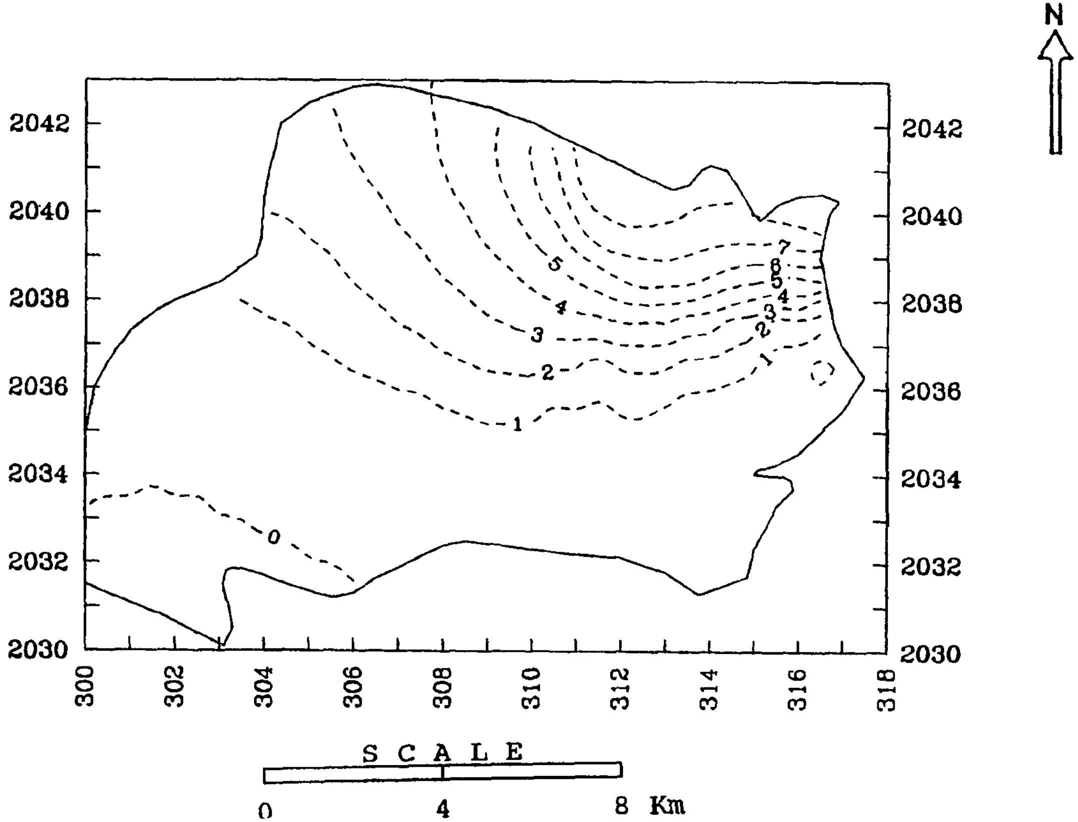

Figure 41 Simulated drawdown for scenario D1 by year 2000 (m).

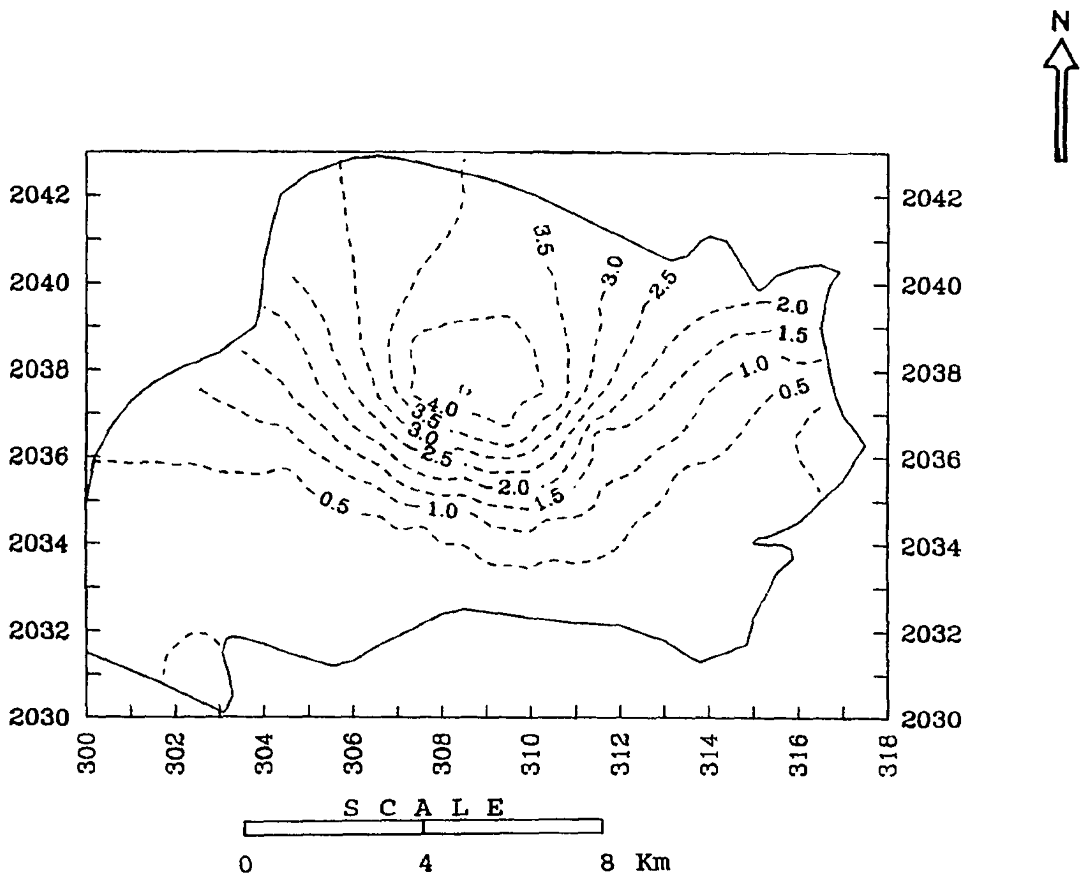

Figure 42 Simulated drawdown for scenario D4 by year 2000 (m).

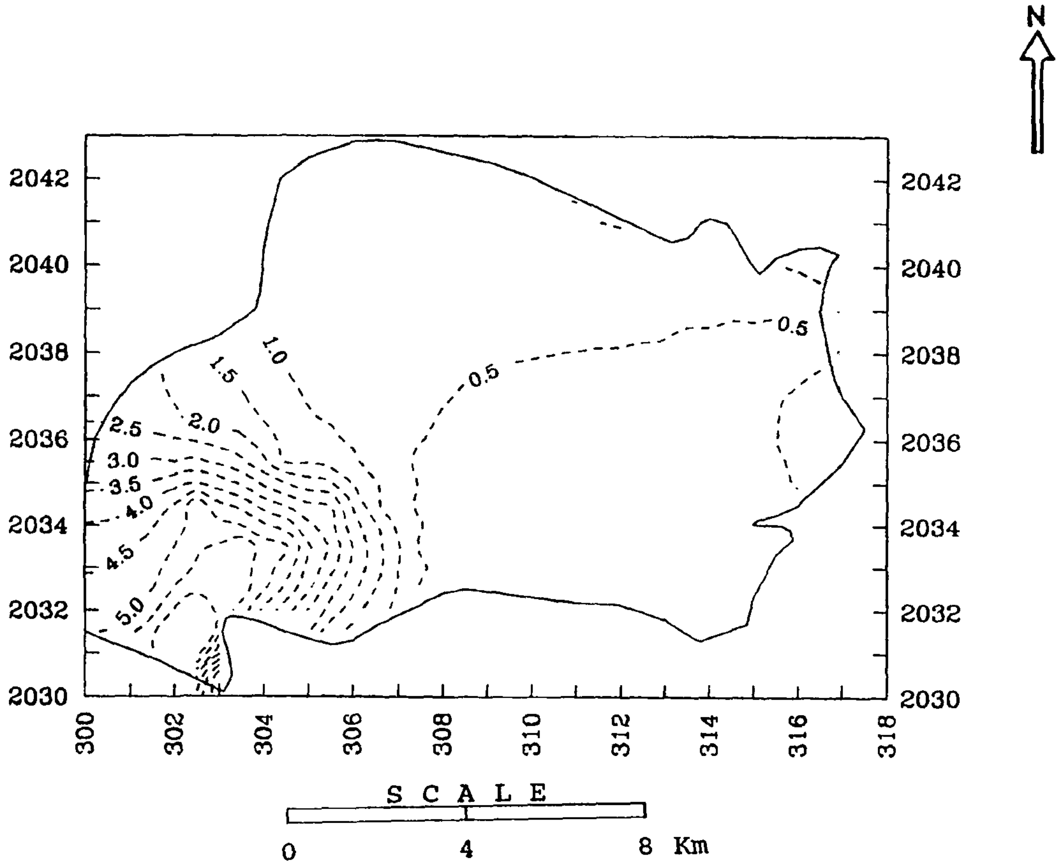

Figure 43 Simulated drawdown for scenario D6 by year 2000 (m).

CHAPTER 5

SUMMARY AND CONCLUSIONS

The primary objectives of this thesis were to: 1) develop a ground-water flow model for evaluating the behavior of a portion of the Azua aquifer, Dominican Republic and estimate the range of the effects on the aquifer derived from the application of proposed and assumed pumping schemes and 2) determine the areas of the Azua Aquifer where new pumpage should be initiated in order to decrease the ground-water levels in the lowland areas affected by critical drainage problems.

The first objective was accomplished by using the Modular Finite-Difference Ground-Water Flow Model of the USGS. A two-dimensional model was developed for the study area by using the available data described in Chapter 3. The effects of pumpage, local recharge due to irrigation return flows, evapotranspiration, the drainage network and leakage from rivers and canals were simulated by using the respective packages of MODFLOW. The aquifer boundary conditions and the initial ground-water elevations were introduced to the model through the Basic Package, whereas the aquifer parameters (transmissivity and storage coefficient) were given through the Block-Centered Flow Package. The Strongly Implicit Procedure Package was chosen for solving the finite-difference differential equations. The Output Control Option was used for controlling the frequency and volume of output from the simulations.

The aquifer conditions were simulated in two steps. The first step consisted of adjusting the model input data in such a way that the steady-state hydraulic head configuration of 1965 was reproduced. Adjustment of transmissivities was made by a trial and error procedure. The areas requiring major adjustments were those in the vicinity of the boundaries, especially where fluxes were highest. Minor adjustments in recharge, drains and river-canals were also needed. The second step involved a verification stage, by matching the computed transient hydraulic heads with the 1971, 1977, and 1988 measurements, and a prediction stage, by assuming seven pumping scenarios based on an actual plan for developing new pumpage. The storage coefficient, whose final value was 0.10, was calibrated together with pumpage and other input data during the verification stage. The total recharge through the boundaries of the modeled area, approximately 40 MCM/year, was considered constant for all of the simulations. Nearly 75% of the recharge occurs through the northern boundary of the modeled area, whereas the remaining 25% occurs through the western boundary. These figures confirm the statement of Chapter 2 regarding the location of the main recharge area in the alluvial fans and the old alluvial deposits north of the Sanchez highway. Seven transient simulations were performed during the prediction process. The pumping schemes outlined in Chapter 3 were based on a plan for developing new pumpage by means of new wells and existing wells whose rehabilitaTion is in progress. The outputs consisted of water budgets and distribution of heads and drawdowns up to year 2010.

For all of the simulations, contour maps of ground-water elevations, drawdowns and differences between observed and simulated hydraulic heads were drawn by means of the contouring package SURFER. For drawing these maps, a program was written (called READER) that takes the unformatted values saved by MODFLOW and transforms them into actual coordinates with respect to the reference system of the area studied. Graphs representing the behavior of hydraulic heads along model rows and columns passing through critical areas were also drawn.

Once the predictive capabilities of the model were demonstrated, a sensitivity analysis of the aquifer parameters was undertaken. The results suggest that ground-water levels are most sensitive to changes in the storage coefficient than to changes in transmissivity. The greatest changes in the water levels occurred in areas where pumpage is largest. The analysis also showed the areas where changes in transmissivity have the largest effects over the entire aquifer and over specific zones, such the lowlands.

The second objective was accomplished by means of a sensitivity analysis of the water levels in the low-land areas to variation of pumpage in three different zones. These zones were the ones determined in the previous sensitivity analysis as having the largest effects on the water levels in the areas of Las Terreras, Pueblo Viejo, La Estancia, Rosario and Los Negros, where the drainage problem is still critical. It was demonstrated that new pumpage should be developed in the central part of the valley and in the Azua-Las Clavellinas area for an immediate reduction of the high water levels at the four first villages. Conversely, the largest reduction of water levels at Los Negros occurred only when local pumpage was allowed to develop.

The following conclusions can be made from this study:

1. The two-dimensional ground-water flow model constructed during this study demonstrates its effectivity for representing the behavior of the Azua Aquifer. The degree of effectiveness of the model is conditioned by the limitations inherent to all models regarding an approximate representation of the true behavior of natural systems.

2. The most important source of error appears to be the lack of sufficiently accurate input data, including water levels, pumping rates, transmissivity and storage coefficient. Another major source of error probably stems from the assumptions imposed on the model. These assumptions are: a) the aquifer system is assumed as a single confined layer with fully penetrating wells; b) the aquifer is isotropic; c) ground-water flow is completely horizontal; d) the system was in steady state in 1965; e) the storage coefficient is uniform for the entire aquifer; f) the rates of recharge and discharge through the aquifer boundaries are constant; and g) recharge from the Tabara River is negligible. Inasmuch as better data were not available, assumptions a), b), e) and f) appear to be unavoidable; assumption c) is inherent to the two-dimensional numerical scheme used; assumption d) relies on the findings of other investigators (Gilboa, 1965), who found,

based on information provided by local farmers, that water levels stayed approximately the same at least from 1963 to 1965. Assumption g) was made based on the analysis and measurements performed in previous studies (Tahal, 1971).

3. The verification process demonstrated the predictive capability of the model by reasonably matching the head measurements of 1971, 1977 and 1988.

3. The rate of lateral subsurface flow entering the Azua aquifer through the northern boundary is three times the rate entering through the western boundary. Fifty five percent of the total inflow comes from the alluvial fans and old alluvial deposits of the Jura and Irabon rivers.

4. The amount of local recharge within the modeled area is small (8 MCM/year in 1978-1988). This is consistent with the results of water balances performed in other studies (Tahal, 1971; INDRHI-TAHAL, 1983; and INDRHI-SNC, 1984).

5. The amounts of river-canal leakage (average 60 MCM/year in 1983-1988) and discharge resulting from the combination of flow to the drainage network and evapotranspiration are of the same order of magnitude.

6. Even though the drainage canals have lowered the water levels in the valley, allowing reclamation of a portion of the previously flooded lands, the problem of

high-water levels still persists in areas that have shown to be insensitive to the operating drainage network.

7. The rational utilization and management of the water resources of the Azua Valley is one of the most important factors regarding the improvement of the present situation and allowing future development. In order for these goals to be accomplished, the development of new pumpage is critical. Among the benefits of increasing controlled ground-water utilization in Azua are:

i) Will cause a reduction of water levels, by means of pumpage from wells presently operating and those proposed for rehabilitation. Three main objectives will be met in the short term: a) reduction of the artesian flows; b) mitigation of the flooding process; and c) reduction of the drainage problems.

ii) Will allow the utilization of additional water for irrigation and other uses. The volume of ground water that can be utilized is estimated in about 100 MCM/year in the long term, roughly 50% of the total irrigation demand for 12000 hectares in VA-1. Inasmuch as deficits of irrigation water in the service area of the Sabana Yegua Dam have been forecasted, ground-water will play an important role.

8. To accomplish the short-term objectives outlined above, priority should be given to those zones of the aquifer where the development of new pumpage causes the fastest reduction of water levels in the areas with critical drainage problems. Thus, existing wells should be rehabilitated and pumpage initiated in the central part of the aquifer and in the Azua-Las Clavellinas area. On the other hand, the development of local pumpage is the most effective way of solving the Los Negros drainage problem.

9. On the long term, the following activities are recommended: a) complete rehabilitation of all existing wells and construct new wells where necessary, b) connect wells to the nearest irrigation canals or reservoirs and start using ground water and c) divert only the strictly necessary volumes of surface water, those required for satisfying the requirements of areas not serviced by wells.

10. Inasmuch as reliable data are needed for future modeling and for a more efficient operation of the system, the following recommendations are made:

i) Perform aquifer tests in all of the rehabilitated wells and new wells. Observation wells should be used and adequate technical supervision should be available;

ii) Collect accurate lithological data from new wells, and

iii) Maintain continuous records of pumpage, water levels and water quality as well as volumes of surface water diverted to the different areas of the valley.

11. In order to avoid deterioration of the water quality as a result of indiscriminated pumpage, especially in those areas at the southern boundary that are highly sensitive to local pumpage, both water levels and water quality should be strictly monitored. Withdrawals should be limited to those amounts not causing excessive drawdowns.

12. Future improvements of the model will, indeed, make of it a more useful tool for the integrated management of the Azua Aquifer System.

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