A-3-3 Evaluation of February 1982 Flood

A flood hydrograph has been observed by JICA study team during the second phase field investigation (Nov. 6, 1981-Mar. 31, 1982) at the point of Mulayyinah (S-1) situated about 500 m upstream from the point S-2. Figure-3 present the observed data of rainfalls and discharges which are computed by use of Manning's formula on the basis of measured river stages and cross-section. Under the assumption that n = 0.038, which is commonly adopted in PAWR's estimates, the peak flood discharge has been estimated at 770 cu.m/sec.

The following are reviewed from observations:

- No precedent rain has been observed in the drainage basin before 13th February. Accordingly, rains during Feb. 13 and up to 5 a.m of Feb. 14 are considered to be initial losses of rains, which are consumed to saturate surface layers of drainage basin.

- After once soil layers have been saturated with rain water, sudden floodings, characterized by a rapid rise, sharp peak and rapid recession, have been observed. Two peaks of the flood are likely caused by series of rainfalls, whose peaks were observed at 8 a.m and 2 to 3 p.m.

- Although large variations in rain intensity (and hence runoff rate) during a storm are reflected in the shape of the resulting hydrograph, the time scale of intensity variations depends mainly on basin size. Based on the conceptions of a unit hydrograph, flood concentration time of 3 hours as read on Figure-3 could be applicable to any other given intensity of storm rainfalls.

- If the former of two sharp peak hydrographs is extracted from observations so as to represent the typical shape of flood hydrograph within the drainage basin under consideration, a ratio of peak discharge to the volume of flood is given as:

$$\text{Ratio} = \frac{\text{Peak Discharge}}{\text{Flood Volume}} = \frac{\text{770 cu.m/sec.}}{\text{7.7158 MCM}} = 9.975 \times 10^{-5} (\text{sec}^{-1})$$

- In connection with the shape of a typical flood hydrograph in Oman, the PAWR's report on "Surface Water Gauging Station Network of the Sultanate of Oman (PAWR 83-2 Report)" mentions as follows:

  "A flood hydrograph is the record of a flood event from the beginning to the end; in a desert hydrograph, the discharge rises sharply from no flow to some peak value and then drops off again, returning somewhat more slowly to zero flow. The shape of the hydrograph will vary from basin to basin and from storm to storm, but within an area that is geographically similar, hydrographs will be similar in shape."

- The coefficient of peak flood runoff, fp, is also calculated as:

$$fp = \frac{\text{770 cu.m/sec x 3.6}}{\text{654 sq.km x 10.5 mm/hr}} = 0.40$$

Where, concentration time of peak food is taken at 3 hours after inspection of observed pattern of rainfall and hydrograph, and rainfall intensity during this period in terms of areal average is seen as 10.5 mm/hr against the second peak of rain and discharge, after the surface layers of drainage basin have once been saturated by rain water.

The same procedure was employed to evaluate the flood observed at the river mouth (S-4) for the same duration of February 1982. The findings are then summarized as below:

- Peak Flood Discharge: Q_{max} = 163 cu.m/sec
- Flood Volume : V = 4.03 MCM
- Ratio of Peak Discharge to Flood Volume

$$= \frac{163 \text{ cu.m/sec}}{4.03 \text{ MCM}} = 4.045 \times 10^{-5} \ (\text{sec}^{-1})$$

A-3-4 Maximum Probable Flood

1. Maximum Probable Rainfall

Rainfall measurements are obtainable from six raingauge stations installed within and the vicinity of the drainage basin of the proposed dam for the period of 11 years from 1974 up to 1984. Areal rainfalls computed by using Thiessen method are tabulated in Tables A-3-4 and A-3-5. Since the observation period is too short, these data are unusable for the foundation of statistical assessment. Therefore, the statistical evaluation of rainfall records should be to a great extent based on the data collected at Muscat, only where a long-term rain records are available since the late 1800's.

Table A-3-8 summarizes the rain records collected from Muscat. A long-term fluctuation of annual rains is visualized as shown in Figure A-3-7. From this, the recent years after 1974, during which rain data are available within the project boundary, are not always abundant in rain with exception of the year 1976.

In areas along the Northeast coast of Oman involving the project area and Muscat, most of annual rainfall is concentrated during winter season from November to April. In addition, in many years, more than 80% of annual rain concentrates in a particular month of the year. Table A-3-9 presents characteristics of maximum rainfalls observed in the Northeast coast area.

As a long-term rain record, only monthly rain data are obtainable from Muscat for a relatively long period of about 90 years. To characterize the maximum probable rainfalls of a relatively short duration, reatios of monthly maximum and daily maximum rains to annual rains are extracted from all of available data and plotted as given in Figure A-3-8. Envelope curves so drawn to enclose all the points would show the maximum probable monthly or daily rainfalls when a reasonable estimate of annual rainfall is made.

Probability study on annual amount of rainfall at Muscat gives 255 mm of annual rainfall with a recurrence interval of 50 years.

With regard to regional difference of annual rains, the dainage basin of the proposed dam has been receiving about 1.4 times of annual rainfall reported at Muscat.

$$\frac{\text{Area Rainfall}}{\text{Muscat Rainfall}} = \frac{127.8 \text{ mm/yr.}}{97.0 \text{ mm/yr.}} = 1.32 = 1.4$$

Maximum Probable Rainfall

- Annual rainfall of 50-years return period at Muscat = 255 mm/yr.*/

- Maximum probable annual rainfall at Muscat
  = 255/06.6 = 425 mm/yr. = 430 mm/yr.

(Note: A ratio of provisional estimate of the maxmimum probable flood to the 50-years return period flood, 1 : 0.6, is referred to in the "Report on Water Resources Study, Phase II, and Technical Proposal for Construction of Water Recharge Projects for the Government of Oman: Ministry of Agriculture and Fisheries.")

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*/ The rainfall of 255 mm per year was decided from a curve which plotted annual rainfall records observed at Muscat from 1894 to 1983 on a logarithmic probability paper. (The rainfall record refer to Table A-3-8 "Monthly Rainfall at Muscat".)

(see Figure A-3-8) = 430 x 0.66 = 284 mm/mo.

- Maximum probable daily rainfall at Muscat
  (see Figure A-3-8) = 430 x 0.40 = 172/mm/day

- Maximum probable monthly rainfall within the
  proposed basin = 284 x 1.4 = 398 mm/mo.

- Maximum probable daily rainfall within the
  proposed basin = 172 x 1.4 = 241 mm/day

2. Maximum Probable Flood

Many empirical formulas have been presented for estimating flood peaks from relatively small basins. For the most part, these formulas are misleading and unsound and they can be discussed only in a general way, because all of them contain coefficients or constants which can be evaluated only where sufficient volume of storm-flood observations are available.

The most widely used design equation for basins in known as the rational formula. The equation states that the rate of runoff equals to the rate of supply (excess rainfall) if the rain lasts long enough to permit the entire area to contribute. As is indicated that the formula is ideally applicable for catchments of 5 to 10 sq. km. and is commonly used for surface drainage design in urban areas, the above assumption is literally correct for an area of very small area and may not involve serious error for basins up to a few square kilometers in area when the time of concentration is taken correct.

For larger areas, the coeeficient of run-off not only expresses the proportion of the total rainfall which runs off but also the effect of overland flow and channel storage on the peak. The coefficient is defined as the ratio of the theoretical absolute peak (100% runoff and no storage) to the actual peak, and fortunately this value can be obtained from the actual observation of flood hydrograph collected at Mulayyinah in February 1982. Moreover, according to the advanced studies made by Kadoya and others of Kyoto University in Japan based on actual measurements of hydrologic data collected from rivers of 0.13 to 740 sq.km catchments, it is varified that the rational formula is still applicable to rivers of more than several hundred square kilometers catchments when the strict application of the concentration time of peak flood discharge is accompanied. With an assumption that the application of this formula can still be extended to the range of 812 sp.km.: -

a) The coefficient of peak flood runoff has been calculated as fp = 0.4

b) If the maximum probable rainfall of 241 mm/day is distributed within a short duration of the concentration time of 3.0 hours, the average intensity of rainfall is then given as:

$$i = \frac{241}{3.0} = 80.3 \text{ mm/hr}$$

c) The maximum probable flood discharge is thus computed as:

$$\begin{aligned} Qp &= 1/3.6 \times fp \times i \times 812 \text{ sq.km} \\ &= 1/3.6 \times 0.40 \times 80.3 \times 812 \\ &= 7,245 \text{ cu.m/sec} \end{aligned}$$

Peak-Volume Relation

A ratio of peak flood discharge to volume of flood runoff, as derived from actual measurement of flood hydrograph is February 1982, was applied to evaluate the maximum probable flood which would be caused by the maximum probable rainfall.

- Flood Volume = 812 sq.km x 241 mm x 40%
  = 78.277 MCM

- Maximum Probable Flood

Q_p = Flood Volume x Peak Discharge Flood Volume
= $78.277 \times 10^6 \times 9.9795 \times {}^{-5}$
= 7,828 cu.m/sec

Table A-3-4 Areal Rainfall Above & Below Dam Site

Areal Rainfall at Dam Site (Above Dam: CA = 812 sq.Km)

UNIT = MM/MONTH

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	ANNUAL
1974	2.9	39.7	5.8	2.3	0.0	0.3	2.0	4.0	2.1	9.6	0.0	0.0	68.6
1975	7.2	63.0	0.7	0.0	2.0	1.9	1.5	15.3	3.0	0.0	0.0	0.0	94.5
1976	26.8	138.4	142.5	53.6	0.0	1.7	4.4	29.5	4.1	0.0	13.5	6.3	420.8
1977	25.5	25.9	0.8	39.0	33.8	22.9	0.0	0.0	0.0	3.9	8.9	0.0	160.7
1978	0.0	39.9	4.6	3.0	0.0	1.9	16.4	30.0	2.1	0.0	0.0	0.0	97.8
1979	21.6	1.2	1.3	4.3	0.5	4.6	6.0	0.0	3.7	5.6	1.7	19.5	69.9
1980	6.1	20.5	21.8	0.0	1.5	0.0	10.7	0.1	0.0	0.0	0.0	9.9	70.5
1981	3.5	0.0	1.1	10.1	17.6	2.4	12.5	0.0	0.0	0.0	0.0	0.0	47.2
1982	0.0	150.9	41.2	0.1	5.4	0.0	0.0	1.4	0.8	0.8	7.9	0.8	209.3
1983	3.6	42.5	29.8	32.3	0.0	0.0	0.4	38.3	0.0	0.0	1.7	0.0	148.6
1984	0.0	0.0	0.0	0.0	0.0	0.0	6.0	0.1	3.3	0.0	0.0	2.5	11.8
TOTAL	97.2	522.0	249.5	144.6	60.6	35.6	59.8	118.6	19.1	19.9	33.8	39.0	1399.9
MEAN	8.8	47.5	22.7	13.1	5.5	3.2	5.4	10.8	1.7	1.8	3.1	3.5	127.3

*** AREAL RATIO ***

DAQIQ = 0.095 KITNAH = 0.131 HAYL(WJ) = 0.204 HAYL(WH) = 0.279 FARFAR = 0.291 SOHAR = 0.0

Areal Rainfall below Dam (CA = 471 sq.Km)

UNIT = MM/MONTH

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	ANNUAL
1974	1.4	62.9	3.5	0.8	0.0	0.0	0.0	1.1	1.0	5.0	0.0	1.5	77.2
1975	4.2	52.1	0.0	0.0	0.7	0.0	0.0	10.4	0.0	0.0	0.0	0.0	67.4
1976	9.5	123.4	77.9	58.6	0.0	0.0	0.5	12.1	1.4	5.7	6.1	12.2	307.4
1977	56.1	26.2	1.8	41.9	22.0	10.1	0.0	0.0	0.0	1.9	16.9	0.1	177.1
1978	0.0	37.4	5.3	3.0	0.0	0.9	4.6	13.8	0.0	0.0	0.0	0.2	65.1
1979	20.3	0.0	1.6	3.8	0.0	0.0	0.0	0.0	0.0	28.3	1.5	44.2	99.8
1980	2.4	6.5	8.7	0.8	0.9	0.0	3.0	0.0	0.0	0.0	0.6	4.9	27.8
1981	4.3	0.0	1.0	16.5	14.0	0.0	7.6	0.0	0.0	0.0	0.0	0.0	43.4
1982	0.0	140.4	52.3	1.4	0.9	0.0	0.0	0.0	0.0	0.4	14.4	7.2	217.0
1983	0.8	53.6	34.4	26.1	0.0	0.0	0.0	13.3	0.0	0.0	0.8	0.0	129.0
1984	0.3	0.0	0.0	0.0	0.0	0.0	0.3	0.0	4.7	0.0	0.0	7.7	12.9
TOTAL	99.3	502.4	186.6	152.9	38.5	10.9	16.0	50.7	7.0	41.3	40.4	78.1	1224.1
MEAN	9.0	45.7	17.0	13.9	3.5	1.0	1.5	4.6	0.6	3.8	3.7	7.1	111.3

Table A-3-5 Areal Rainfall at River Mouth

Areal Rainfall at River-Mouth (CA = 1,283 sq.Km)

UNIT = MM/MONTH

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	ANNUAL
1974	2.3	48.2	5.0	1.7	0.0	0.2	1.3	2.9	1.7	7.9	0.0	0.6	71.8
1975	6.1	59.0	0.4	0.0	1.5	1.2	0.9	13.5	1.9	0.0	0.0	0.0	84.5
1976	20.5	132.9	118.7	55.4	0.0	1.1	3.0	23.1	3.1	2.1	10.8	8.5	379.1
1977	36.8	26.0	1.2	40.1	29.5	18.2	0.0	0.0	0.0	3.2	11.9	0.1	166.8
1978	0.0	39.0	4.9	3.0	0.0	1.5	12.1	24.1	1.3	0.0	0.0	0.1	85.8
1979	21.1	0.7	1.4	4.1	0.3	2.9	3.8	0.0	2.3	13.9	1.7	28.6	80.9
1980	4.7	15.4	17.0	0.3	1.2	0.0	7.9	0.1	0.0	0.0	0.2	8.0	54.8
1981	3.8	0.0	1.1	12.4	16.3	1.5	10.7	0.0	0.0	0.0	0.0	0.0	45.8
1982	0.0	147.1	45.2	0.6	3.7	0.0	0.0	0.9	0.5	0.7	10.3	3.2	212.1
1983	2.6	46.6	31.5	30.0	0.0	0.0	0.3	29.1	0.0	0.0	1.3	0.0	141.4
1984	0.1	0.0	0.0	0.0	0.0	0.0	3.9	0.0	3.8	0.0	0.0	4.4	12.2
TOTAL	98.0	514.8	226.4	147.6	52.5	26.6	43.7	93.7	14.6	27.8	36.2	53.4	1335.3
MEAN	8.9	46.8	20.6	13.4	4.8	2.4	4.0	8.5	1.3	2.5	3.3	4.9	121.4

*** AREAL RATIO ***

DAQIQ = 0.060 KITNAH = 0.083 HAYL(WJ) = 0.129 HAYL(WH) = 0.224 FARFAR = 0.249 SOHAR ≐ 0.255

Rainfall at Sohar

UNIT = MM/MONTH

YEAR	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	ANNUAL
1974	0.6	71.6	2.7	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.2	77.7
1975	2.6	44.5	0.0	0.0	0.0	0.0	0.0	6.2	0.0	0.0	0.0	0.0	53.3
1976	2.5	118.0	51.9	55.2	0.0	0.0	0.0	0.0	0.0	8.2	0.0	17.3	253.1
1977	68.1	26.3	2.6	38.7	10.9	0.0	0.0	0.0	0.0	0.0	17.6	0.2	164.4
1978	0.0	34.2	6.5	2.0	0.0	0.0	3.6	0.0	0.0	0.0	0.0	0.3	46.6
1979	21.4	0.0	1.5	2.3	0.0	0.0	0.0	0.0	0.0	36.6	0.7	58.4	120.9
1980	1.7	2.2	3.9	1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.8	0.0	9.8
1981	4.8	0.2	1.5	18.2	12.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	37.1
1982	0.0	137.2	56.3	2.0	0.0	0.0	0.0	0.0	0.0	0.0	16.6	10.4	222.5
1983	0.2	60.0	37.9	23.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	121.8
1984	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.9	0.0	0.0	9.7	15.0
TOTAL	102.3	494.0	164.8	143.9	23.5	0.0	3.6	6.2	4.9	44.8	35.7	98.5	1122.2
MEAN	9.3	44.9	15.0	13.1	2.1	0.0	0.3	0.6	0.4	4.1	3.2	9.0	102.0

Table A-3-6 Flood Runoff at Proposed Dam Site

Flood Runoff at Proposed Dam Site

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1974	-	2.35	-	-	-	-	-	-	-	-	-	-	2.35
		(0.35)											
		(5.10)											
1975	-	5.45	-	-	-	-	-	0.20	-	-	-	-	5.65
		(3.35)	(1.05)										
		(2.55)	(2.90)	(4.00)				(0.40)					
		(8.60)	(9.85)	(0.15)				(0.20)					
1976	1.30	14.50	13.80	4.15	-	-	-	0.60	-	-	0.35	-	34.70
						(0.20)							
						(0.15)							
1977	0.75	1.10	-	3.30	2.55	0.35	-	-	-	-	0.05	-	8.10
1978	-	2.90	-	-	-	-	-	0.35	-	-	-	-	3.25
1979	0.30	-	-	-	-	-	-	-	-	-	-	0.20	0.50
1980	-	0.20	0.90	-	-	-	-	-	-	-	-	-	1.10
1981	-	-	-	-	0.65	-	0.15	-	-	-	-	-	0.80
		(0.16)											
		(10.50)	(0.24)										
		(6.38)	(0.78)										
1982	-	17.04	1.02	-	-	-	-	-	-	-	-	-	18.06
				(0.30)									
				(0.77)									
1983	-	3.92	0.38	1.07	-	-	-	1.74	-	-	-	-	7.11
1984	-	-	-	-	-	-	-	-	-	-	-	-	-
Total													81.62

Table A-3-7 Flood Runoff at River-Mouth

Flood Runoff at River-mouth

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1974	-	1.37	-	-	-	-	-	-	-	-	-	-	1.37
		(0.04)											
		(1.59)											
1975	-	1.63	-	-	-	-	-	-	-	-	-	-	1.63
		(2.38)	(0.26)										
		(2.28)	(3.27)										
1976	0.40	4.66	3.53	1.46	-	-	-	-	-	-	-	-	10.05
1977	0.70	0.65	-	1.33	0.83	-	-	-	-	-	-	-	3.51
1978		1.13											1.13
1979	0.12	-	-	-	-	-	-	-	-	-	-	0.38	0.50
1980	-	-	0.24	-	-	-	-	-	-	-	-	-	0.24
1981	-	-	0.25	-	-	-	-	-	-	-	-	-	0.25
		(3.14)	(0.22)										
		(1.68)	(0.47)										
1982	-	4.82	0.69	-	-	-	-	-	-	-	-	-	5.51
				(0.10)									
				(0.30)									
1983	-	1.56	0.36	0.40	-	-	-	0.53	-	-	-	-	2.85
Total	1.22	15.82	5.07	3.19	0.83	-	-	0.53	-	-	-	0.38	27.04
Mean													2.704

Table A-3-8 Monthly Rainfall at Muscat

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.		June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Remark
1893	-	39	*	*	*	*	*	*	*	*	1	13	*	British Emb.
94	11	57	7	5	-	-	8	-	-	-	-	31	119	"
95	106	16	65	-	-	-	-	-	-	-	18	-	205	"
96	63	4	39	-	-	-	-	-	-	4	77	-	187	"
97	13	16	1	-	-	-	-	-	-	-	-	-	30	"
98	2	3	36	-	-	64	-	-	-	-	14	3	122	"
99	-	7	40	-	-	-	-	-	-	-	-	1	48	"
1900	64	34	17	-	-	-	-	-	-	-	23	63	201	"
01	-	12	28	-	-	-	-	-	-	-	-	13	53	"
02	-	7	-	7	-	-	-	-	-	25	-	13	52	"
03	10	-	-	11	-	1	-	-	-	-	1	3	26	"
04	-	3	3	-	-	-	-	-	-	-	18	1	25	"
05	31	46	56	-	-	-	-	-	-	-	5	2	140	"
06	15	33	37	-	-	6	-	1	-	-	-	40	132	"
07	6	79	-	22	-	-	-	-	-	-	5	4	116	"
08	6	-	10	3	-	-	-	-	-	-	-	5	24	"
09	115	-	-	-	-	-	-	-	-	-	-	54	169	"
1910	24	-	11	-	-	-	-	-	-	-	-	38	73	"
11	67	3	7	-	-	-	-	-	-	-	18	6	101	"
12	60	12	0	97	-	-	-	7	-	-	5	25	206	"
13	-	99	22	-	-	-	-	-	-	-	-	14	135	"
14	3	42	2	-	-	9	3	1	-	14	45	22	141	"
15	7	1	3	32	-	-	-	-	-	-	-	7	50	"
16	98	30	5	98	-	-	-	15	-	20	-	-	266	"
17	60	19	-	2	-	-	-	-	-	-	-	24	105	"
18	4	-	10	8	-	-	-	-	-	-	-	39	61	"
19	22	22	20	-	4	-	-	-	-	-	-	-	68	"
1920	6	14	1	-	-	-	-	-	-	-	4	-	25	"
21	4	-	-	-	-	-	-	-	-	-	25	15	44	"
22	6	6	-	-	-	-	-	-	-	-	-	-	12	"
23	7	8	-	36	-	-	-	-	-	-	20	39	110	"
24	3	6	-	-	-	-	-	-	-	-	-	19	28	"
25	7	2	5	-	-	-	-	-	-	44	1	-	59	"
26	25	-	9	6	-	-	-	-	-	-	-	32	72	"
27	-	17	-	10	-	-	4	-	-	-	10	9	50	"
28	47	56	-	-	-	-	-	-	-	-	53	19	175	"
29	8	3	-	-	-	-	-	-	-	-	34	116	161	"
1930	142	1	1	6	-	-	-	-	-	-	-	-	150	Ministry of
31							No Record Available							Defence
32							"							
33							"							
34							"							
35							"							
36	143	6	28	-	-	-	-	-	-	-	12	-	189	British Emb.
37	29	45	-	-	-	-	-	-	-	-	-	27	101	"
38	-	-	-	-	-	-	-	-	-	10	-	20	30	"
39	-	75	-	-	-	-	-	-	-	-	-	23	98	"
1940	21	-	10	-	-	-	-	-	-	-	-	55	86	"

Table A-3-8 Monthly Rainfall at Muscat (Contined)

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Remark
41	-	0.5	13.5	20	-	-	-	-	-	-	-	-	34	British Emb.
42	4	29	-	-	-	-	-	-	-	-	-	8.5	41.5	
43	87	*	*	*	*	*	*	*	*	*	*	*	*	
44	*	*	*	*	*	*	*	*	*	*	*	166	*	
45	1	-	-	-	-	*	*	*	*	*	*	*	*	
46	-	*	*	-	-	-	-	-	-	-	-	14	*	
47	-	2.5	14	-	-	-	-	*	*	*	*	*	*	
48	29	44.5	25	2.5	-	-	*	*	*	*	*	*	*	
49	*	*	*	*	*	*	*	*	*	*	*	*	*	
1950	16.5	4	1.5	1.5	2.5	-	0.5	*	*	*	*	*	*	
51	-	-	62	-	-	-	-	-	-	1	-	6	69	
52	52	-	-	1	-	-	-	-	-	-	-	16	69	
53	2	27	-	-	5	-	-	-	-	-	-	15	49	
54	11	10	1	2	-	-	-	-	-	-	-	1	25	
55	97	7	70	-	-	-	-	-	-	-	-	14	188	
56	12.2	13.2	-	0.5	-	-	37.1	-	-	-	-	171	234	
57	109.0	-	-	62	9	-	-	-	-	-	9	36	225	
58	53.0	-	-	-	2	-	5	-	-	-	-	16	76	
59	10.9	-	21.8	2.5	-	-	-	-	-	-	68.6	12.7	115.5	
1960	13.7	-	-	18.3	36.8	-	-	-	-	-	24.1	16	108.9	
61	2.3	-	2.3	12.2	14.5	-	0.5	-	-	-	1.8	10	34.6	
62	19.8	-	-	6.9	-	-	72.1	-	-	-	-	20.3	119.1	
63	-	2.3	-	24.9	94	-	-	-	-	-	8.4	11.4	141.0	
64	11.4	-	10.4	-	-	-	-	-	-	-	-	5.1	26.9	
65	22.6	-	-	83.1	-	-	-	-	-	-	2.0	-	107.7	
66	-	88	1	7	-	-	-	-	-	-	-	-	96.6	P.D.O.
67	-	-	0.6	7.9	6.2	-	1.9	0.1	-	-	-	5.9	22.6	"
68	22.8	90.3	2.5	-	-	-	-	-	-	-	-	29.2	144.8	
69	56.4	5.6	12.2	-	-	-	-	-	-	-	-	-	74.2	
1970	31.7	-	-	-	-	-	-	110	-	-	-	-	141.7	
71	15.2	-	-	-	-	-	-	-	-	-	37.5	44.9	97.6	
72	103.9	95.5	47.7	-	-	-	18.3	-	-	-	-	-	265.4	
73	96.8	-	-	-	-	-	-	-	-	-	-	-	96.8	
74	-	3.0	-	0.3	-	-	-	-	-	-	-	20.0	23.3	DAR SITE
75	2.7	79.9	-	-	-	-	-	-	-	-	-	1.0	83.6	"
76	51.0	56.0	66.3	43.3	-	-	10.0	-	-	-	-	2.0	228.6	"
77	64.5	22.0	5.7	32.0	-	6.8	-	-	-	-	56.9	-	187.9	"
78	12.4	39.4	13.4	1.0	-	-	-	-	-	-	0.6	-	66.8	"
79	6.9	-	-	-	-	-	-	-	-	-	-	32.3	39.2	"
1980	0.8	2.6	0.3	-	-	-	-	-	-	-	-	-	3.7	"
81	4.2	-	16.0	-	103	-	-	-	-	0.5	-	-	123.7	"
82	2.9	59.6	35.1	0.9	-	-	-	-	-	-	4.5	29.5	132.5	"
83	-	25.6	4.5	46.7	-	-	-	0.9	-	-	-	3.6	80.3	"

Source: Water Resources Department

Notes: 1. - no rainfall
2. * no data

Table A-3-9 Characteristics of Maximum Rainfall

Characteristics of Maximum Rainfall

Rain Gauge	River	Altitude (m)	Record	Annual Rainfall Maximum (1)	Annual Rainfall Minimum	Annual Rainfall Average	(Rate)	Monthly Maximum (1)	Monthly Maximum (2)/(1)	Daily Maximum (3)	Daily Maximum (3)/(1)
Wadi Jizzi Basin											
Daqiq	Al Jizzi	800	1974-84	630.8	17.5	182.1	(1.88)	234.8	(0.37)	74.3	(0.12)
Kitnah	"	655	1974-84	353.1	6.8	108.7	(1.12)	140.8	(0.40)	78.0	(0.22)
Hayl (W.J)	"	500	1974-84	328.5	19.0	104.6	(1.08)	155.7	(0.47)	71.0	(0.22)
Hayl (W.H)	"	430	1974-84	464.0	0.0	116.2	(1.20)	190.9	(0.41)	64.9	(0.14)
Farfar	"	560	1974-84	406.1	0.0	144.1	(1.49)	155.0	(0.38)	67.0	(0.16)
Sohar	"	15	1974-84	253.1	9.8	102.0	(1.05)	137.2	(0.54)	91.0	(0.36)
Basin Average at Dam			1974-84	420.8	11.8	127.8	(1.32)	150.9	(0.36)	54.4	(0.13)
Other Basin											
Ghozaifah	Ahin	480	1974-83	281.1	51.6	115.1	(1.19)	125.3	(0.45)	48.6	(0.17)
Qufays	"	600	1974-83	345.7	31.2	145.8	(1.50)	201.5	(0.58)	59.4	(0.17)
Haybi	"	570	1974-83	268.6	28.3	148.9	(1.54)	163.0	(0.61)	59.0	(0.22)
Hawqayn			1975-83	314.0	39.0	160.2	(1.65)	131.0	(0.42)	98.0	(0.31)
Rustaq		350	1974-83	377.6	78.2	172.8	(1.78)	123.8	(0.33)	81.5	(0.22)
Nakhl			1974-83	267.7	16.0	139.4	(1.44)	145.0	(0.54)	70.0	(0.26)
Muscat	Aday		1893-83	266.0	3.7	101.5	(1.05)	171.0	(0.64)	-	-
-do-	"		1974-83	228.6	3.7	97.0	(1.00)	103.0	(0.45)	-	-
Quryat	Miglas		1975-84	339.5	35.0	145.7	(1.50)	140.0	(0.41)	140.0	(0.41)

Table A-3-10 Calculation of Recharge Rate Based on Hydrological Analysis

Calculation for Recharge Rate Based on Hydrological Analysis

Description	Before Project	After Project
(1) Areal Rainfall at Dam Site (mm)	128	128
(2) Areal Rainfall at River Mouth (mm)	121	121
(3) Analyzed Flood Runoff at Dam (CMC)	6.63	6.63
(4) Expected Flood Runoff at River Mouth (MCM)	9.94 1/	9.94 1/
(5) Analyzed Flood Runoff at River Mouth (MCM)	3.47	0.0
(6) Flood Runoff below Dam (MCM): (4) - (3)	3.31	3.31
(7) Total Recharge below Dam (MCM): (4) - (5)	6.47	9.94
(8) Recharge Rate (%): (7)/(4) x 100	65.1	100.0

Remark: 1/ (3) x Areal Ratio x Rainfall Ratio
= 6.63 x 1,283/812 x 121/128

A-4 Outlfow Capacity of Outlet Conduit

A recharge potential at the downstream Wadi course of the dam was estimated to get the optimal recharge rate for the development plan.

Figure A-4-1 will provide a theoretical curve to represent relationship between peak flood discharges at the dam site and river mouth when the equivalent amount of areal rainfalls are expected in both the drainage basin.

Theoretical Curve for Peak Flood Discharges

Areal Rainfall (mm)	Proposed Dam-Site Specific Runoff (mm)	Runoff Volume (MCM)	Peak Discharge (m^3/sec)	River - Mouth Specific Runoff (mm)	Runoff Volume (MCM)	Peak Discharge (m^3/sec)
11.5	0.21	0.1705	17.0	0	0	0
13.0	0.30	0.2436	24.3	0.09	0.1155	4.7
15.0	0.42	0.3410	34.0	0.20	0.2566	10.4
20.0	0.83	0.6740	67.3	0.45	0.5774	23.4
25.0	1.35	1.0962	109.4	0.68	0.8724	35.3
30.0	2.00	1.6240	162.1	0.90	1.1547	46.7
35.0	2.73	2.2168	221.2	1.11	1.4241	57.6
40.0	3.50	2.8420	283.6	1.31	1.6807	68.0
45.0	4.30	3.4916	348.5	1.51	1.9373	78.4
50.0	5.20	4.2224	421.4	1.70	2.1811	88.2
60.0	7.33	5.9520	594.0	2.09	2.6815	108.5
70.0	9.85	7.9982	798.2	2.47	3.1690	128.2
80.0	12.60	10.2312	1021.0	2.82	3.6181	146.4

Note : Peak discharges are converted from runoff volume by use of ratios of peak discharge to flood volume.
(1) At dam site ratio = 9.9795×10^{-5}
(2) At river mouth ratio = 4.045×10^{-5}

However, analyzed flood runoffs are not always plotted on the theoretical curve, but plotted elsewhere around the curve, mainly due to difference in catchment's rainfall. These relations are also shown in Figure A-4-1, meaning that (1) when the entire Wadi Jizzi basin receives the same amount and distribution of areal rains the dam can discharge 17.0 cu.m/sec of water without wasting any portion of surface water into the sea, and (2) when downstream area receives a greater amount of rains than upstream the dam can release a smaller amount of discharge not so as to waste water into the sea. In consideration of such condition, an envelope curve was so drawn, as shown in Figure A-4-1, as to enclose all the points distributed above the theoretical curve, for the purpose of finding optimal release rate of stored water in the dam.

Consequently, 13.0 cu.m/sec was obtained from the study as the optimal rate of release.

An outlet conduit is to be embedded beneath the dam body in consideration of the existing condition of the water route, effective recharging of groundwater in the downstream river deposits, and necessity to flush-out the fine sediments around the entrance of the outlet conduit.

A circular shape steel pipe conduit with the inner diameter of 1,500 is planned for the outlet facilities of the detention dam. At the entrance of outlet conduit, trush-rake is equipped in order to prevent the objectionable materials flowing into the conduit.

On outlet discharge of this conduit at the varying reservoir water levels can be obtained as a pipeline flow by the following equation.

$$Q = \frac{\sqrt{2g} \cdot A}{\sqrt{fv + fe + fr + fb}} \cdot \sqrt{H}$$

$$A = 0.7854 \ D^2$$

where, Q; outlet discharge of conduit as a pipeline flow

g; gravitational acceleration

A; flor area = $0.7854 \ D^2 = 1.767$

D; inner diameter of outlet conduit pipe, adopted by 1.50m fv; coefficient of changing velocity loss, adopted by 1.0

fe; coefficient of the entrance loss adopted by 0.5

fr; coefficient of friction loss, $fr = 124.5xn^2/D^{4/3}xL = 2.056$

fb; coefficient of bend loss, fb = 0.99

n; coefficient of roughness, adopted by 0.015

L; length of conduit, adopted by 126.0 m

H; total head, measured from top surface at the end of conduit

Based on the results of the above-mentioned computation, the relationship between an outlet discharge and a reservoir water level can be estimated as follows:

$$Q = 3.669 \sqrt{H}$$

$$Q_{max} = 3.669 \sqrt{163.9 \text{ m} - 151.9 \text{ m}} = 12.7 \text{ m}^3/\text{sec}$$

Table A-4-1 Single Event Flood Runoffs at Dam Site and River Mouth

Single Event Flood Runoffs at Dam Site and River Mouth

Unit: MCM and (cu.m/sec)

YEAR	JAN.	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	NOV.	DECEMBER
1974		(190.4) 1.908 (71.1) 1.758								
1975		(28.3) (413.3) 0.284 4.141 (2.1) (82.5) 0.051 2.040						(16.2) 0.162 (-) -		
1976	(105.4) 1.056 (20.8) 0.513	(271.4) (206.7) (696.9) 2.720 2.071 6.983 (123.5) (118.3) 3.054 2.925	(85.1) (235.0) (798.2) 0.853 2.355 7.998 (13.5) (169.7) 0.334 4.195	(324.1) (12.2) 3.248 0.122 (75.8) (-) 1.873 -				(32.4) (16.2) 0.325 0.162 (-) (-) - -	(28.3) 0.284 (-) -	
1977	(60.8) 0.609 (36.3) 0.898	(89.1) 0.893 (33.7) 0.834		(267.5) 2.680 (69.0) 1.706	(206.7) 2.071 (43.1) 1.065	(16.2) (12.2) 0.162 0.122 (-) (-) - -			(4.1) 0.041 (-) -	
1978		(235.0) 2.355 58.7 1.450						(28.3) 0.284 (-) -		
1979	(24.3) 0.244 (6.2) 0.154									(16.2) (-) 0.162 - (-) (19.7) - 0.488
1980		(16.2) 0.162 (-) -	(73.0) 0.731 (12.5) 0.308							
1981					(52.7) 0.528 (12.9) 0.321		(12.2) 0.122 (-) -			
1982		(13.0) (850.9) (517.0) 0.130 8.526 5.181 (-) (163.0) (87.2) - 4.029 2.155	(19.5) (63.2) 0.195 0.633 (11.4) (24.4) 0.282 0.603							
1983		(317.6) 3.183 (80.9) 2.001	(30.8) 0.309 (18.7) 0.462	(24.3) (62.4) 0.244 0.625 (5.2) (15.6) 0.128 0.385				(141.0) 1.413 (27.5) 0.680		

Remark: The upper row shows single event flood volume in MCM and peak discharge (parenthesized) in cum/sec at the proposed dam site, while the lower row presents those figures at the Wadi Fizzi river mouth.

Figure A-4-1 Food Peak Discharges at Dam and River Mouth

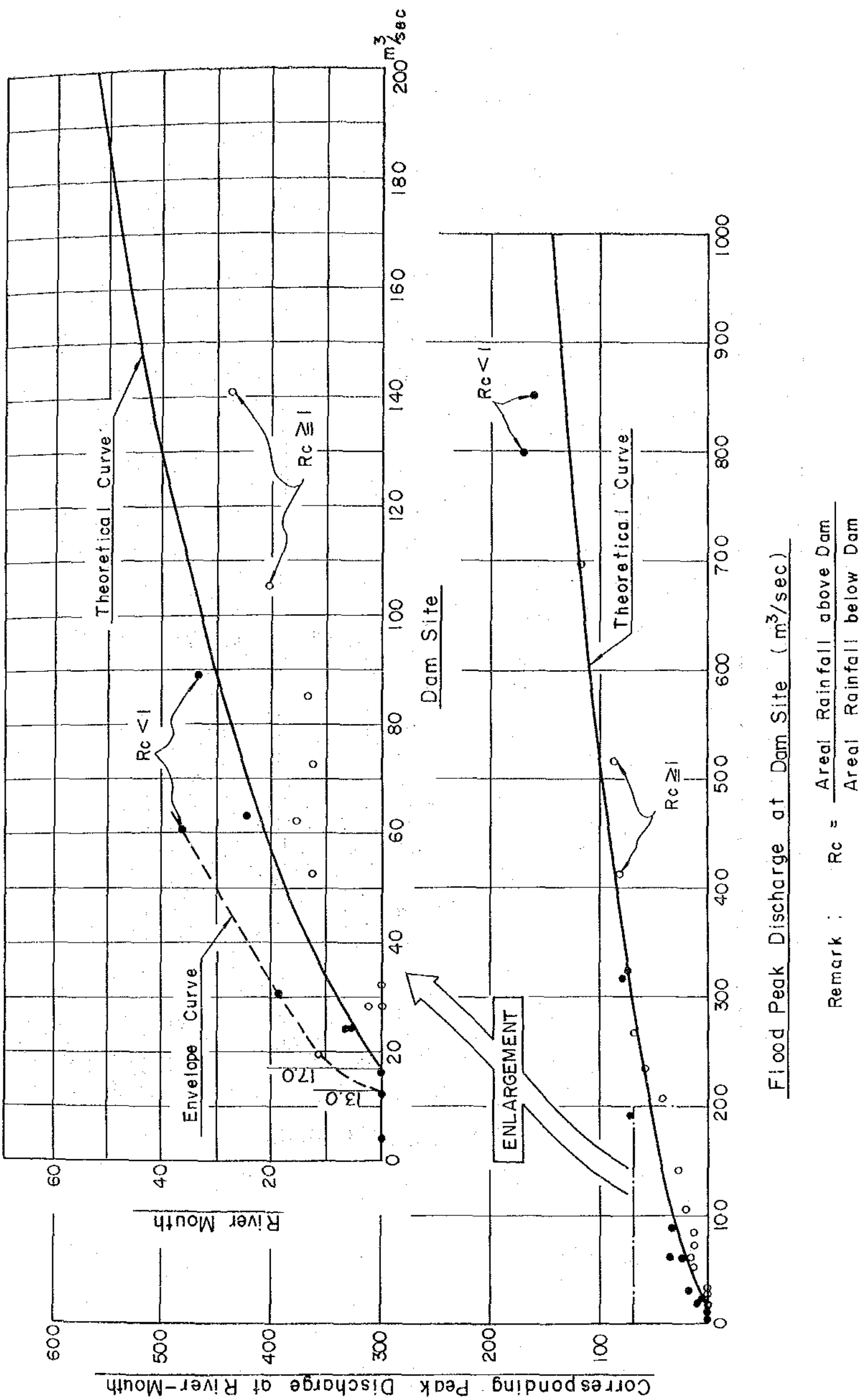


Figure A-4-2 Reservoir Emptying Time

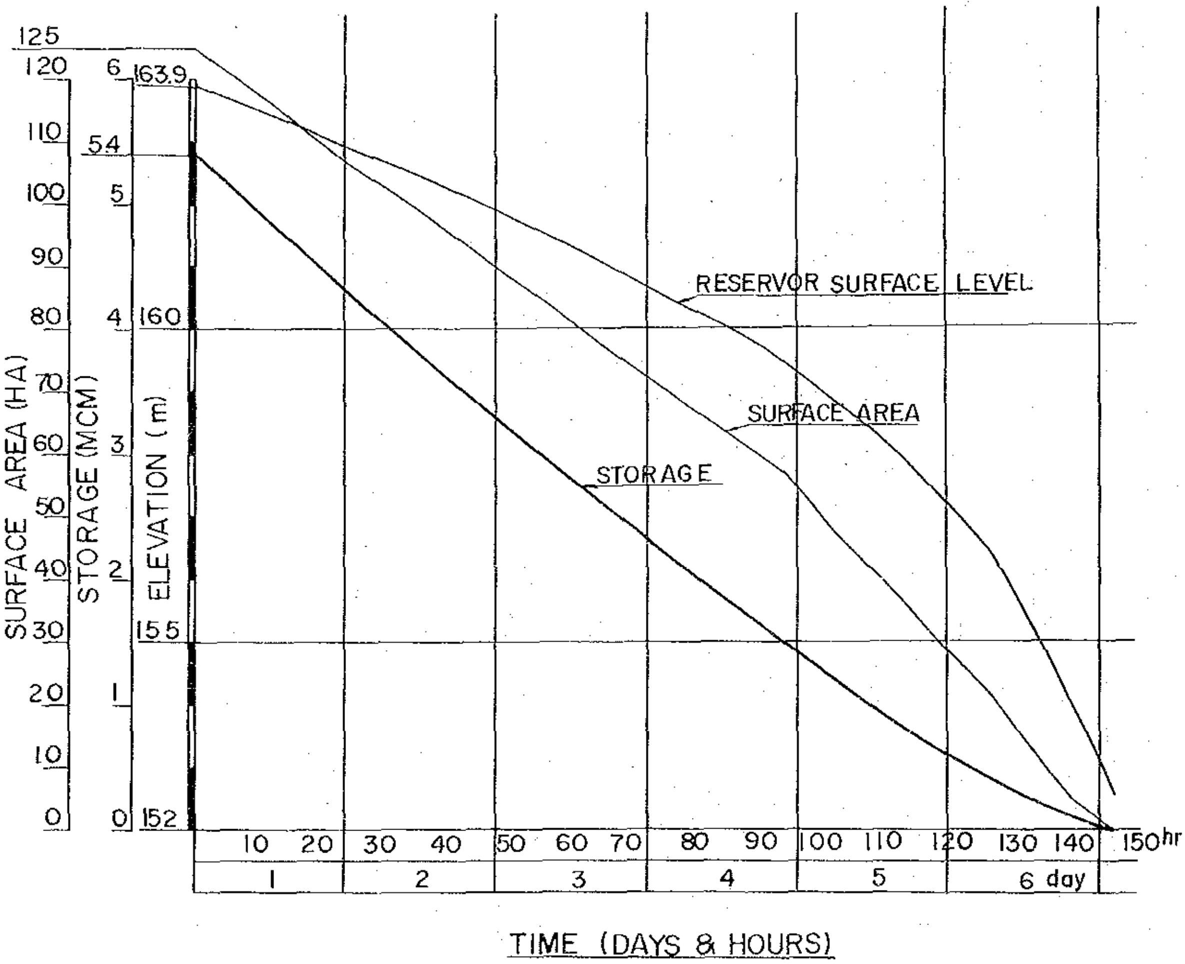
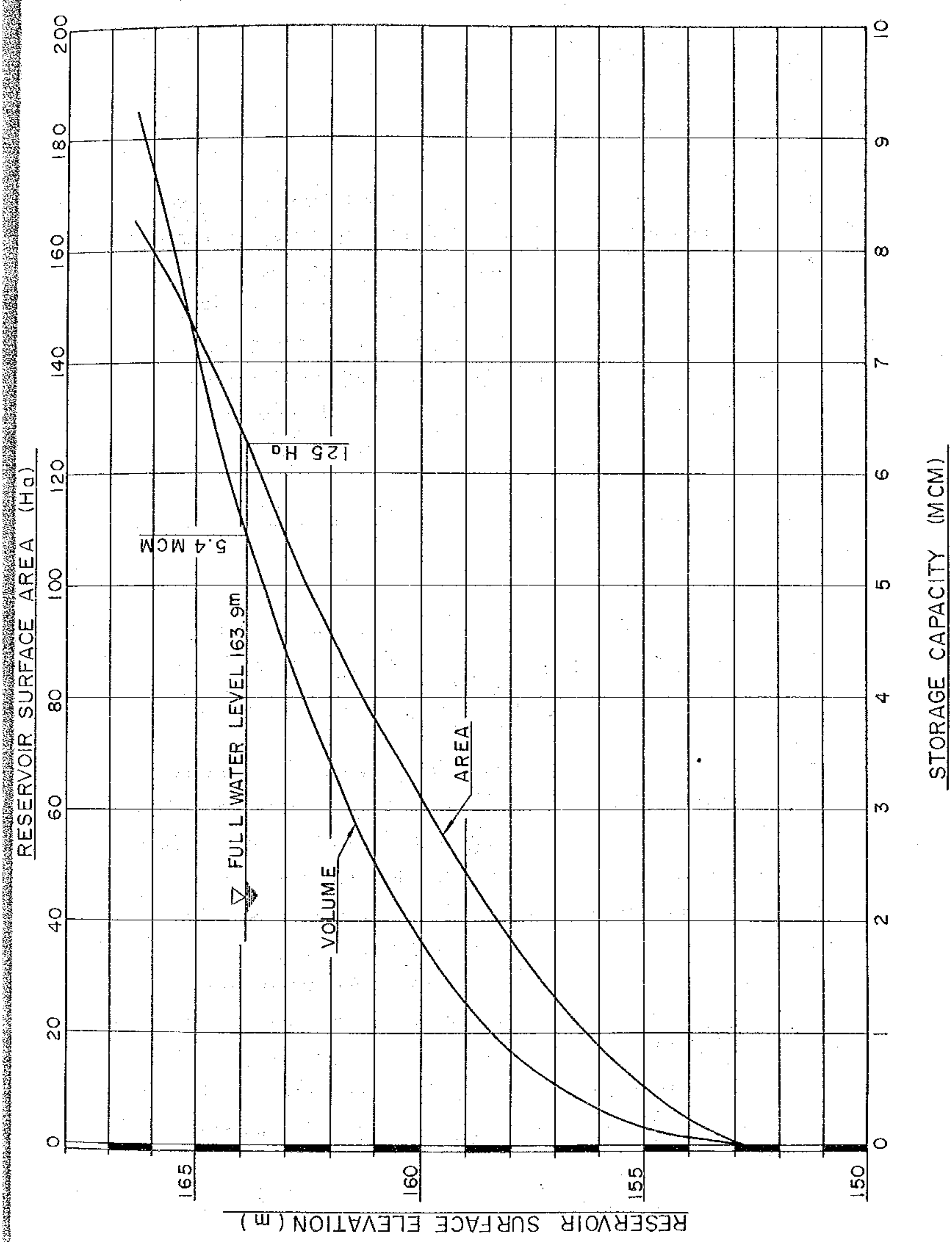


Figure A-4-3 Reservoir Capacity Curve



A-5 Irrigation Water Requirement

Present Irrigation Conditions

1. Existing Irrigation System

The irrigation system in the area is predominantly basin irrigation. Each basin is either circular one meter to three meters in diameter and 10 centimeters to 60 centimeters deep holding one tree, or rectangular five square meters to 30 square meters in size and a few centimeters lower than the conveyance canal holding one or two trees and occasionally undergrown by feed crops. The later may almost be called the border-strip irrigation.

There are three types of water conveyance systems to lead water to these basins.

i) direct diversion from the main and lateral canals.
ii) a series of basins used as a conveyance canal.
iii) combination of i) and ii).

No. i) is the system where the water is directly diverted from the conveyance canal to the basin. A few farms adopt this system. However, seldom is the main canal straight or wellkept for efficient conveyance of water, causing excessive seepage and evaporation losses from the canal. Yet this system has the potentiality to be most efficient of the three, as far as the water saving is concerned, because there is no intermediate conveyance losses before the cropping area. No. ii) is the system where water is fed directly into the first basin. After filling the first basin the water flows into the second one and then the third to the last basin. This is the most inefficient method of the three as the percolation loss as the beginning of the system is enormous and application rate towards the end of the system is insufficient. There are varying degrees of combination of the above two. If sensibly laid out this No. iii) system proves to be the most economical of the three as this saves the expensive construction of the long main and lateral canals and also the spaces occupied by them. Most of the farm lands employ the last system. But the basin-to-basin conveyance distance is usually too much (30 meters to 60 meters) resulting in the deficiency described above for No. ii).

Apart from the basin irrigation on the border-strip and furrow irrigation seem to gain popularity especially among the newly developed large scale farms along the highway. The conveyance canals are generally straight and well kept. But there is still a lot of room for improvement in canal routing and quality as well as for introduction of other irrigation facilities.

2. Present Irrigation Practices

The procedure of irrigation in the survey area varies little from place to place, be it of different efficiency, i.e., pumped water is let to the crop area by gravity through ditches, furrows and basins. All the irrigation water in the area is presently supplied by well water including in the village of Falaj Al Awhi on the north-western corner where the irrigation water was formerly supplied by the Falaj. The pumps are usually installed singly or in a pair in a pit about five meters deep. Suction pipes extend into the adjoining well of 80 centimeters to 120 centimeters diameter. The depth to the water surface was observed to be about seven to 10 meters below ground surface. The pump sizes are mainly 3" x 3" and sometimes 4" x 3".

Water is first poured into a rectangular concrete basin of the size 1.2 m x 1.8 m to 2 m x 3 m and 20 centimeters to 50 centimeters deep. The top rim of the basin is 60 centimeters to 70 centimeters higher than the ground. Water is then let out through an elevated concrete ditch 15 centimeters to 20 centimeters wide and 10 centimeters to 15 centimeters deep. The length of this concrete ditch is usually less than 10 meters and from there on a crude earth ditch continues. The earth ditch is usually winding and not of a uniform shape. Water is diverted by breaking a side of the ditch and clogging the immediate downstream by filling mud into the ditch. This daily operation helped in deforming and deteriorating the ditch, such a ditch will cause more evaporation losses due to low speed of flow and much seepage due to constantly disturbed and softened ditch surfaces.

Farmers seem to know approximately the right way of irrigating each crop after many years or generations of experience. Basin irrigation is adopted for tree crops, border-strip for fodder crops and furrow irrigation for various vegetables. However, it was almost always the case that the conveyance route was arranged in a rather complicated and wasteful way with a lot of detours which could be simplified and rationalized. These detours are possibly the consequences of the efforts to achieve best results in uniform water distribution. By the time the purpose was achieved the conveyance route was apparently grossly elongated. It is therefore important to study the topography carefully when rectifying the canal route.

3. Irrigation Efficiency

The classification according to the crop condition described in previous paragraph is reconsidered here in terms of irrigation intensity.

In No. 1 (good condition) area it was observed that water was abundantly applied both in quantity and frequency. In some cases water was applied every day or every other day, where the quantity was estimated to be from 100 percent up to even 500 percent of the consumptive use. On these locations plant growth generally looked good. Possible reasons for it is ample application of fertilizers and good leaching of hazardous salts coupled with sufficient supply of water and favourable soil conditions. Further study on harmful effect of over-irrigation should be made.

In No. 2 (moderate condition) area water application was estimated at between 50 percent to 100 percent of the consumptive use of water. On these farms the irrigation method and facilities were usually poor to fair. Only the beginning part of the irrigation system received ample to excessive supply of water and the end part of it suffered from water deficiency.

No. 3 (poor condition) are obtained only 20 percent to 50 percent of water compared to the consumptive use of the plants. In these areas the farmers attended the farm only occasionally or only part of the farm due to lack of sufficient available water.

The summary of water consumption of each of these areas is given below:

Classification	Area Percentage	Water Application Percentage of Consumptive Use	Average
	(%)	(%)	(%)
Good area	20	100	100[1/]
Moderate area	49	50 - 100	75
Poor area	31	20 - 50	35

1/ Some inconsistencies involved in the Feasibility Study Report were revised. The maximum water application percentage of consumptive use was considered to be 100%.

Under these conditions the overall irrigation efficiencies are considered to be 50 percent. Most of the losses occur through deep percolation and evaporation caused by inefficient and deficient conveyance canals and partly due to over-irrigation.

4. Present Irrigation Water Requirement

From the hydrogeological study, about 17 kilometers long coastal plain extending from the Wadi Khadaq to the Village of Majis was determined to be beneficial areas covered by the Wadi Jizzi river basin. The gross cultivated area within this coastal plain is about 3,830 hectares and its net cultivation area is 2,640 hectares. The water resources for irrigation in these areas are wells with a depth of 20 meters to 30 meters on average, and numerous small scale pumps are installed for lifting groundwater as irrigation water.

Since the estimation of present annual amounts used for irrigation by means of pumping operation during a year was found out to be difficult due to numerous pump installation as explained above, the amounts were estimated based upon the cropping areas and consumptive use of each crop.

a) Present Cropping Area and Cropping Pattern

According to the field survey and collected data on the existing land use, the existing cropping acreage is summarized as shown below and its cropping pattern is shown in Figure A-5-1 and A-5-2.

Existing Cropping Acreage

Vegetable			
Oion	:	96 ha	(3.4%)
Garlic	:	25	(0.9)
Tomato	:	16	(0.6)
Potato	:	15	(0.5)
Okra	:	8	(0.3)
Others	:	23	(0.8)
Fruit Crop			
Dates	:	1,820	(64.2)
Lime	:	309	(10.9)
Banana	:	128	(4.5)
Mango		83	(2.9)
Feed Crop			
Alfalfa	:	211	(7.4)
Sorghum	:	101	(3.6)
Total	:	2,835	(100 %)

b) Irrigation Water Requirement

1) Potential Evapo-transpiration

The reference crop potential evapo-transpiration (ETPc), which is generally recognized as a fairly reliable index for calculating consumptive use, can be determined by a number of methods, such as the evaporation measurement with evaporation pan and the application of empirical formula based on the climatological data. In the Project, the monthly evapo-transpiration was estimated on the monthly basis, by applying modified Penman method based on the climatological data observed at Sohar meteorological station.

2) Crop Water Supply Requirements

(a) Crop Evapo-transpiration

Crop evapo-transpiration (ETc) (consumptive use of crops) of each crop has been estimated by multiplying the estimated ETPc values by crop coefficients which express the relationship between the reference crop potential and the actual evapo-transpiration during the vegetative stage of the crops. The crop evapo-transpiration (ETc) of each crop was estimated based upon the above procedures and is shown in Tables A-5-2(1) to A-5-2(4).

(b) Water Supply Requirements

For the estimation of water supply requirement for each crop, the following equation has been applied:

$$V = \frac{10}{EP} \left[\frac{A\ (ETc - RE)}{1 - LR} \right]$$

where;
- A : area (ha)
- ETc : crop evapo-transpiration (mm)
- RE : effective rainfall (mm)
- EP : project irrigation efficiency
- LR : leaching requirement

In the estimation, effective rainfall and leaching requirements have been accounted based upon the following bases.

Effective Rainfall

The effective rainfall for crops was estimated on the monthly basis by applying the criteria provided in FAO Irrigation and Drainage Paper No. 24, Table 34 "Average Monthly Effective

Rainfall as related to Average Monthly EPTc and Mean Monthly Rainfall": (see Table A-5-3)

Leaching Requirements

With regard to Water Quality, the groundwater in the Wadi Jizzi plain, which is the water sources of irrigation for the Project, has been found to need some countermeasure for salinity control such as leaching. The leaching requirements are the minimum amount of irrigation water that must be supplied and drained through the root zone to keep the soil salinity at a specific level. For sandy loam soils with good drainage and in the areas where rainfall is small, the leaching requirements can be obtained from the following equation.

Leaching requirement (LR) for drip and high frequency sprinkler:

$$LR = \frac{EC(W)}{2MAX.\ EC(E)} \cdot \frac{1}{LE}$$

where, EC(W) : electric conductivity of irrigation water, 0.56 mmhos/cm (average of five water samples)

Max. EC(E) : maximum tolerable electircal conductivity of the soil saturation extract for crops, derived from FAO Irrigation and Drainage Paper No. 24, Table 36.

LE : leaching efficienty, 0.8 (sandy loan)

Water supply requirement for each crops were calculated employing the above mentioned procedures as shown in Tables A-5-2(1) to A-5-2(4), and the results are summarized with respect to crops and land use as given in Tables A-5-4 and A-5-5, respectively. Consequently the annual consumption of irrigation water for the existing cultivated areas of 2,835 ha is estimated at 11.0 MCM, taking present status of irrigation water application into consideration.

Table A-5-1 Penman's Reference Crop Evapotraspiration

Penman's Reference Crop Evapotranspiration

Place: SOHAR Latitude: 24°20' Longitude: 56°40' Altitude: 15m

Factor	Unit	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Note/Equation
Tmean	°C	18.2	18.6	22.0	25.9	30.1	32.1	32.6	31.6	29.6	25.9	21.7	19.6	given data
RHmean	%	68.5	68.4	67.4	59.9	55.2	61.8	70.5	74.0	69.6	63.7	66.9	70.5	- do -
ea	mbar	20.9	21.4	26.4	33.4	42.7	47.9	49.2	46.5	41.5	33.4	26.0	22.8	
ed	mbar	14.3	14.6	17.8	20.0	23.6	29.6	34.7	34.4	28.9	21.3	17.4	16.1	ea x RHmaen/100
(ea-ed)	mbar	6.6	6.8	8.6	13.4	19.1	18.3	14.5	12.1	12.6	12.1	8.6	6.7	(1)
U_2	Km/day	56.4	61.0	67.2	70.3	73.3	75.2	85.3	84.9	71.0	58.6	49.6	46.3	given data
f(u)	-	0.42	0.43	0.45	0.46	0.47	0.47	0.50	0.50	0.46	0.43	0.40	0.40	$0.27(1+U_2/100)$
1 - W	-	0.34	0.33	0.29	0.25	0.22	0.20	0.19	0.20	0.22	0.25	0.29	0.32	
(1-W)f(u)(ea-ed)	mm/day	0.94	0.96	1.12	1.54	1.97	1.72	1.38	1.21	1.28	1.30	1.00	0.86	(2)
Ra	mm/day	10.1	11.8	13.9	15.4	16.4	16.6	16.5	15.8	14.5	12.6	10.6	9.6	
n	hr/day	7.2	7.7	8.1	8.7	10.3	9.9	8.7	8.5	9.3	9.1	8.4	7.6	given data
N	hr/day	10.7	11.3	12.0	12.7	13.3	13.6	13.5	13.0	12.3	11.6	10.9	10.6	
R_s	mm/day	5.92	6.97	8.17	9.12	10.45	10.19	9.44	9.11	9.11	8.09	6.73	5.84	(0.25_0.50n/N)Ra
Rns	mm/day	4.4	5.2	6.1	6.8	7.8	7.6	7.1	6.8	6.8	6.1	5.1	4.4	0.75Rs
fTmean	-	14.2	14.4	15.0	15.9	16.7	17.2	17.4	17.1	16.6	15.9	14.9	14.5	
f(ed)	-	0.17	0.17	0.15	0.14	0.13	0.10	0.08	0.08	0.10	0.14	0.16	0.16	$0.34-0.044\sqrt{ed}$
f(n/N)	-	0.70	0.71	0.70	0.71	0.79	0.76	0.69	0.69	0.78	0.81	0.79	0.75	0.1+0.9n/N
Rnl	mm/day	1.7	1.7	1.6	1.6	1.7	1.3	1.0	0.9	1.3	1.8	1.9	1.7	f(Tmean)f(ed)f(n/N)
Rn	mm/day	2.7	3.5	4.5	5.2	6.1	6.3	6.1	5.9	5.5	4.3	3.2	2.7	Rns-Rnl
W	-	0.66	0.67	0.71	0.75	0.78	0.80	0.81	0.80	0.78	0.75	0.71	0.68	1-(1-W)
W.Rn	mm/day	1.8	2.3	3.2	3.9	4.8	5.0	4.9	4.7	4.3	3.2	2.3	1.8	(3)
(1)+(3)		2.7	3.3	4.3	5.4	6.8	6.7	6.3	5.9	5.6	4.5	3.3	2.7	(2)+(3)
c		1.06	1.08	1.10	1.12	1.13	1.13	1.13	1.13	1.12	1.10	1.07	1.06	
ET_0	mm/day	2.9	3.6	4.7	6.0	7.7	7.6	7.1	6.7	6.3	5.0	3.5	2.9	
- do -	mm/month	89.9	100.8	145.7	180.0	238.7	228.0	220.1	207.7	189.0	155.0	105.0	89.9	

Table A-5-2 (1) Unit Irrigation Water Requirement

Onion: MEC(E) = 8, LR = 0.044

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Sep.	0.4	189.0	1.00	189.0	-	0.10	395
Oct.	4.1	155.0	1.00	155.0	2.8	0.40	1,274
Nov.	3.2	105.0	0.70	73.5	2.3	1.00	1,489
Dec.	9.0	89.9	0.75	67.4	6.4	1.00	1,276
Jan.	9.3	89.9	0.65	58.4	6.3	0.66	720
	26.0				17.8		5,154

Garlic: MEC(E) = 15, LR = 0.023

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Sep.	0.4	189.0	0.35	66.1	-	0.50	677
Oct.	4.1	155.0	0.50	77.5	3.0	1.00	1,525
Nov.	3.2	105.0	0.70	73.5	2.3	1.00	1,457
Dec.	9.0	89.9	0.75	67.4	6.4	1.00	1,249
Jan.	9.3	89.9	0.80	71.9	6.8	1.00	1,332
Feb.	44.9	100.8	0.80	80.6	32.6	1.00	983
Mar.	15.0	145.7	0.80	116.6	12.1	1.00	2,138
Apr.	13.1	180.0	0.70	126.0	10.9	1.00	2,356
May	2.1	238.8	0.50	119.3	1.5	0.50	1,206
	101.1				75.8		12,923

Tomato: MEC(E) = 13, LR = 0.027

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Sep.	0.4	189.0	1.00	189.0	-	0.10	388
Oct.	4.1	155.0	1.00	155.0	2.8	0.10	313
Nov.	3.2	105.0	0.45	47.2	1.4	1.00	942
Dec.	9.0	89.9	0.55	49.4	5.7	1.00	899
Jan.	9.3	89.9	0.85	76.4	6.9	1.00	1,428
Feb.	44.9	100.8	1.00	100.8	33.5	1.00	1,383
Mar.	15.0	145.7	0.85	123.8	12.4	1.00	2,290
Apr.	13.1	180.0	0.70	126.0	10.9	1.00	3,366
May	2.1	238.7	0.65	155.2	0.9	0.50	1,585
	101.1				74.6		11,594

Potato: MEC(E) = 10, LR = 0.035

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Nov.	3.2	105.0	0.45	47.2	1.4	1.00	949
Dec.	9.0	89.9	1.05	94.4	6.5	1.00	1,821
Jan.	9.3	89.9	1.35	121.4	7.6	1.00	2,358
Feb.	44.9	100.8	1.25	126.0	35.6	0.50	937
	66.4				51.1		6,066

Table A-5-2 (2) Unit Irrigation Water Requirement

Okra: MEC(E) = 10, LR = 0.035

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Feb.	44.9	100.8	0.40	40.3	30.1	0.50	105
Mar.	15.0	145.7	0.70	102.0	11.5	1.00	1,875
Apr.	13.1	180.0	0.85	153.0	11.1	1.00	2,941
May	2.1	238.7	0.80	191.0	0.5	1.00	3,947
Jun	0.0	228.0	0.60	136.8	-	1.00	2,835
	75.1				53.3		11,704

Others: MEC(E) = 10, LR = 0.035

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Nov.	3.2	105.0	0.40	42.0	1.6	1.00	837
Dec.	9.0	89.9	0.70	62.9	6.2	1.00	1,175
Jan.	9.3	89.9	0.85	76.4	6.9	1.00	1,440
Feb.	44.9	100.8	0.80	80.6	32.6	1.00	995
Mar.	15.0	145.7	0.60	87.4	11.3	0.50	788
	81.4				58.7		5,236

Dates: MEC(E) = 32, LR - 0.011

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Jan.	9.3	89.9	0.70	62.9	6.5	1.00	1,142
Feb.	44.9	100.8	0.70	70.6	32.0	1.00	779
Mar.	15.0	145.7	0.70	102.0	11.5	1.00	1,829
Apr.	13.1	180.0	0.70	126.0	10.9	1.00	2,328
May	2.1	238.7	0.70	167.1	1.0	1.00	3,359
Jun.	0.0	228.0	0.70	159.6	-	1.00	3,228
Jul.	0.3	220.1	0.70	154.1	-	1.00	3,116
Aug.	0.6	207.7	0.70	145.4	-	1.00	2,940
Sep.	0.4	189.0	0.70	132.3	-	1.00	2,675
Oct.	4.1	155.0	0.70	108.5	2.7	1.00	2,139
Nov.	3.2	105.0	0.70	73.5	2.3	1.00	1,439
Dec.	9.0	89.9	0.70	62.9	6.2	1.00	1,146
	102.0				73.2		26,120

Lime: MEC(E) = 8, LR ≒ 0.044

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Jan.	9.3	89.9	0.50	44.9	6.0	1.00	815
Feb.	44.9	100.8	0.50	50.4	30.3	1.00	420
Mar.	15.0	145.7	0.50	72.8	11.1	1.00	1,291
Apr.	13.1	180.0	0.50	90.0	9.8	1.00	1,677
May	2.1	238.7	0.50	119.3	1.5	1.00	2,465
Jun.	0.0	228.0	0.50	114.0	-	1.00	2,385
Jul.	0.3	220.1	0.50	110.1	-	1.00	2,302
Aug.	0.6	207.7	0.50	103.8	-	1.00	2,173
Sep	0.4	189.0	0.50	94.5	-	1.00	1,977
Oct.	4.1	155.0	0.50	77.5	3.0	1.00	1,559
Nov.	3.2	105.0	0.50	52.5	1.5	1.00	1,068
Dec.	9.0	89.9	0.50	44.9	5.8	1.00	820
	102.0				69.0		18,951

Table A-5-2 (3) Unit Irrigation Water Requirement

Banana: MEC(E) = 8, LR = 0.044

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Jan.	9.3	89.9	0.90	80.9	6.9	1.00	1,548
Feb.	44.9	100.8	0.90	90.7	33.0	1.00	1,207
Mar.	15.0	145.7	0.90	131.1	12.5	1.00	2,481
Apr.	13.1	180.0	0.90	162.0	11.5	1.00	3,149
May	2.1	238.7	0.90	214.8	0.8	1.00	4,477
Jun.	0.0	228.0	0.90	205.2	-	1.00	4,293
Jul.	0.3	220.1	0.90	198.1	-	1.00	4,144
Aug.	0.6	207.7	0.90	186.9	-	1.00	3,911
Sep.	0.4	189.0	0.90	170.1	-	1.00	3,559
Oct.	4.1	155.0	0.90	139.5	3.0	1.00	2,856
Nov.	3.2	105.0	0.90	94.5	1.8	1.00	1,939
Dec.	9.0	89.9	0.90	80.9	6.7	1.00	1,553
	102.0				76.2		35,117

Mango: MEC(E) = 8, LR = 0.044

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Jan.	9.3	89.9	0.70	62.9	6.5	1.00	1,181
Feb.	44.9	100.8	0.70	70.6	32.0	1.00	806
Mar.	15.0	145.7	0.70	102.0	11.5	1.00	1,893
Apr.	13.1	180.0	0.70	126.0	10.9	1.00	2,408
May	2.1	238.7	0.70	167.1	1.0	1.00	3,475
Jun	0.0	228.0	0.70	159.6	-	1.00	3,339
Jul.	0.3	220.1	0.70	154.1	-	1.00	3,223
Aug.	0.6	207.7	0.70	145.4	-	1.00	3,042
Sep.	0.4	189.0	0.70	132.3	-	1.00	2,767
Oct.	4.1	155.0	0.70	108.5	2.7	1.00	2,213
Nov.	3.2	105.0	0.70	73.5	2.3	1.00	1,489
Dec.	9.0	89.9	0.70	62.9	6.2	1.00	1,186
	102.0				73.2		27,021

Alfarfa: MEC(E) = 16, LR = 0.022

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Jan.	9.3	89.9	0.85	76.4	6.9	1.00	1,421
Feb.	44.9	100.8	0.85	85.7	32.8	1.00	1,081
Mar.	15.0	145.7	0.85	123.8	12.4	1.00	2,278
Apr.	13.1	180.0	0.85	153.0	11.1	1.00	2,902
May	2.1	238.7	0.85	202.9	0.3	1.00	4,143
Jun.	0.0	228.0	0.85	193.8	-	1.00	3,963
Jul.	0.3	220.1	0.85	187.1	-	1.00	3,826
Aug.	0.6	207.7	0.85	176.5	-	1.00	3,610
Sep.	0.4	189.0	0.85	160.6	-	1.00	3,285
Oct.	4.1	155.0	0.85	131.7	3.2	1.00	2,628
Nov.	3.2	105.0	0.85	89.3	2.0	1.00	1,785
Dec.	9.0	89.9	0.85	76.4	6.7	1.00	1,425
	102.0				75.5		32,348

Table A-5-2 (4) Unit Irrigation Water Requirement

Sorghum: MEC(E) = 18, LR = 0.019

Month	Rain	ETo	KC	ET(C)	R(E)	A	V
Oct.	4.1	155.0	0.35	54.3	2.2	0.50	531
Nov.	3.2	105.0	0.60	63.0	1.9	1.00	1,246
Dec.	9.0	89.9	1.00	89.9	6.6	1.00	1,699
Jan.	9.3	89.9	1.00	89.9	6.8	1.00	1,694
Feb.	44.9	100.8	0.75	75.6	32.4	1.00	880
Mar.	15.0	145.7	0.60	87.4	11.3	0.50	776
	85.5				61.2		6,825

Table A-5-3 Average Monthly Effective Rainfall as Related to Average Monthly ETcrop and Mean Monthly Rainfall

Average Monthly Effective Rainfall as Related to Average Monthly ETcrop and Mean Monthly Rainfall

Monthly Mean Rainfall (mm)		12.5	25.0	37.5	50.0	62.5	75.0	87.5	100.0	112.5	125.0	137.5
	25	8	16	24								
	50	8	17	25	32	39	46					
	75	9	18	27	34	41	48	56	62	69		
Average	100	9	19	28	35	43	52	59	66	73	80	87
Monthly	125	10	20	30	37	46	54	62	70	76	85	92
ETcrop	150	10	21	31	39	49	57	66	74	81	89	97
in mm	175	11	23	32	42	52	61	69	78	86	95	103
	200	11	24	33	44	54	64	73	82	91	100	109
	225	12	25	35	47	57	68	78	87	96	106	115
	250	13	25	38	50	61	72	84	92	102	112	121

Source: FAO Irrigation and Drainage Paper No. 24 (1977 Edition), Table 34

Remarks: Effective rainfall = Effective rainfall as calculated × Correction factor

Correction factor = 1.04 (for effective storage of 130 mm)

Table A-5-4 Unit Irrigation Water Supply Requirement by Crops

Average Monthly Water Supply Requirement per Hectar by Crops

(Unit: cu.m/ha)

Month	Vegetable							Fruit Crop					Feed Crop		
	Onion	Garlic	Tomato	Potato	Okra	Others	Average	Dates	Lime	Banana	Mango	Average	Alfalfa	Sorghum	Average
	96ha	25ha	16ha	15ha	8ha	23ha	183ha	1,820ha	309ha	128ha	83ha	2,340ha	211ha	101ha	312ha
Jan.	720	1,332	1,428	2,358		1,440	1,059	1,142	815	1,548	1,181	1,122	1,421	1,694	1,509
Feb.		983	1,383	937	105	995	462	779	420	1,207	806	756	1,081	880	1,016
Mar.		2,138	2,290		1,875	788	673	1,829	1,291	2,481	1,893	1,796	2,278	776	1,792
Apr.		2,356	2,366		2,941		657	2,328	1,677	3,149	2,408	2,290	2,902		1,963
May		1,206	1,585		3,947		476	3,359	2,465	4,477	3,475	3,306	4,143		2,802
Jun.					2,835		124	3,228	2,385	4,293	3,339	3,179	3,963		2,680
Jul.								3,116	2,302	4,144	3,223	3,069	3,826		2,587
Aug.								2,940	2,173	3,911	3,042	2,895	3,610		2,441
Sep.	395	677	388				334	2,675	1,977	3,559	2,767	2,634	3,285		2,222
Oct.	1,274	1,525	313				904	2,139	1,559	2,856	2,213	2,104	2,628	531	1,949
Nov.	1,489	1,457	942	949		837	1,246	1,439	1,068	1,939	1,489	1,419	1,785	1,246	1,611
Dec.	1,276	1,249	899	1,821		1,175	1,216	1,146	820	1,553	1,186	1,127	1,425	1,699	1,514
Total	5,154	12,923	11,594	6,066	11,704	5,236	7,151	26,120	18,951	35,117	27,021	25,697	32,348	6,825	24,086

Table A-5-5 Unit Irrigation Water Supply Requirement by Land Use

Average Monthly Water Supply Requirement per Hectare by Land Use

(Unit: cu.m/ha)

Month	Dates	Orchard				Upland Crop								
	Dates	Lime	Banana	Mango	Average	Onion	Garlic	Tomato	Potato	Okra	Others	Alfalfa	Sorghum	Average
	1,820ha	309ha	128ha	83ha	520ha	96ha	25ha	16ha	15ha	8ha	23ha	211ha	101ha	495ha
Jan.	1,142	815	1,548	1,181	1,054	720	1,332	1,428	2,358		1,440	1,421	1,694	1,343
Feb.	779	420	1,207	806	675		983	1,383	937	105	995	1,081	880	811
Mar.	1,829	1,291	2,481	1,893	1,680		2,138	2,290		1,875	788	2,278	776	1,370
Apr.	2,328	1,677	3,149	2,408	2,156		2,356	2,366		2,941		2,902		1,480
May	3,359	2,465	4,477	3,475	3,121		1,206	1,585		3,947		4,143		1,942
Jun.	3,228	2,385	4,293	3,339	3,007					2,835		3,963		1,735
Jul.	3,116	2,302	4,144	3,223	2,902							3,826		1,631
Aug.	2,940	2,173	3,911	3,042	2,740							3,610		1,539
Sep.	2,675	1,977	3,559	2,767	2,493	395	677	388				3,285		1,524
Oct.	2,139	1,559	2,856	2,213	1,983	1,274	1,525	313				2,628	531	1,563
Nov.	1,439	1,068	1,939	1,489	1,350	1,489	1,457	942	949		837	1,785	1,246	1,476
Dec.	1,146	820	1,553	1,186	1,058	1,276	1,249	899	1,821		1,175	1,425	1,699	1,403
Total	26,120	18,951	35,117	27,021	24,219	5,154	12,923	11,594	6,066	11,704	5,236	32,340	6,825	17,825

Figure A-5-1 Present Cropping Pattern

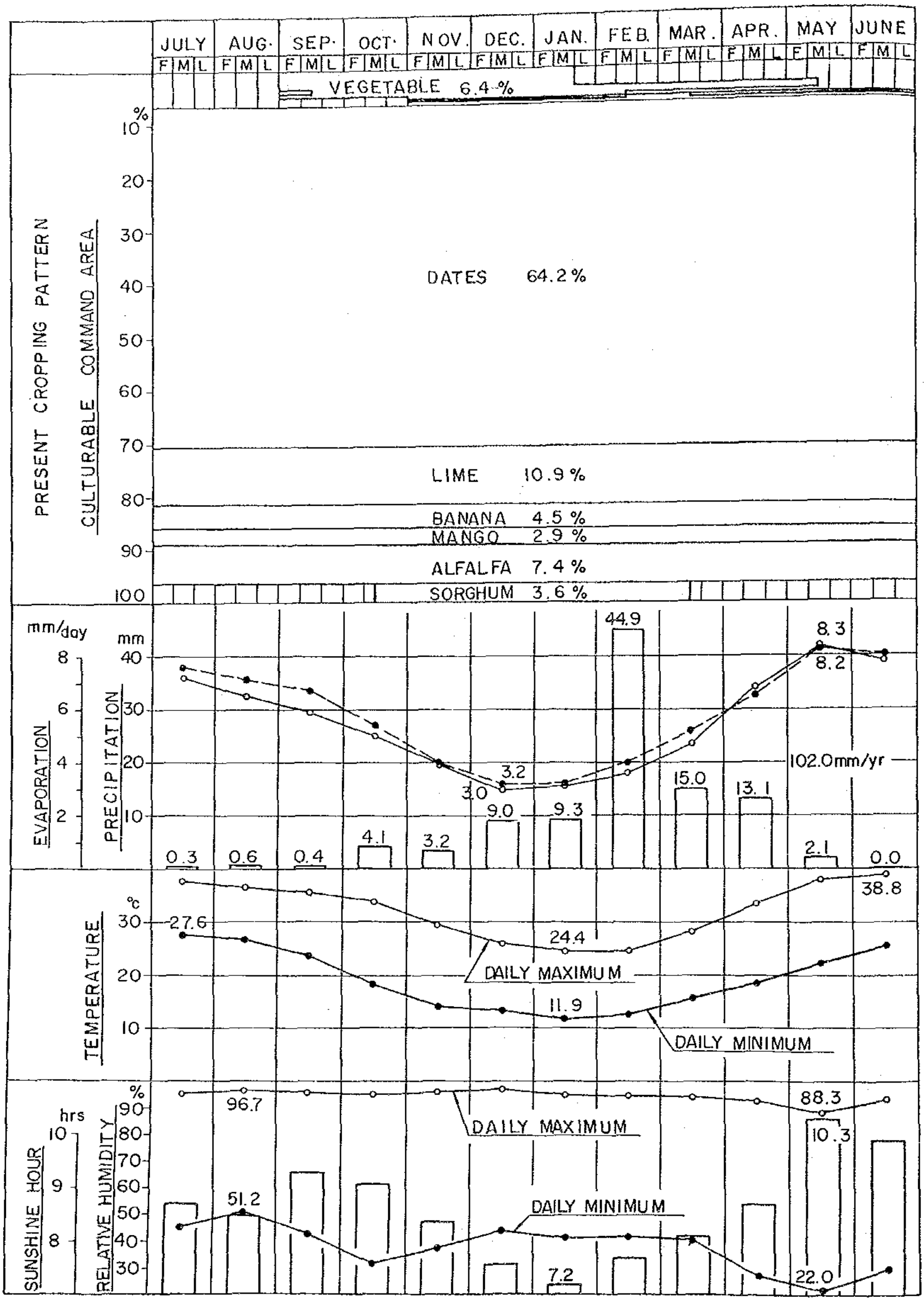
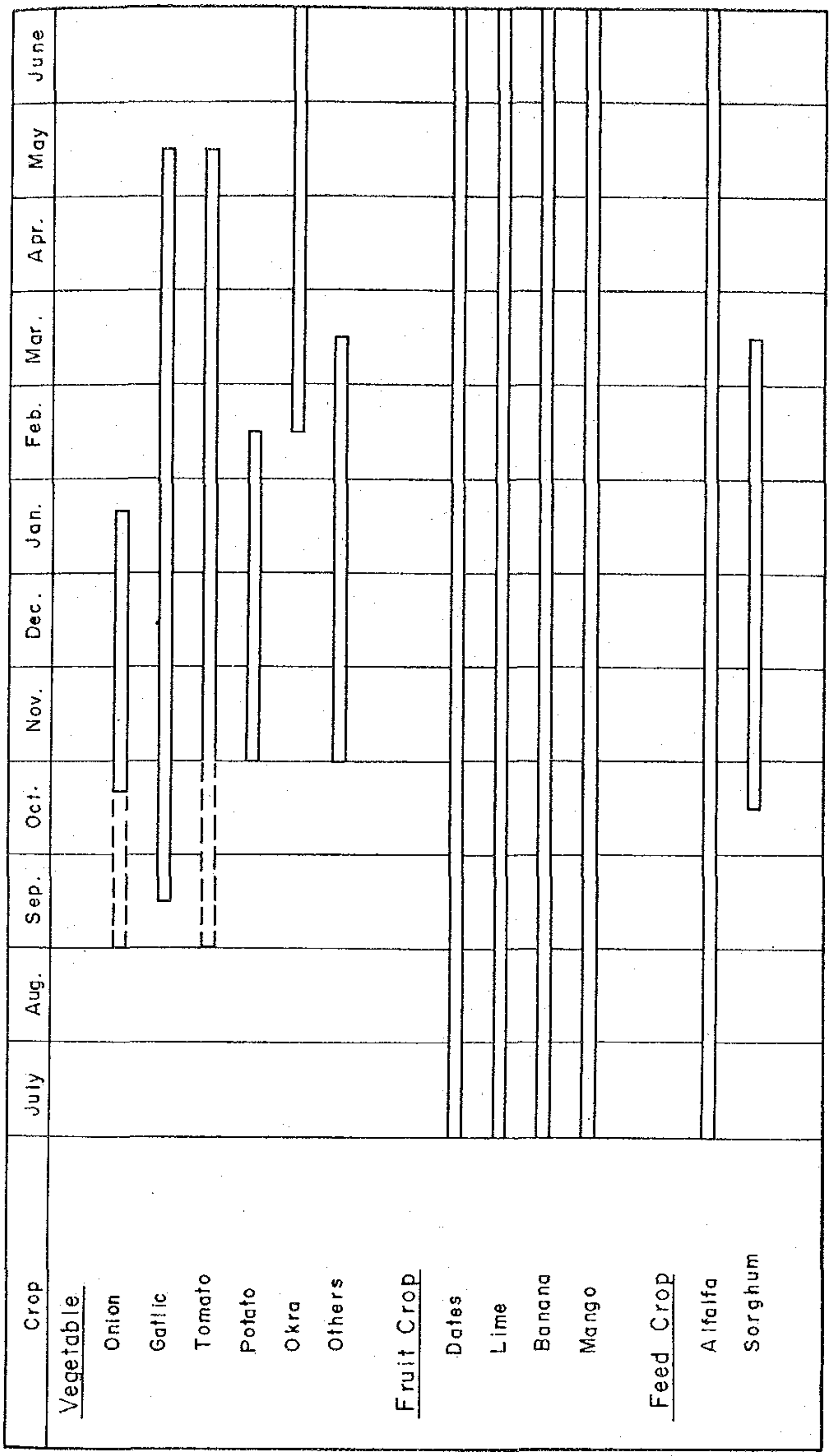


Figure A-5-2 Present Cropping Calender



Present Cropping Calender

ANNEX B GROUNDWATER

ANNEX B

CONTENTS

ANNEX B

LIST OF TABLES AND FIGURES

Annex B Groundwater

Groundwater

B-1. Hydrogeological Unit

The Project Area consists of the following three hydrogeological units; the impervious formations, terrace deposits, and alluvial deposits.

The impervious formations consist of the Hawasina group, basic volcanic rocks, and tertiary sedimentary formations forming the main central ranges and their flanks in the upstream of the Wadi Jizzi.

The Hawasina group consists of silicified limestone, mudstone, and chart with the well stratified beds of some ten centimeters in thickness. The basic volcanic rocks are mainly composed of Oman Ophiolite forming the main central ranges in the mountainous area in the mid-basin of the Wadi Jizzi.

The tertiary sedimentary formations consist of mudstone and limestone, etc., forming the basemant of terrace deposits and low hills in the west edges of the gravel plain. The formations were found by the exploratory well drillings at the gravel plain in the depth of 40 to 50 meter below ground surface in weakly consolidated condition.

The Terrace deposits have a large exposure in the middle stream of the Wadi Jizzi and the west edge of the gravel plain but their distribution is limited to an upper stream of the Wadi. The deposits are divided into four kinds of sediments based on height of their platform, three of which are distributed in the Wadi

Jizzi basin and the lower one is distributed in the mouth of the catchment of the Wadi Bani Umar, forming the alluvial fan.

The deposits are composed of partially cemented sand and gravel of fluvial origin with various sizes of grains of the basic volcanic and sedimentary rocks.

Although the deposits seem aquifure, occasionally their uncemented thin layers of sand and granule among the deposits function as aquifers. Therefore, they act as part of aquifer in terms of hydrogeology.

The alluvial deposits are exposed in a limited area along the wadi course in the catchment; however, they have a large exposure in the coastal plain. The deposits consist of sand and gravel with partially cemented beds of alluvial origin. Thickness of the deposits range from several meters at the river beds in the catchment to 10 m in the mouth of gravel plain and more than 80 m in coastal plain where the deposits grow excellent as unconfined aquifers.

B-2 Aquifer Characteristics

The main aquifers in the Project Area are restricted to the terrace deposits and alluvial deposits. The aquifer characteristics in the coastal plain, especially of alluvial aquifers, have been obtained by the aquifer tests since 1973. Specific capacity and transmissivity of the alluvial aquifers in the east edge of the gravel plain range from 30 to 60 cu.m/hr/m and from 4,000 to 50,000 sq.m/day, respectively. Storativity which was obtained by aquifer tests at production well No. 1 of Sohar Expansion Farm was calculated at 0.05 in an average.

Permeability coefficient ranges from 2.5×10^{-3} to 8.0×10^{-1} m/min with an average of 2.0×10^{-2} m/min.

B-3 Hydrogeological Structure

The groundwater basin comprising the terrace deposits and alluvial deposits is enclosed by the impervious formations at the north and west edges with depth less than 40 m and its thickness to the east reaches more than 100 m at the coast. The basin ends near Qabail and Majis where the impervious formations crop out near the sea. Location of the south end of the basin developing in the downstream of the Wadi Jizzi is found at the north of Wadi Kadaq where trenches of groundwater table extend. The entire groundwater basin extends along the coastal plain over an area of about 10 km long and 10 to 20 km wide. The groundwater being recharged from the Wadi Jizzi has been considered to flow down along two ridges of groundwater table that extend from Fasiqah and Qabail toward inland.

B-4 Movement of Groundwater

Groundwater tables observed in the Wadi Jizzi area are shown in Table B-1. The transition of groundwater tables obtainable from 1974 to 1984 data is given in Figure B-2. In eleven years during when groundwater level observations are available, the years 1977 and 1978 are considered to be high-water years. On the contrary the years in and after 1983 are recognized to be low-water years. Groundwater movement in both the high-water period and low-water period are briefly summarized as follows:

4-1 Groundwater Movement in Low-water Period

Groundwater contour lines drawn from the data observed in

April, 1985 are shown in Figure B-3. As is seen in the said figure, the groundwater table during the low-water period is characterized by three ridges extending from Qabail, Fasiqah and Sohar to the Wadi Jizzi and Wadi Khadaq and by two trenches. Among those ridges, two ridges which extend from Qabail and Fasiqah toward inland ranges along the middle reaches of the Wadi Jizzi, and that extending from Sohar toward inland ranges the Wadi Khadaq. Therefore these ridges are considered to form channels of groundwaters being recharged from both the Wadis.

Groundwaters flowing along such channels are usually provided in the direction of trenches. Due to a small amount of groundwater provided toward trenches, the shape of such trenches become clear especially during the low-water period. The groundwater level is normally kept higher than 1.0 m above mean sea level around Fasiqah, while it is commonly lower than 0.5 m above msl around Amq and Sohar where groundwater trenches exist. The average hydraulic gradient of groundwater in the coastal plain is calculated at 1 : 930.

4-2 Groundwater Movement in High-water Period

Figure B-4 presents groundwater contour lines observed in 1978. The contour lines run almost parallel with the coast line and the groundwater tables are maintained higher than 1.0 m above msl at all places along the coast line. The average hydraulic gradient of groundwater is measured at 1 : 480.

4-3 Mean Groundwater Table

The standardized groundwater tables obtainable by means of arithmetical mean are shown in Figure B-5. The contour lines generally run parallel with the coast line with a little uneven portion. The contour of 1.0 m above msl is distributed at a distance of about 1.0 km from the coast line. Average hydraulic gradient of groundwater is also calculated at 1 : 590.

To express annual changes of groundwater tables, standard deviations are calculated as shown in Figure B-6. As a general tendency, deviations of less than 0.5 m are observed in the area along the coast whereas values exceeding 2.0 m are generally found inland. However even along the coast, a larger value of 1.5 m is collected from Amq where a trench of groundwater table exists.

B-5 Groundwater Flow in View of Iso-Ec Contour Line

Distribution of EC at the surface layer of groundwater along the Wadi Jizzi is presented in Figure B-7. The Iso-Ec contour lines also run parallel with the coast line indicating conductivities ranging from 5,000 to 3,000 micro mho/cm. 500 mmho/cm contour line is distributed at a distance about 3.5 km from the coast line, and a stable value of conductivity of 470 mmho/cm is observed elsewhere inside of 500 mmho/cm line. The groundwater indicating conductivity higher than 500 mmho/cm is considered to be affected by sea water intrusion.

According to the EC loggings collected at OA-2, the vertical section of groundwater along the coast line can be divided into three layers. The upper and middle layers are formed by groundwater runoff, while the lower layer is characterized by sea water intrusion. Variations of depth and conductivity in cach layer are shown in Table B-2 as well as in Figure B-8. As is seen in the said table and figure, no significant difference has been observed in the last eleven years. It is therefore concluded that continuous intrusion of sea water has not occurred yet in the Project area.

B-6 Chemical Quality of Groundwater

The result of chemical analysis conducted in 1982 and 1985 are presented in Table B-3 and are plotted on a key diagram of hydrochemistry as shown in Figure B-9 and B-10. The majority of groundwaters are categorized in type I field indicating fresh water. The groundwater sampled at the coast is classified in type III field, which characterizes hydrochemical contamination caused by sea water intrusion. Waters collected from Wadis and groundwater channels are categorized in type IV field.

B-7 Change of Groundwater Storage

A change of groundwater storage can be defined as the change of groundwater table multiplied by storativity. Storativity of 0.05 was adopted for the study. An average annual change of groundwater table in the recent eleven years from 1974 to 1984 is calculated at 43 mm in defect, corresponding to a volume decrease of 1.02 MCM per annum. Annual changes of groundwater storage so computed are shown in Table B-4 as well as in Figure B-11. The maximum increase of groundwater storage is found at 6.36 MCM per annum between two years 1975 and 1976, while the maximum decrease at 11.54 MCM/yr between 1981 and 1982.

B-8 Groundwater Runoff to the Sea

Groundwater runoff to the sea is obtainable by use of Darcy's formula. Darcy's formula is given in the following equation:

$Q = K.\ A.\ I$

where Q : Flow volume

K : Permeability coefficient = 2.0×10^{-2} m/min.

A : Flow area

$A = L \cdot H$

L : Length of coast line = 10 km

H : Depth of upper layer by EC logging = 17.5 m

Thus, $A = 1.75 \times 10^5$ sq.m

I : Hydraulic Gradient (Refer to in Table B-5)

The hydraulic gradient of groundwater table in the coast area is calculated at 1.70×10^{-3} (1 : 590) as an average in eleven years from 1974 up to 1984. Groundwater to the sea is thus computed at 3.13 MCM per annum

Figure B-12 presents annual change of groundwater runoffs to the sea. From this, the maximum runoff is seen at 4.26 MCM in 1977 and the minimum runoff at 1.15 MCM in 1983.

Table B-1 Groundwater Level

Groundwater Level

	Mean Groundwater Level (Depth) (m)												Average (m)	Standard Deviation(m)
	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985		
OA−1	—	21.32	11.42	17.42	22.30	—	—	—	—	—	—	—	—	—
AE−49	—	10.27	6.40	2.11	4.48	6.62	6.53	11.88	—	2.81	4.72	7.26	—	—
OA−2	6.49	6.15	6.05	6.07	5.79	6.12	6.03	6.25	6.26	6.45	6.47	6.60	6.23	0.35
EA−1	24.38	(24.35)	(22.33)	21.38	21.65	23.31	22.85	22.05	25.60	(24.67)	24.47	25.60	23.30	1.58
WSI−26	11.10	(10.95)	10.37	(10.11)	10.16	10.12	9.45	11.15	11.54	(11.82)	12.36	11.78	10.99	0.98
AE−61	(9.82)	9.88	9.57	9.49	8.57	(9.55)	(9.47)	(9.85)	(10.17)	10.18	10.70	9.99	9.77	0.59
AE−62	(9.07)	8.69	8.85	8.51	8.23	8.59	8.56	9.02	9.82	9.63	9.32	9.54	8.99	0.82
AE−91	(8.78)	8.29	7.70	7.28	7.29	(8.11)	(8.00)	(8.82)	9.06	9.07	9.91	10.41	8.56	1.37
AE−93	(1.78)	1.71	1.28	1.19	1.33	(1.52)	(1.46)	(1.79)	1.69	2.14	2.28	2.14	1.69	0.52
AE−99	(1.73)	1.82	1.52	1.54	1.47	(1.62)	1.58	(1.74)	(1.87)	1.89	1.86	2.02	1.72	0.24
AE−101	(9.69)	9.80	9.09	8.84	9.03	(9.34)	(9.03)	(9.71)	10.49	8.43	10.65	10.55	9.55	0.72
AE−104	5.85	6.16	6.35	5.74	5.82	5.87	6.29	6.56	(6.32)	(6.37)	6.00	6.37	6.14	0.29
AE−142	2.03	2.11	1.74	1.78	2.06	1.88	1.70	2.44	2.26	4.06	2.05	2.52	2.22	0.78
AE−162	(4.32)	4.37	4.11	3.96	4.01	(4.19)	(4.16)	(4.33)	(4.46)	4.77	4.21	4.58	4.29	0.25

Table B-2 Change of EC during 1974-1985

Change of EC during 1974～1985 at OA-2

	Feb.74		Mar.82		Apr.85	
	Depth (mbgs)	EC (μm/cm)	Depth (mbgs)	EC (μm/cm)	Depth (mbgs)	EC (μm/cm)
1st Layer	～27	880	～22	900	～24	1,150
Transitional Layer	27～32	880～ 3,300	22～34	900～ 5,500	24～36	1,150～ 6,400
2nd Layer	32～40	3,500	34～40	5,500	36～42	6,400
Transitional Layer	30～61	3,500～17,000	40～61	5,500～32,000	42～57	6,400～50,000
3rd Layer	61～ 147	18,000	61	50,000+	57～	50,000

Table B-3 Chemical Analysis Data

Chemical Analysis Data

(1982)

Sample No.	Well No.	PH	EC Umbos/cm 25℃	Cations , me/L					Anions, me/L				
				Na^+	K^+	Ca^{++}	Mg^{++}	Total	$CO_3^=$	HCO_3^-	CL^-	$SO_4^=$	Total
1	JA-1	7.0	1,006	4.35	0.11	1.10	4.10	9.66	0.6	2.80	4.25	1.66	9.31
2	JA-2	7.4	367	1.74	0.06	—	2.50	3.74	0.6	1.45	1.10	0.16	3.31
3	JA-3	7.6	518	1.55	0.07	0.40	2.80	4.83	—	2.30	1.80	0.66	4.76
4	JA-4	7.7	489	1.30	0.07	0.20	3.45	5.02	0.6	2.30	1.30	0.42	4.62
5	JA-5	7.45	413	1.30	0.07	0.20	2.90	4.48	0.4	1.85	1.25	0.42	3.92

(1985)

Sample No.	Well No.	PH	EC Umbos/cm 25℃	Cations , me/L					Anions, me/L				
				Na^+	K^+	Ca^{++}	Mg^{++}	Total	$CO_3^=$	HCO_3^-	CL^-	$SO_4^=$	Total
1	JA-1	8.1	953	4.78	0.08	1.70	3.10	9.66	—	3.40	4.40	1.66	9.46
2	JA-2	7.9	379	1.35	0.04	0.26	2.14	3.84	—	2.30	1.20	0.16	3.66
3	JA-3	8.0	440	1.65	0.05	0.27	2.43	4.40	—	2.40	1.40	0.42	4.22
4	JA-4	8.0	562	2.22	0.05	0.55	3.05	5.87	—	2.80	2.05	0.66	5.51
5	JA-5	8.15	391	1.44	0.05	0.55	2.15	4.19	—	2.30	1.15	0.66	4.11
6	JA-6	8.05	367	1.27	0.10	1.48	0.97	3.82	—	2.30	0.60	0.66	3.56
7	OA-2	7.95	1,002	4.05	0.08	1.18	4.57	9.88	—	2.50	6.20	1.16	9.86
8	EA-1	7.90	416	1.99	0.08	0.58	1.57	4.22	—	2.40	1.75	—	4.15
9	WSI-26	8.15	623	2.93	0.06	1.07	2.48	6.54	—	3.50	2.35	0.66	6.51
10	AE-49	8.10	1,063	6.11	0.06	2.60	2.90	11.67	—	4.20	3.35	3.91	11.46
11	AE-62	8.05	966	3.15	0.08	1.05	5.65	9.93	—	2.80	5.20	1.91	9.91
12	AE-91	8.35	831	4.52	0.08	0.53	3.67	8.80	—	5.20	2.45	0.99	8.64
13	AE-93	8.05	1,467	10.67	0.09	1.05	3.60	15.43	—	2.40	11.40	0.99	14.74
14	AE-101	8.10	2,078	8.04	0.09	2.60	13.35	24.08	—	3.10	13.90	7.33	24.33
15	AE-2	8.15	574	2.37	0.08	1.05	2.20	5.70	—	2.60	3.05	—	5.65

Table B-4 Water Level Fluctuation and Change of Storage

Water Level Fluctuation and Change of Storage

		1974 ~1975	1975 ~1976	1976 ~1977	1977 ~1978	1978 ~1979	1979 ~1980	1980 ~1981	1981 ~1982	1982 ~1983	1983 ~1984	Average 1974~1984
Water Level Fluctuation (m/yr)	OA－2	+0.34	+0.10	-0.02	+0.28	-0.33	+0.19	-0.22	-0.01	-0.09	-0.02	
	EA－1	+0.03	+2.02	+0.95	-0.27	-1.66	+0.46	+0.80	-3.55	+0.93	+0.20	
	WSI-26	+0.15	+0.58	+0.27	-0.05	-0.46	+1.17	-1.70	-0.39	-0.28	-0.54	
	AE－61	-0.05	+0.31	+0.08	+0.92	-0.98	+0.08	-0.38	-0.32	-0.01	-0.52	
	AE－62	+0.38	-0.16	+0.34	+0.28	-0.36	+0.03	-0.46	-0.80	+0.19	+0.31	
	AE－91	+0.49	+0.59	+0.42	-0.01	-0.82	+0.11	-0.82	-0.24	-0.01	-0.84	
	AE－93	+0.07	+0.43	+0.09	-0.14	-0.19	+0.06	-0.33	+0.10	-0.45	-0.14	
	AE－99	+0.09	+0.30	-0.02	+0.07	-0.15	+0.04	-0.16	+0.13	-0.02	+0.03	
	AE－101	-0.11	+0.71	+0.25	-0.19	-0.31	+0.31	-0.68	-0.73	+2.06	-2.22	
	AE－104	-0.31	-0.19	+0.61	-0.08	-0.05	-0.42	-0.27	+0.24	-0.05	+0.37	
	AE－142	-0.08	+0.37	-0.04	-0.28	+0.18	+0.18	-0.74	+0.18	-1.80	+2.01	
	AE－162	-0.05	+0.26	-0.15	-0.05	-0.18	+0.03	-0.17	-0.13	-0.31	+0.56	
Average (m)		+0.11	+0.43	+0.23	+0.04	-0.44	+0.18	-0.43	-0.49	+0.01	-0.07	-0.04
Change of Storage (MCM/yr)		+1.88 (+2.51)	+10.13	+5.42	+0.94	-10.36	+4.24	-10.13	-11.64	+0.24	-1.65	-1.02

Table B-5 Hydraulic Gradient and Groundwater Run-off

Hydraulic Gradient and Groundwater Run-off

		1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	Average 1974~1984
Hydraulic Gradient (10^{-3})	OA−2	1.74	2.23	2.37	2.34	2.74	2.27	2.40	2.09	2.07	1.80	1.77	2.17
	AE−62	0.49	0.53	0.46	0.60	0.72	0.57	0.58	0.39	0.06	0.14	0.27	0.44
	AE−91	0.97	1.23	1.53	1.74	1.74	1.32	1.37	0.95	0.83	0.83	0.39	1.17
	AE−93	1.08	1.15	1.58	1.67	1.53	1.34	1.40	1.07	1.17	0.72	0.58	1.21
	AE−99	0.77	0.68	0.98	0.96	1.03	0.88	0.92	0.76	0.63	0.61	0.64	0.81
	AE−101	3.09	2.98	3.69	3.94	3.75	3.44	3.75	3.07	2.29	4.35	2.13	3.32
	AE−104	1.34	1.03	0.84	1.45	1.37	1.32	0.90	0.63	0.87	0.82	1.19	1.07
	AE−142	1.20	1.12	1.49	1.45	1.17	1.35	1.53	0.79	0.99	-0.83	1.18	1.04
	AE−162	1.46	1.42	1.64	1.77	1.73	1.57	1.60	1.45	1.34	1.07	1.56	1.51
	Average	1.68	1.69	2.08	2.19	2.10	1.96	2.05	1.46	1.45	0.59	1.43	1.70
Groundwater Run-off (MCM/yr)		3.09	3.11	3.83	4.03	3.86	3.61	3.77	2.69	2.67	1.09	2.63	3.13

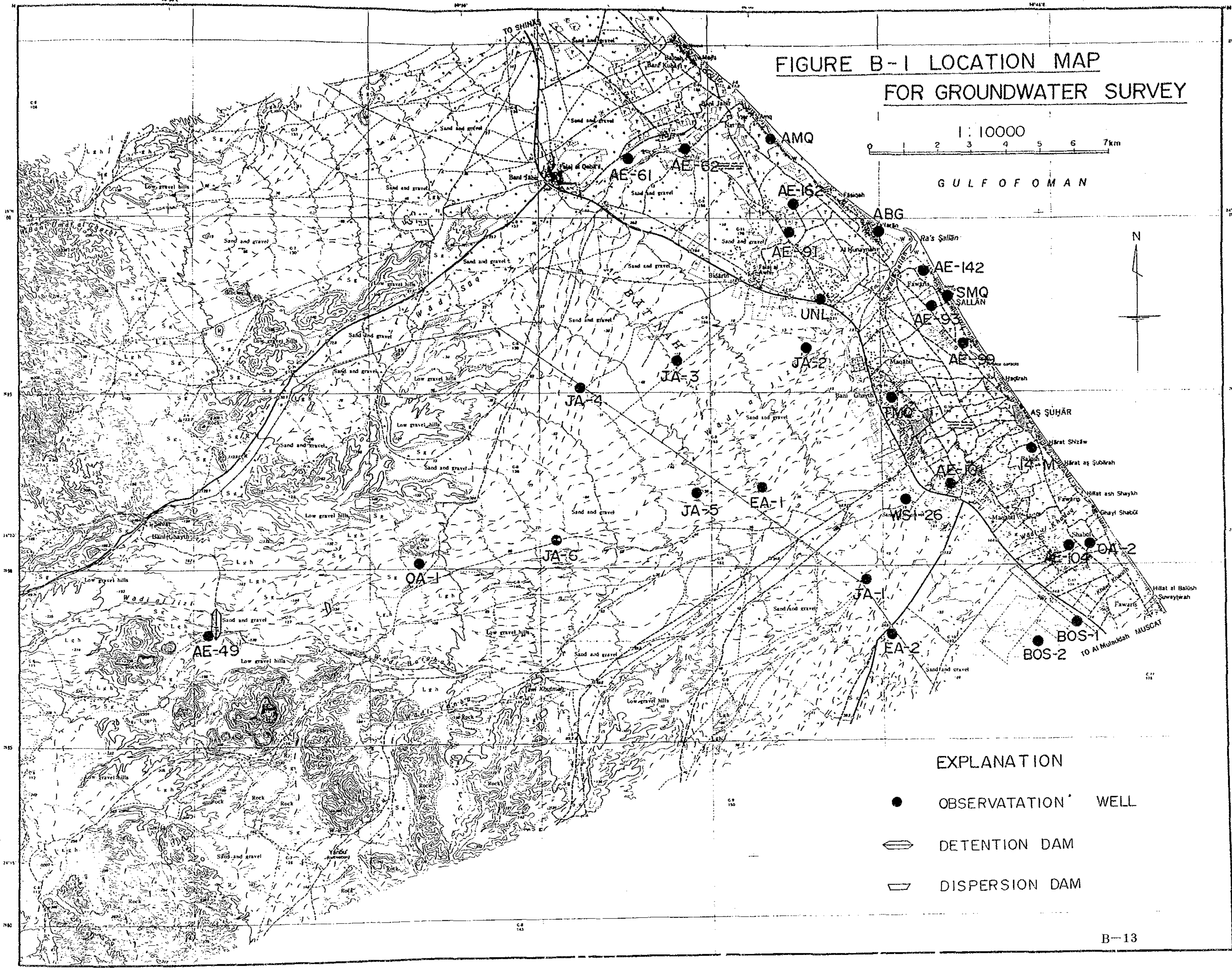
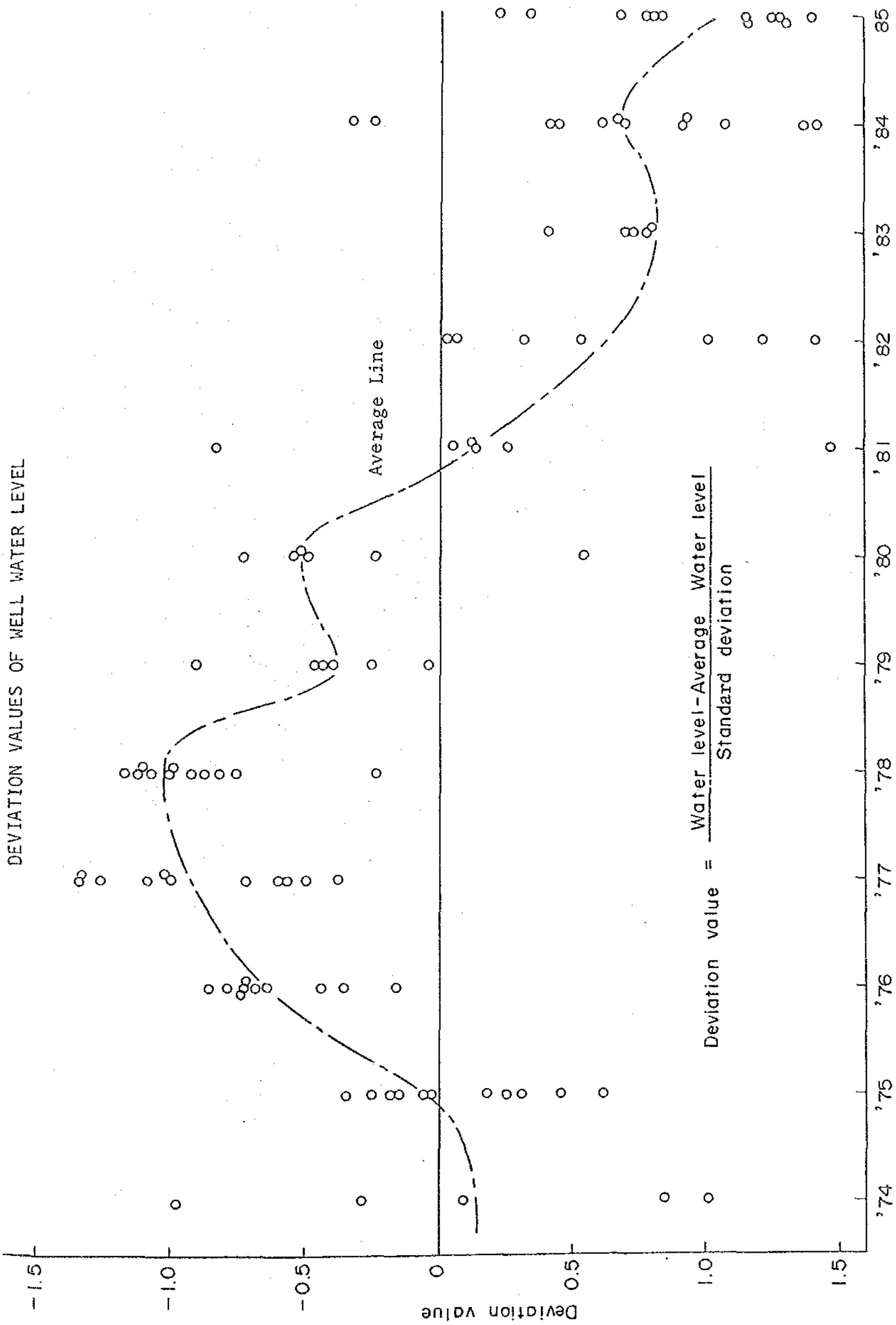


FIGURE B-1 LOCATION MAP FOR GROUNDWATER SURVEY

Figure B-2 Deviation Values of Well Water Level



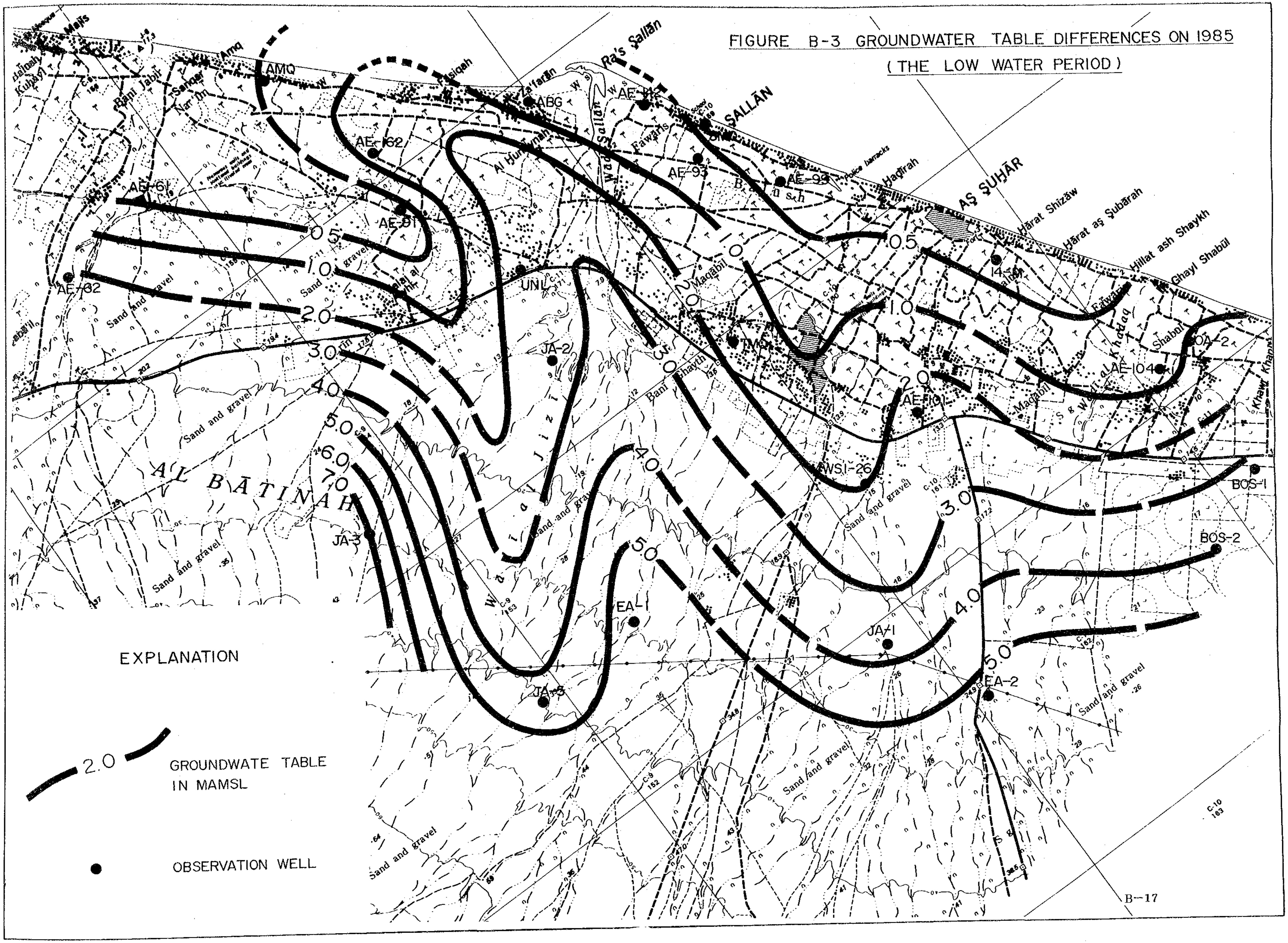


FIGURE B-3 GROUNDWATER TABLE DIFFERENCES ON 1985 (THE LOW WATER PERIOD)

FIGURE B-4 GROUNDWATER TABLE DIFFRENCES ON 1978
(THE HIGH WATER PERIOD)

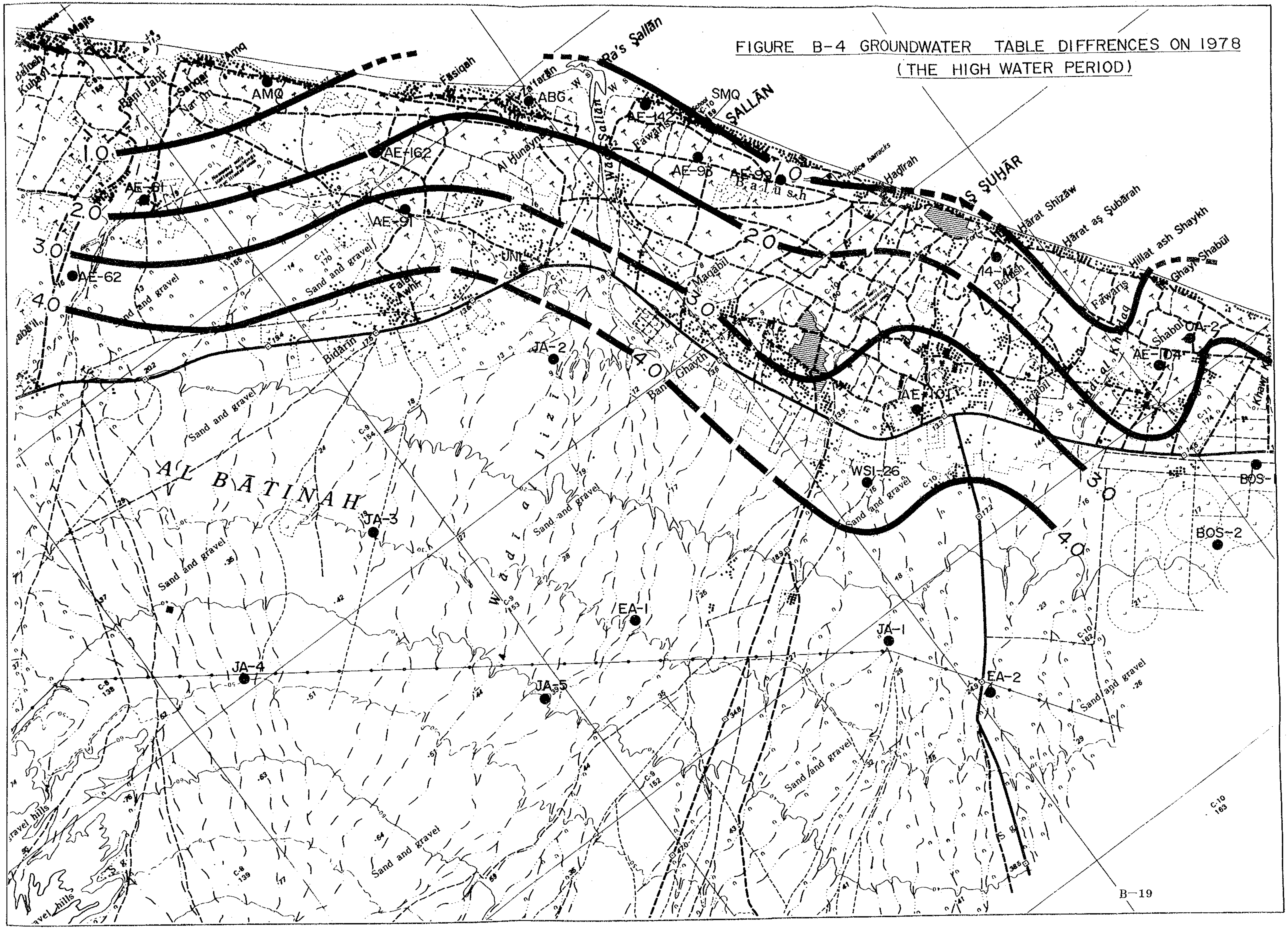


FIGURE B-5 MEAN GROUNDWATER TABLE DIFFERENCES BETWEEN APR. 1974 AND MAY. 1985

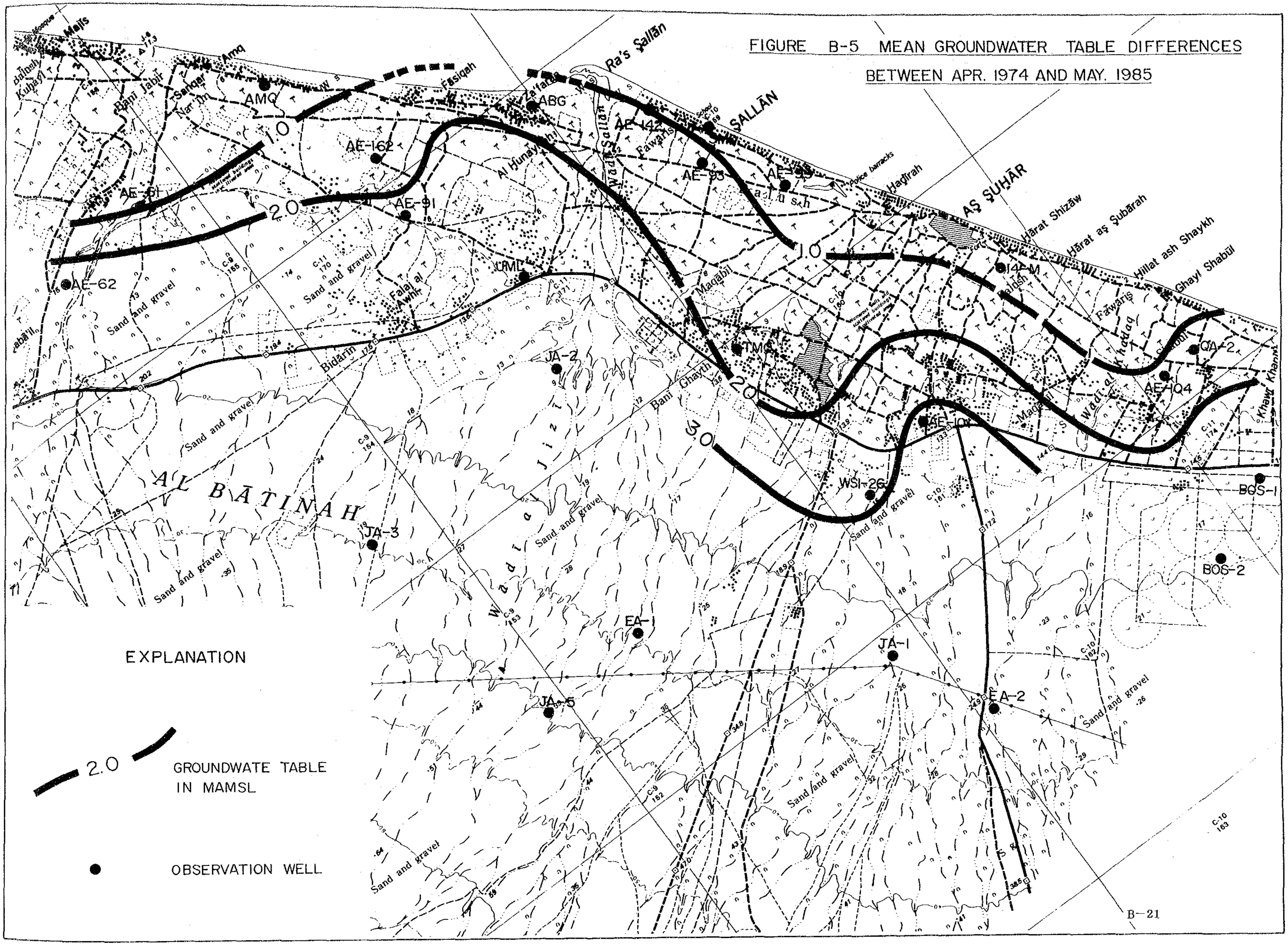
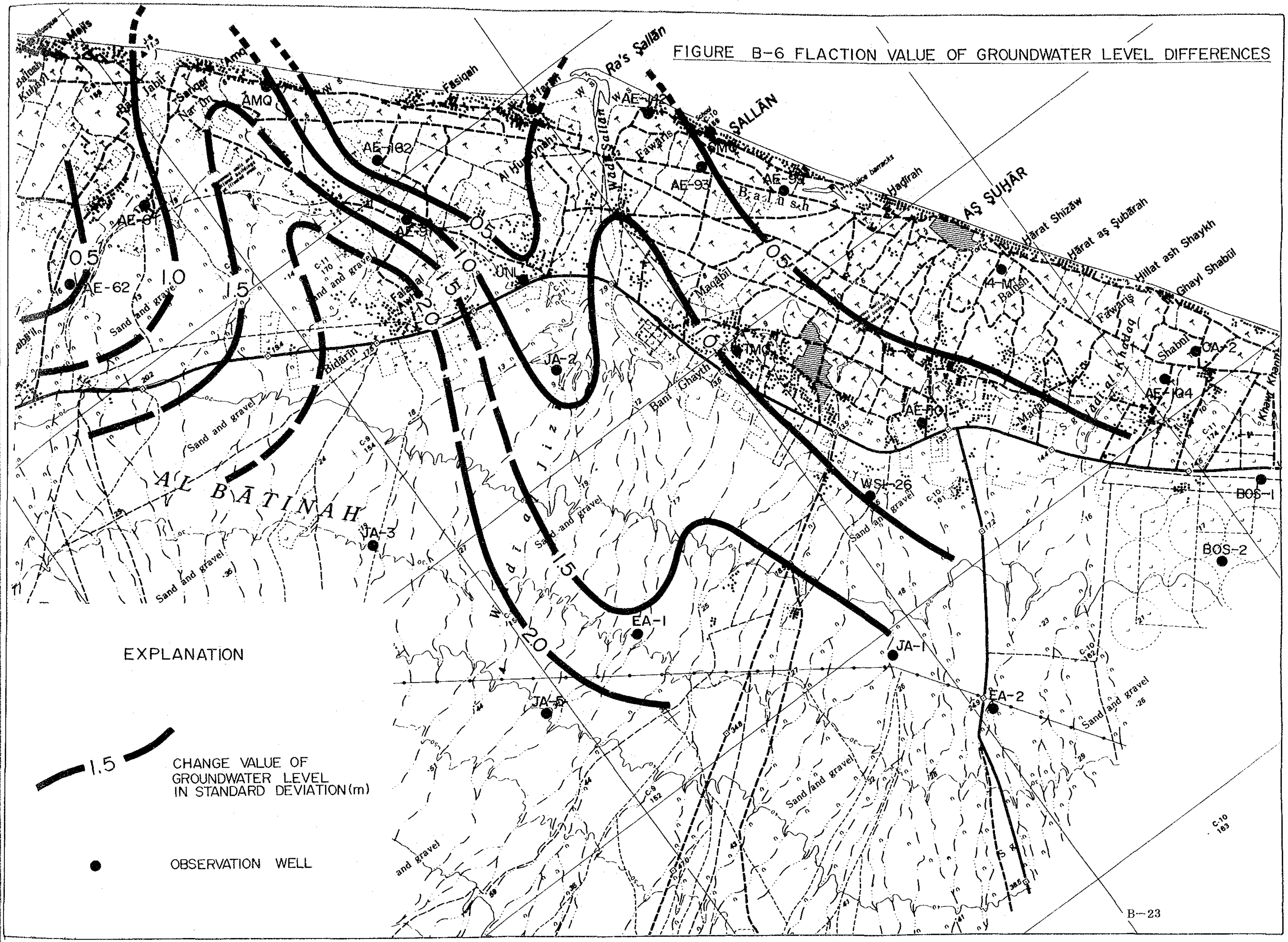


FIGURE B-6 FLACTION VALUE OF GROUNDWATER LEVEL DIFFERENCES



EXPLANATION

1.5 CHANGE VALUE OF GROUNDWATER LEVEL IN STANDARD DEVIATION (m)

● OBSERVATION WELL

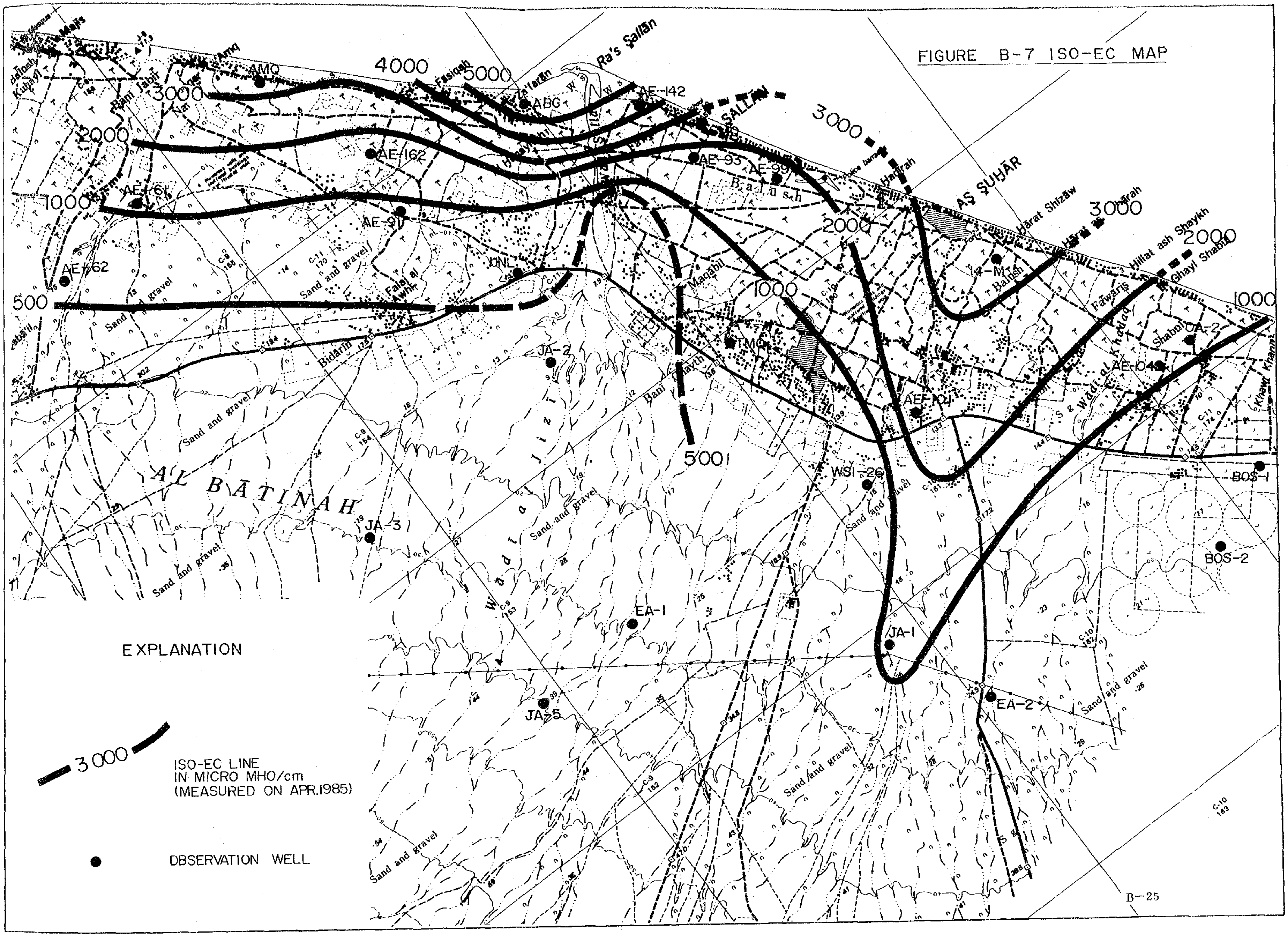
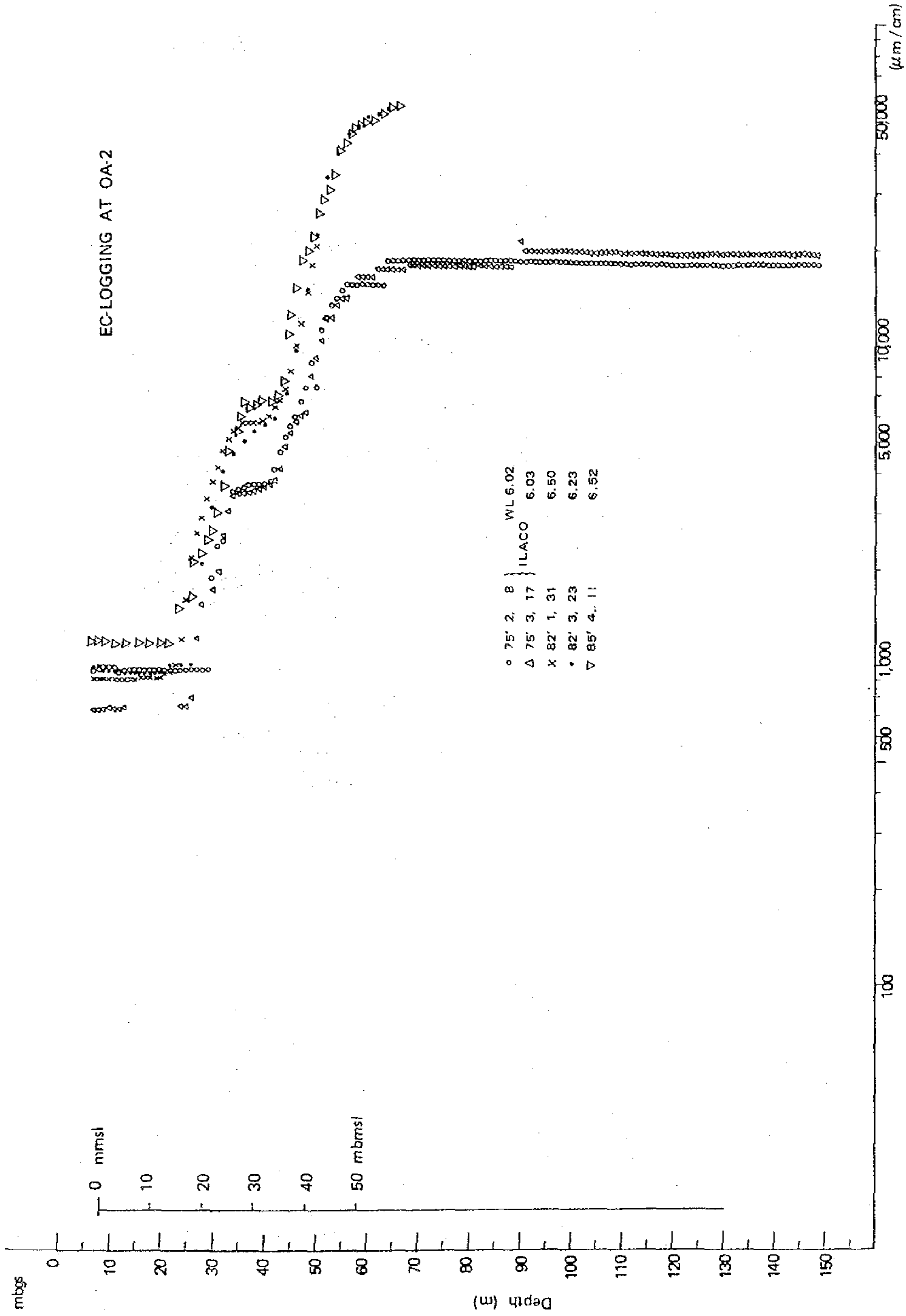


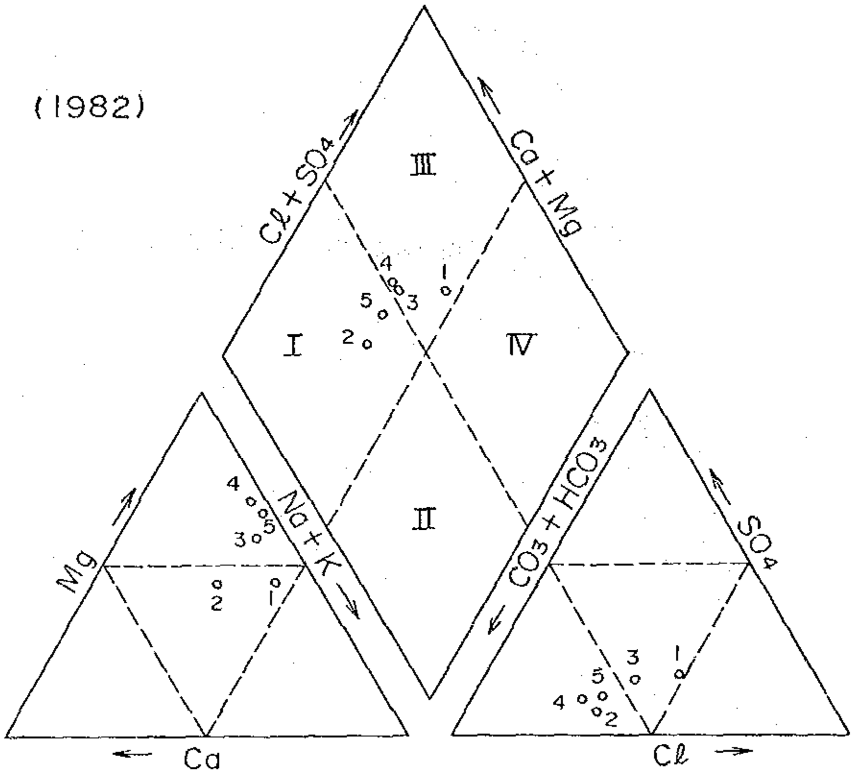
FIGURE B-7 ISO-EC MAP
AL BĀTINAH
AŞ ŞUHĀR
AMQ
ABG
AE-142
AE-162
AE-93
AE-99
AE-161
AE-91
AE-162
UNL
JA-2
TMC
AEF-101
AE-104
OA-2
WSI-26
BOS-1
BOS-2
JA-3
EA-1
JA-1
EA-2
JA-5
14-M
4000
5000
3000
2000
1000
500
Sand and gravel
EXPLANATION
3000
ISO-EC LINE
IN MICRO MHO/cm
(MEASURED ON APR.1985)
OBSERVATION WELL

Figure B-8 EC-Logging at OA-2

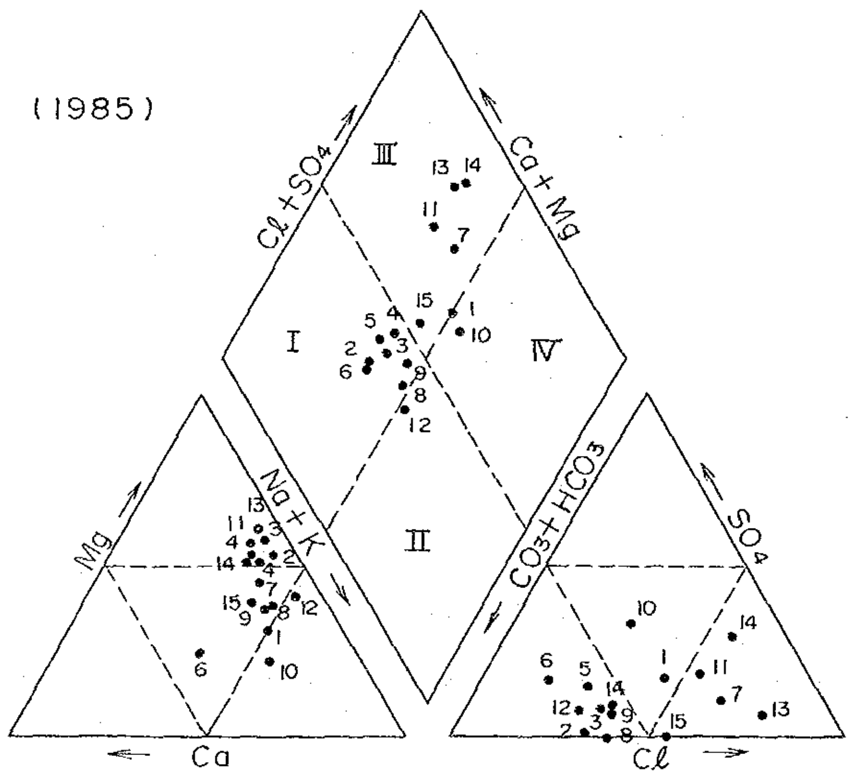


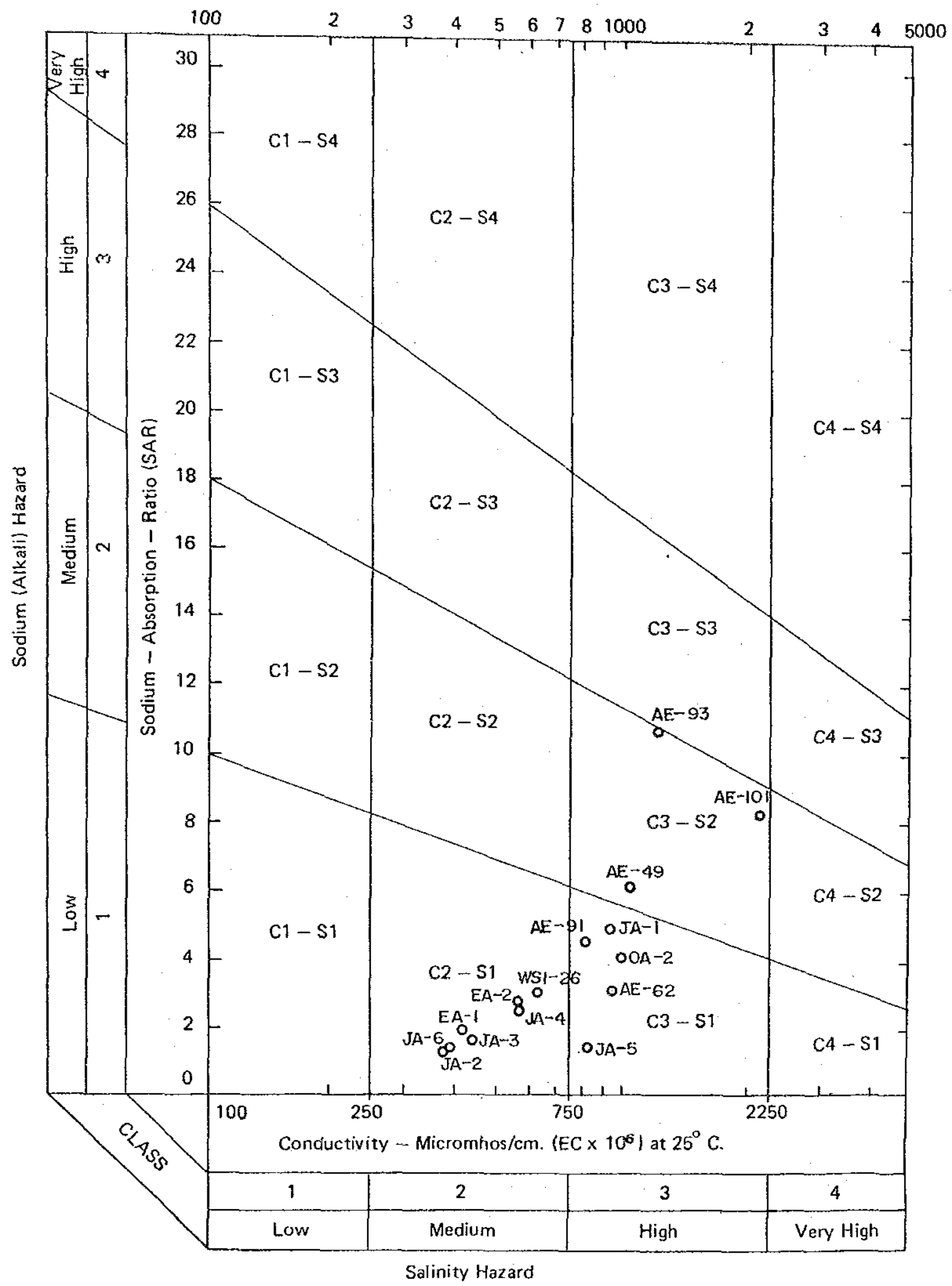
FIGERU B-9 WATER ANALYSIS DIAGRM

(1982)



(1985)



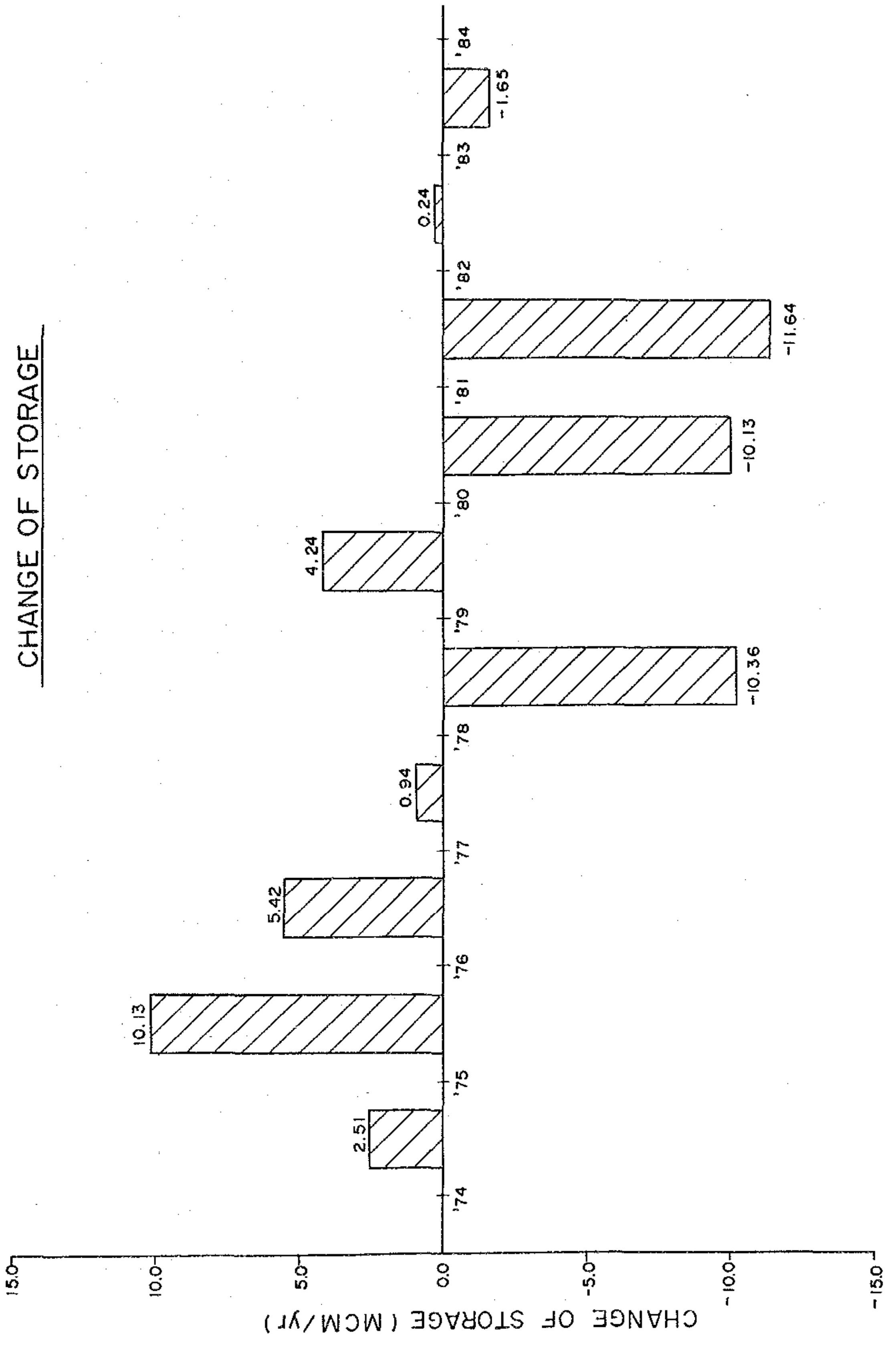


Source: Agricultural Handbook 60, U.S. Dept. of Agriculture

Class	*Salinity or Conductivity*	*Sodium-Absorption Ratio*
Low 1	*Low-Salinity Water* (C1) can be used for irrigation with most crops on most soils with little likelihood that soil salinity will develop. Some leaching is required, but this occurs under normal irrigation practices except in soils of extremely low permeability.	*Low-Sodium Water* (S1) can be used for irrigation on almost all soils with little danger of the development of harmful levels of exchangeable sodium. However, sodium-sensitive crops such as stone-fruit trees and avocados may accumulate injurious concentrations of sodium.
Medium 2	*Medium-Salinity Water* (C2) can be used if a moderate amount of leaching occurs. Plants with moderate salt tolerance can be grown in most cases without special practices for salinity control.	*Medium-Sodium Water* (S2) will present an appreciable sodium hazard in fine-textured soils having high cation-exchange capacity, especially under low-leaching conditions, unless gypsum is present in the soil. This water may be used on coarse-textured or organic soils with good permeability.
High 3	*High-Salinity Water* (C3) cannot be used on soils with restricted drainage. Even with adequate drainage, special management for salinity control may be required and plants with good salt tolerance should be selected.	*High-Sodium* (S3) may produce harmful levels of exchangeable sodium in most soils and will require special soil management--good drainage, high leaching, and organic matter additions. Gypsiferous soils may not develop harmful levels of exchangeable sodium from such waters. Chemical amendments may be required for replacement of exchangeable sodium, except that amendments may not be feasible with waters of very high salinity.
Very High 4	*Very High Salinity Water* (C4) is not suitable for irrigation under ordinary conditions, but may be used occasionally under very special circumstances. The soils must be permeable, drainage must be adequate, irrigation water must be applied in excess to provide considerable leaching, and very salt-tolerant crops should be selected.	*Very High Sodium Water* (S4) is generally unsatisfactory for irrigation purposes except at low and perhaps medium salinity, where the solution of calcium from the soil or use of gypsum or other amendments may make the use of these waters feasible.

FIGURE B-10 DIAGRAM FOR CLASSIFICATION OF IRRIGATION WATER

Figure B-11 Change of Storage



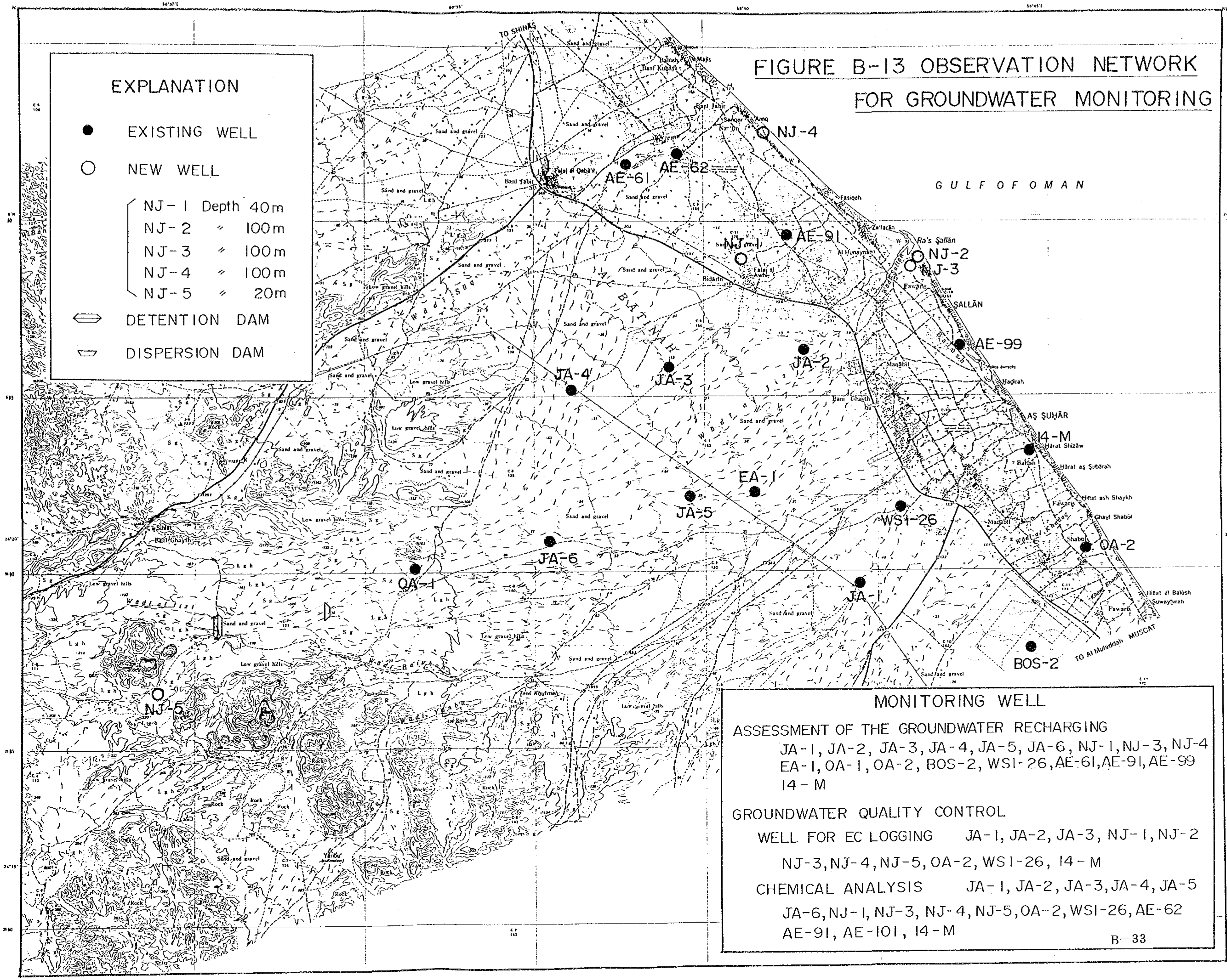


FIGURE B-13 OBSERVATION NETWORK FOR GROUNDWATER MONITORING

B-9 Computer Simulation of Groundwater Basin

9-1 Simulation Model*/

A general purpose digital computer is used in developing and testing a two-dimensional diffusion model of a groundwater basin, aiming at (1) the development of the model and (2) the use of the model for analyzing and predicting dynamic behavior of the basin under imposed conditions.

Currently as in the past, the extractions from the coastal plain have exceeded its natural replenishment. As a result, water levels continue to recede. In addition, the groundwater quality near the sea-coast is impaired by the intrusion of sea water. These difficulties can be surmounted by constructing a detention dam for additional recharge of greater quantities of fresh water, and by controlling extractions. These and other remedial measures are evaluated and applied through groundwater basin management. Optimum results will be achieved when the dynamics of the basin are better understood.

In order to study the dynamic behavior of the groundwater basin, the details of its geometry, physical characteristics, and its associated replenishment and extraction data must be obtained. Unfortunately, the cost of determining such physical characteristics (transmission and storage properties) in the requied detail by field measurements is prohibitively large. On the other hand, the geometric details can, in many instances, be readily determined from a variety of records that accrue from geophysical studies, water quality analyses, well logs, etc. If such records are available for a given basin, and if replenishment and extraction data can be obtained in detail, then the computer can be used to find a representative set of transmission and

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*/ Ground-water Management for the Nation's Future - Computer Simulation of Ground-water Basins; by H.N. Tyson and E.M. Weber Journal of the Hydraulics Division, Proceedings of ASCE, July, 1964

storage properties or coefficients. This is accomplished by a cut-and-try adjustment of the coefficients until the dynamic responses of the model satisfactorily matches the historical water level elevations measured in the field.

9-2 Physical Situation of Ground-water Basin

The groundwater basin comprising the terrace deposits and alluvial deposits coincides with depth of the impervious formations beneath the gravel plain.

The groundwater basin is enclosed by the impervious formations at the north and west edges with depth of less than 80 m and it thickens to the east up to more than 100 m at the coast. The basin ends near Qabail and Majis where the impervious formations crop out near the sea. Location of the south end of the basin is estimated at the south of Wadi Ahin where the impervious formations are croped out near the sea. An entire area of the groundwater basin mainly developing in the downstream of Wadi Jizzi extends about 20 km in length along the coast with 8 km width. Furthermore, depth of it is estimated 50 to 60 m at the west edge of the basin and it deepens to the sea up to more than 100 m. Depth of the basin, especially at the west edge is verified by the exploratory drilling at JA-5 and 6, the production well for Mining Co., TS-6, 7, 8 and 9, and geo-electric survey at lines ES-1 and ES-V4.

The groundwater basin in the west edge of the gravel plain is composed aquifers of the terrace deposits with depth ranging 40 m in maximum to less than 20 m at an outlet of catchment and it extends to the wadi beds in the catchment decreasing thickness of aquifer.

For the initial computer analysis, the groundwater complex is replaced by a simplified model. This model consists of a single equivalent aquifer whose local properties are composites of the corresponding properties of the several aquifers that make up the actual structure. The thickness of the single aquifer is allowed to vary with position, and yet it is considered to be small compared to its lateral dimensions. The bounding edges of the model are irregular in shape. The model is divided into small polygonal zones which enclose observation wells. The dynamic response of the portion of the model included within each zone is represented by a single water level elevation. The size of the zones is dependent on the variations in replenishment, extraction, transmission, storage, and water level data. For purposes of testing the model against historical water level data, provision is made for the extraction or injection of time-varying flow rates from each of the zones.

9-3 Mathematical Model

The equation of continuity of an unconfined aquifer, in which there is no vertical variation of properties, is given by;

$$\nabla\bar{V} + S\frac{\partial h}{\partial t} + Q = 0 \quad \text{......................} \quad (1)$$

in which $$h = \delta + z \quad \text{......................} \quad (2a)$$

and $$\delta = \delta_0 + \bar{\delta} \quad \text{......................} \quad (2b)$$

δ_0 and $\bar{\delta}$ are, respectively, the mean and perturbation components of δ.

Darcy's Law provides the equation of motion;

$$\bar{V} = -\rho g\frac{k}{\mu}\nabla h \quad \text{..........................} \quad (3)$$

In Equations 1, 2a, 2b and 3, z is the reference elevation, h is the head, δ refers to the local thickness of saturated portion of the aquifer, $\bar{V}$ denotes the velocity, S refers to the storage coefficient, Q describes the volumetric flow rate per unit area, ρ is the density, g refers to the acceleration of gravity, k is the permeability, μ is the absolute viscosity, and t denotes the time.

Equations 1 and 3 are combined and linearized to yield a single equation that, subject to appropriate boundary conditions, describes the dynamics of flow in the aquifer. The thickness of the aquifer is assumed to be small compared to its lateral dimensions. This equation is;

$$\nabla T \nabla h - S\frac{\partial h}{\partial t} - Q = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which

$$T = \frac{\delta_0 \rho g k}{\mu} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

The quantities T and S are, respectively, the local transmissibility and storage coefficient of the aquifer. The source flow rate, Q, is in most cases time dependent. This flow rate is the algebraic sum of several component extraction and replenishment flows. The replenishment flows are precipitation, imported water, stream percolation, artificial recharge and subsurface inflow across boundaries. The extraction flows consist mainly of the water pumped from the aquifer for consumptive use and subsurface outflow across boundaries.

Herein, the Equation 4 is replaced by an equivalent system of difference - differential equations, the simultaneous solution of which gives the wanted function, h, at a finite number of node points lying within the boundaries of the aquifer.

The attendant system of difference - differential equations is;

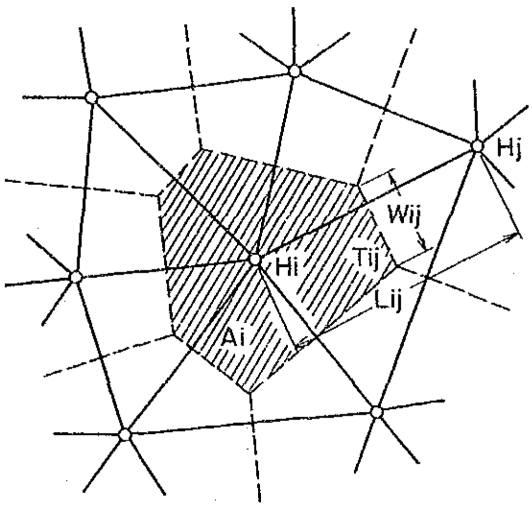
$$\sum_j (h_j - h_i) \cdot Y_{j,i} = A_i \cdot S_i \cdot \frac{dh_i}{dt} + A_i \cdot Q_i$$

and

$$Y_{j,i} = \frac{W_{j,i} \cdot T_{j,i}}{L_{j,i}} \quad \ldots\ldots \quad (6)$$

in which A_i denotes the area associated with node i in sq·km, $Y_{j,i}$ is the conductance of pass between nodes j and i, in MCM per month per meter, S_i is the storage coefficient of polygonal zone associated with node i, Q_i is the volumetric flow rate per unit area at MCM per month per sq·km, $T_{j,i}$ denotes the transmissibility at midpoint between nodes j and i, in MCM per month per meter, $L_{j,i}$ refers to the distance between nodes j and i, and $W_{j,i}$ describes the length of perpendicular bisector associated with nodes j and i, in km.

A typical node point, its associated polygonal zone, and contiguous node points are shown as follows:



The terms in the first equation of the Equation 6 are interpreted physically with the aid of the above figures as follows:

The left side represents the sum of the flows within the aquifer to the polygonal zone, A_i, and the first term on the right side represents the rate of water storage within A_i. The remaining term represents the extraction or replenishment flow from A_i.

9-4 Digital Computer Solution

The Equation 6 is solved on the general purpose digital computer by an implicit numerical integration techinique:

$$\sum_j (H_j^{t+1} - H_i^{t+1}) \cdot Y_{j,i} = \frac{A_i \cdot S_i}{\Delta t}(H_i^{t+1} - H_i^{t}) + A_i \cdot Q_i^{t+1} \quad \ldots (7)$$

in which the superscripts t denotes points along the time coordinate. This method of integration has the advantage that the magnitude of the time step, Δt, does not depend on a stability criterion.

The computation procedure is as follows:

(1) Initial water levels, H_{H0i}, are impressed at the terminals labeled H_i^t (i=1,2,...,N; N=number of nodes)

(2) Then, for a given set of coefficients L, W, T, A, S and Q, the values H_i^{t+1} are implicitly determined at the end of a step in time, Δt.

(3) Once determined, these value become the initial water elevations for the next succeeding step in time.

(4) Consecutive numbers are given to all nodes and line segments connecting such nodes, as illustrated in Figure B-14 for the present condition of the groundwater basin as well as in Figure B-15 for the proposed condition (Three observation wells are added, as proposed, for the proposed condition).

(5) All the source flow rates at each node (A•Q) are calculated first on the basis of findings obtained through hydrological analyses.

(6) Then, all the node-to-node subsurface flows (O) and all the storage flows (ST) are calculated.

(7) Next, all the flows (subsurface, storage and extraction) are balanced at each node by setting their sum equal to a residual term (RES).

(8) Water level elevation at each node is adjusted by the magnitude of the residual, attenuated by a relaxation coefficient. The relaxation coefficient, RELAX, is defined as;

$$RELAX_i = \frac{1}{\sum_j Y_{i,j} + \frac{A_i \cdot S_i}{\Delta t}}$$

(9) After all the values of h have been adjusted in this manner, a sum is formed of the nodal residuals. This sum is compared with a threshold value (ERROR), which is defined as the maximum acceptable sum of nodal flow residuals at any time step.

(10) If the sum of the residuals is less than or equal to the threshold value, the calculation of the values of h is complete for that time step. Otherwise the calculation is repeated as many times as is required to reduce the sum of residuals to a value less than or equal to the threshold value.

The digital computer programs for the groundwater basin model are written, provisionally, using the FORTRAN system. The FORTRAN programs developed and used for the project are attached at the end of the Appendix B-9, and a part of its corresponding flow chart is shown in Figure B-16.

9-5 Modeling of the Groundwater Basin

Based on the conception previously mentioned, the groundwater basin is divided into a number of polygonal zones as shown in Figure B-14 for the present condition of the basin before constraction of the proposed detension dam, as well as in Figure B-15 for the proposed condition after construction of the dam.

9-6 Input Data

Among the basic parameters to furnish the model, Falaj Use and base flow runoff in and from the catchment of the proposed dam, direct surface runoffs from the dam catchment as well as from residual area below the dam, and direct recharge in the area are referred to in the hydrological investigations.

Rainfall-Runoff Relation

Areal Rain (mm)	Runoff at Dam (mm)	Runoff at Rivermouth (mm)
8.0	0.0	-
10.0	0.13	-
11.5	(-)	0.0
20.0	0.83	0.43
30.0	1.98	0.90
40.0	3.45	1.30
50.0	5.23	1.70
60.0	7.29	2.08
70.0	9.74	2.47
80.0	12.55	2.82

These values are obtainable from the areal rainfall in the areas above and below the dam, and hence the model requires spot rainfall records at Daqiq, Kitnah, Hayl (Wadi Jizzi), Hayl (Wadi Hayl), Farfar and Sohar to be inputted.

Groundwater recharges in the polygonal zones from the surface flow are estimated in proportion to the length of pass of percolation along the main course of the Wadi Jizzi.

Zone No. Present	Proposed	Length of pass (km)	Rate of Percolation
1	1	3.80	0.33
2	2	2.05	0.18
4	4	1.20	0.11
5	5	1.10	0.10
10	10	2.35	0.21
13	15	0.85	0.07
	Total	11.35	1.00

Seasonal variation of irrigation water requirement to be extracted from the groundwater storage is obtained from the study made for evaluation of irrigation water consumption.

Seasonal Variation of Irrigation Consumption

(Unit: MCM/month/1000ha)

Month	Consumption
January	0.220
February	0.140
March	0.283
April	0.336
May	0.468
June	0.438
July	0.419
August	0.396
September	0.371
October	0.327
November	0.258
December	0.226

The FAO method of estimating effective rainfalls is also used to examine the balance of rain waters. Table B-6 presents the monthly average crop evapo-transpiration so prepared to calculate effective rainfalls through the criteria provided in FAO Irrigation and Drainage Paper No.24.

The initial water level in each observation well, permeability coefficients, storativity, and topographic parameters such as thickness of the permeable layors are also referred to in the findings of hydro-geological investigations, as shown in Figures B-17 to B-19.

9-7 Groundwater Simulation

Four cases of the groundwater simulation study are conducted as under;

Case No.	Description of Simulation Condition
0	Present condition with water extraction of 11.0 MCM/yr for irrigation and 0.2 MCM/yr for domestic use.
1	Proposed condition with Project, with extraction same as in the present conditon.
2	In addition to the Case 1, additional water extraction of 1.06 MCM/yr for domestic uses (in total 1.26 MCM/yr) is considered.
3	In addition to the Case 2, additional water extraction of 1.39 MCM/yr for irrigation purpose is considered.

All computations are progressed for the entire period of available data of 11 years from 1974 up to 1984 employing a monthly time interval. The data used, simulated result of the groundwater balance and well water levels, and the simulated water levels in terms of the deviation value, with their 12-months moving averages, are presented in Figures B-21 to B-23 and Tables B-7 to B-10, Figures B-24 to B-26 and Tables B-11 to B-14, Figures B-27 to B-29 and Tables B-15 to B-18, Figures B-30 to B-32 and Tables B-19 to B-22, respectively for the Cases 0, 1, 2, and 3.

Figure B-23 is compaired with Figure B-2 for the purpose of model verification, and the followings may bring a conclusion.

i) As compaired with observed movement of groundwater levels, the simulated behavior of groundwater levels almost follows its actual pattern, with exception in and after 1982.

ii) The above differences may be caused by the differences in the amount of water extraction between the actual field performance and the estimated water consumption of a uniform rate throughout a number of years.

iii) The model parameters and input data may be modified and updated after several years of study and experience to simulate more accurate behavior of the groundwater movement.

iv) Consequently, the model simulation will be useful to predict the groundwater behavior under the inposed condition, subsequently making the model of a useful tool for the management of groundwater resources.

Figure B-20 summarizes the simulated changes of groundwater storage for the respective computation cases, resulting in a conclusion that the Project would provide the required water demand associated with the Case 3 without producing any undesired result on the groundwater resources, and proving that an expected value of permeability of 2.0×10^{-2} 1/ m/min. and storativity of 0.05 2/ are, as a whole, reasonable.

1/, 2/: The values of permeability and storativity were computed by using the data of pumping up tests.

Table B-6 Average Crop Evapo-Transpiration

(For calculation of effective Rainfall)

Crop	Dates	Lime	Banana	Mango	Onion	Garlic	Tomato	Potato	Okra	Others	Alfalfa	Sorghum	Average
Area(ha)	1820	309	128	83	96	25	16	15	8	23	211	101	2835
Jan.	62.9	44.9	80.9	62.9	58.4	71.9	76.4	121.4	-	76.4	76.4	89.9	64.0
Feb.	70.6	50.4	90.7	70.6	-	80.6	100.8	126.0	40.3	80.6	85.7	75.6	68.8
Mar.	102.0	72.8	131.1	102.0	-	116.6	123.8	-	102.0	87.4	123.8	87.4	97.4
Apr.	126.0	90.0	162.0	126.0	-	126.0	126.0	-	153.0	-	153.0	-	115.3
May	167.1	119.3	214.8	167.1	-	119.3	155.2	-	191.0	-	202.9	-	152.4
Jun.	159.6	114.0	205.2	159.6	-	-	-	-	136.8	-	193.8	-	143.6
Jul.	154.1	110.1	198.1	154.1	-	-	-	-	-	-	187.1	-	138.3
Aug.	145.4	103.8	186.9	145.4	-	-	-	-	-	-	176.5	-	130.5
Sep.	132.3	94.5	170.1	132.3	189.0	66.1	189.0	-	-	-	160.6	-	126.8
Oct.	108.5	77.5	139.5	108.5	155.0	77.5	155.0	-	-	-	131.7	54.3	106.1
Nov.	73.5	52.5	94.5	73.5	73.5	73.5	47.2	47.2	-	42.0	89.3	63.0	72.2
Dec.	62.9	44.9	80.9	62.9	67.4	67.4	49.4	94.4	-	62.9	76.4	89.9	63.8

Note: Unit = mm/month

Figure B-14 Modeling of Groundwater Basin

(Present Condition)

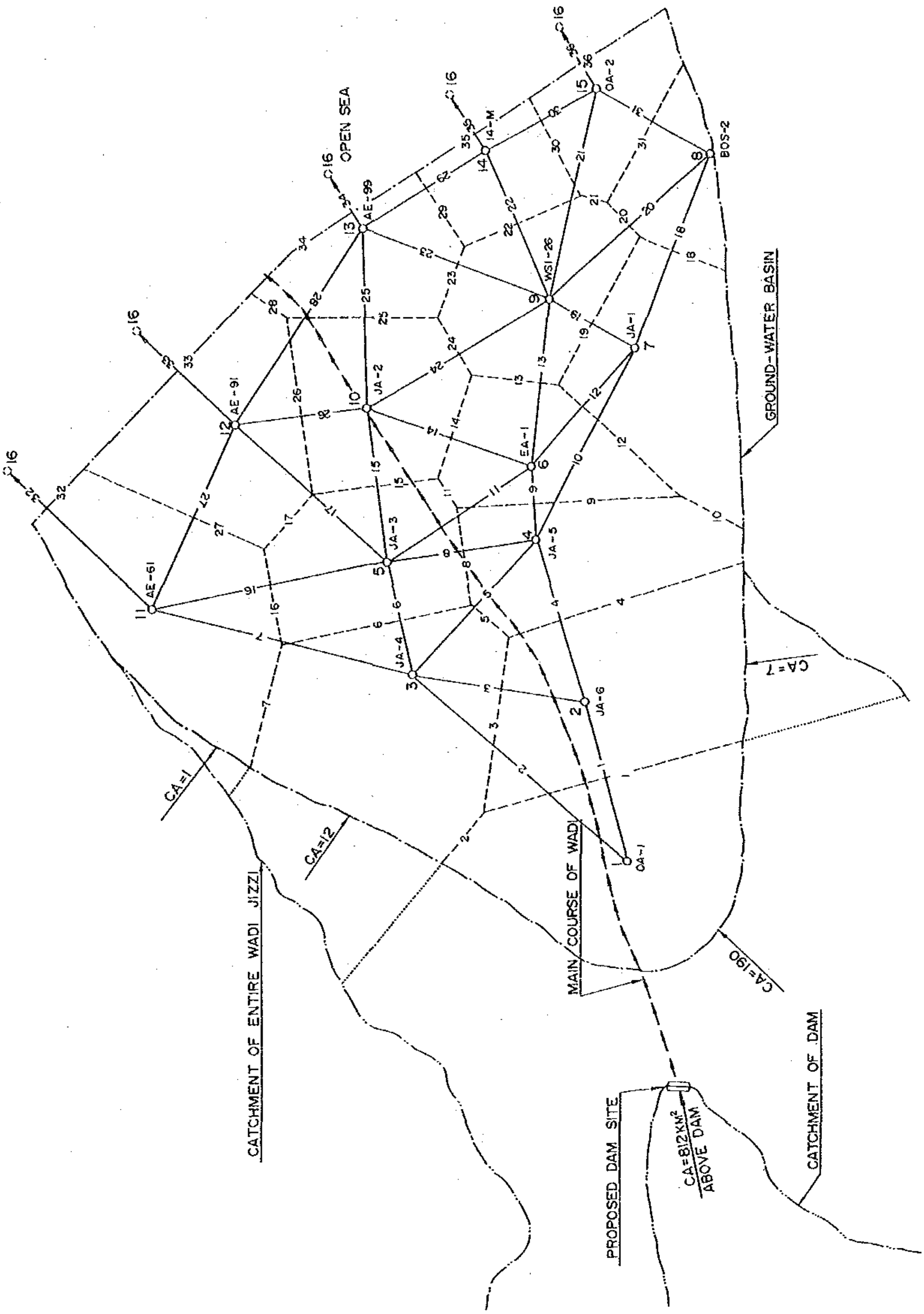


Figure B-15 Modeling of Groundwater Basin

(Proposed Condition)

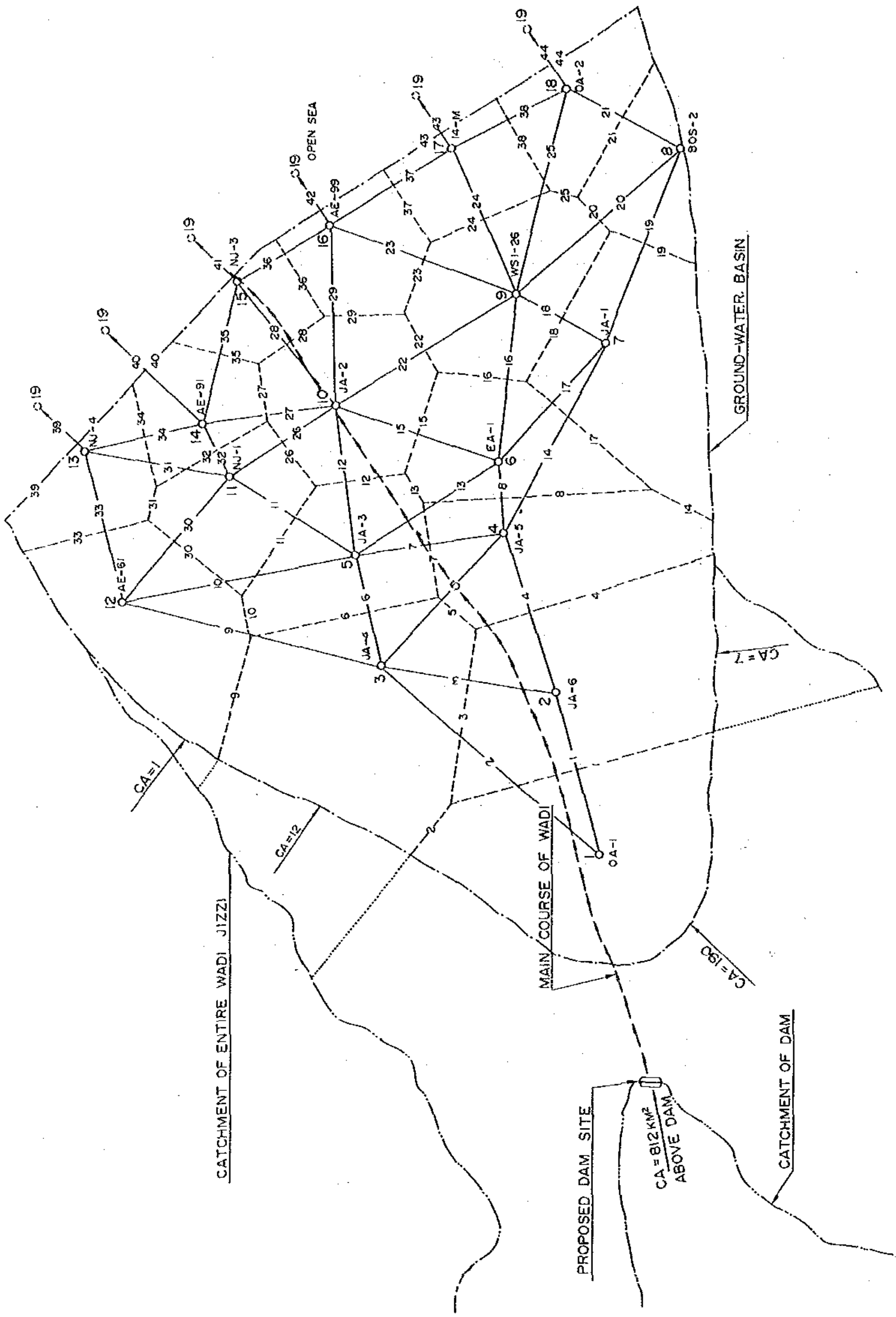


Figure B-16 Simplified Flow Chart for Digital Computer Solution of Ground-Water Problem

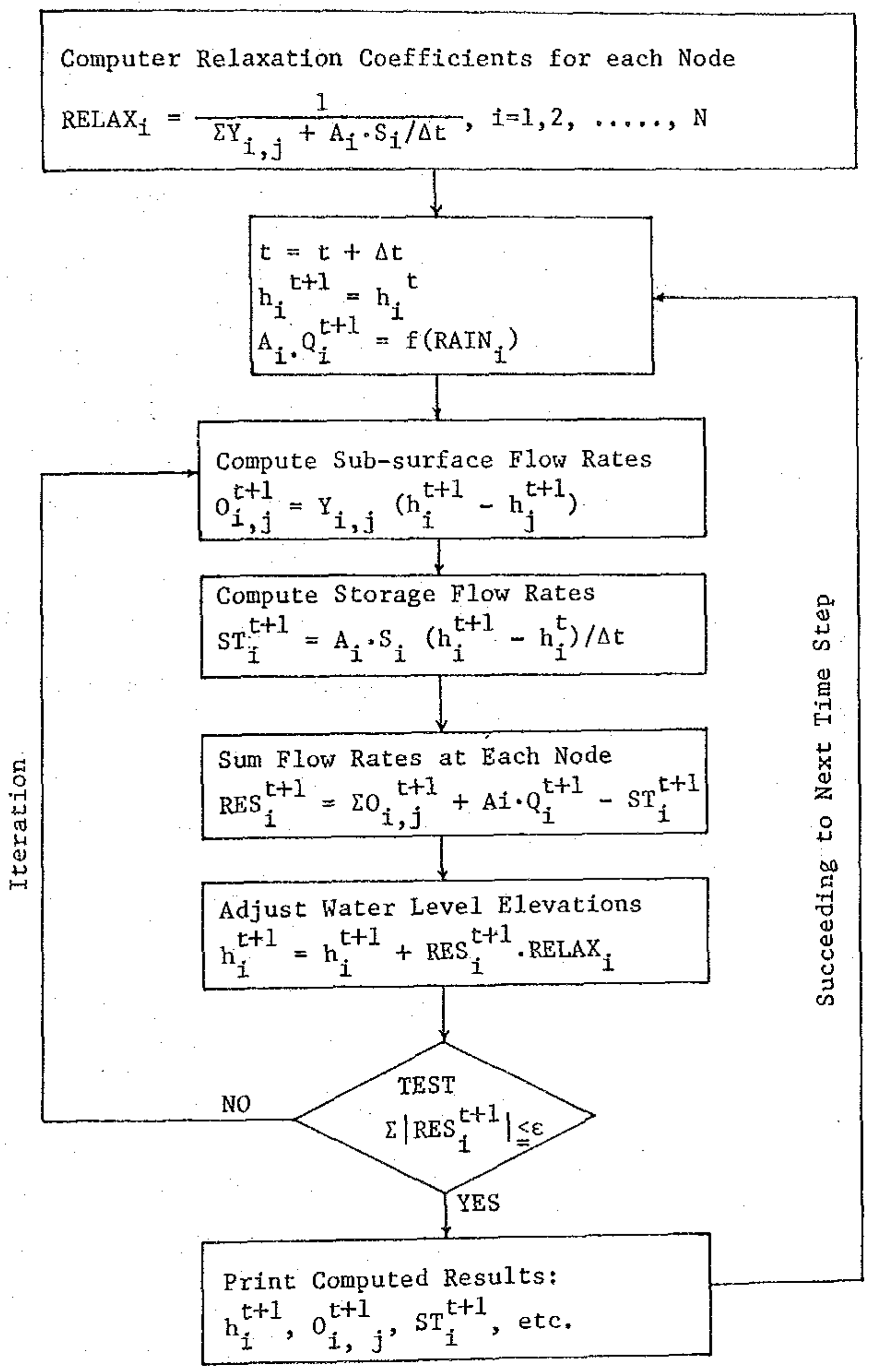


Figure B-17 Contour Line of Ground Surface Elevation

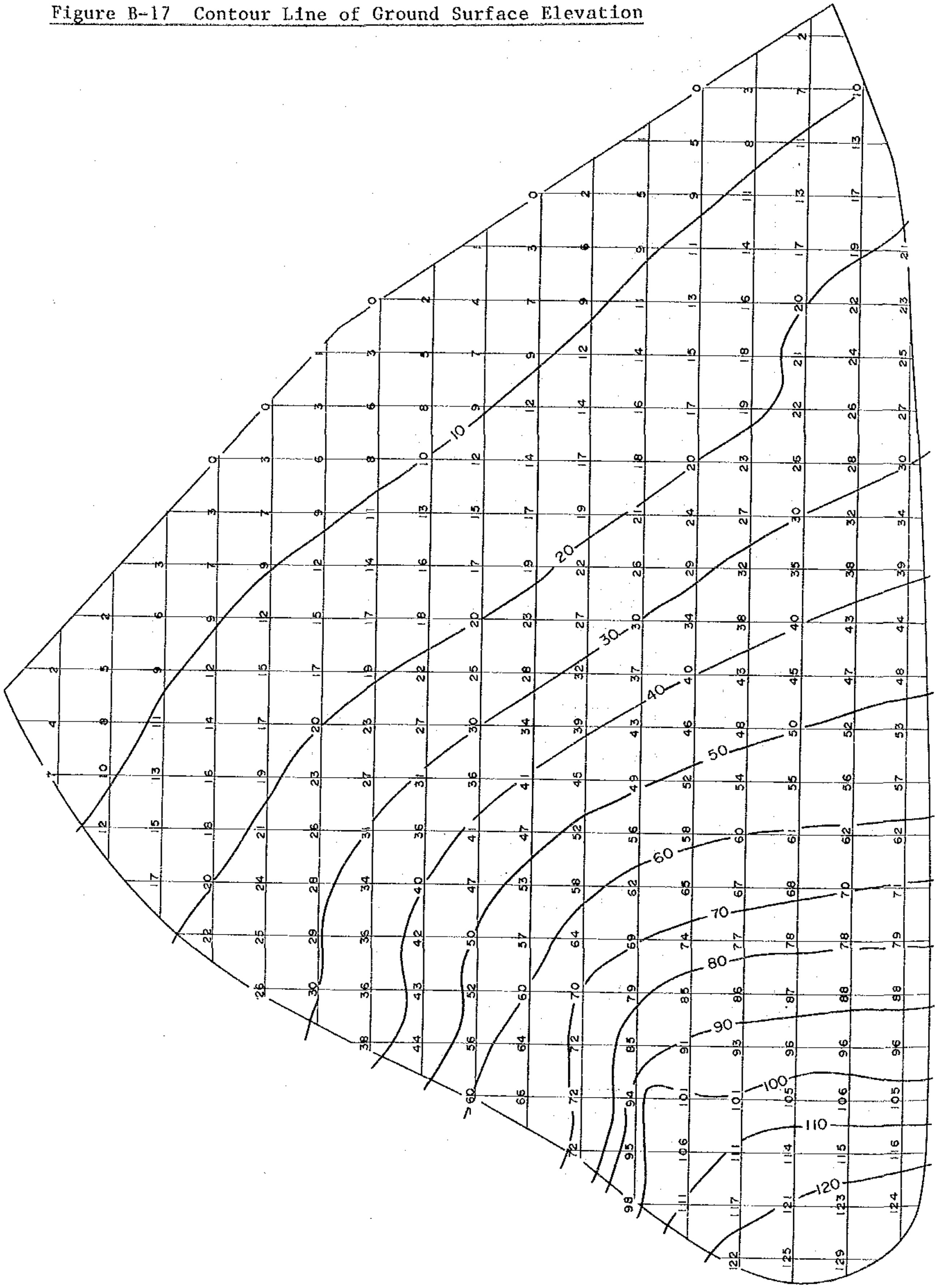


Figure B-18 Contour Line of Impermeable Foundation

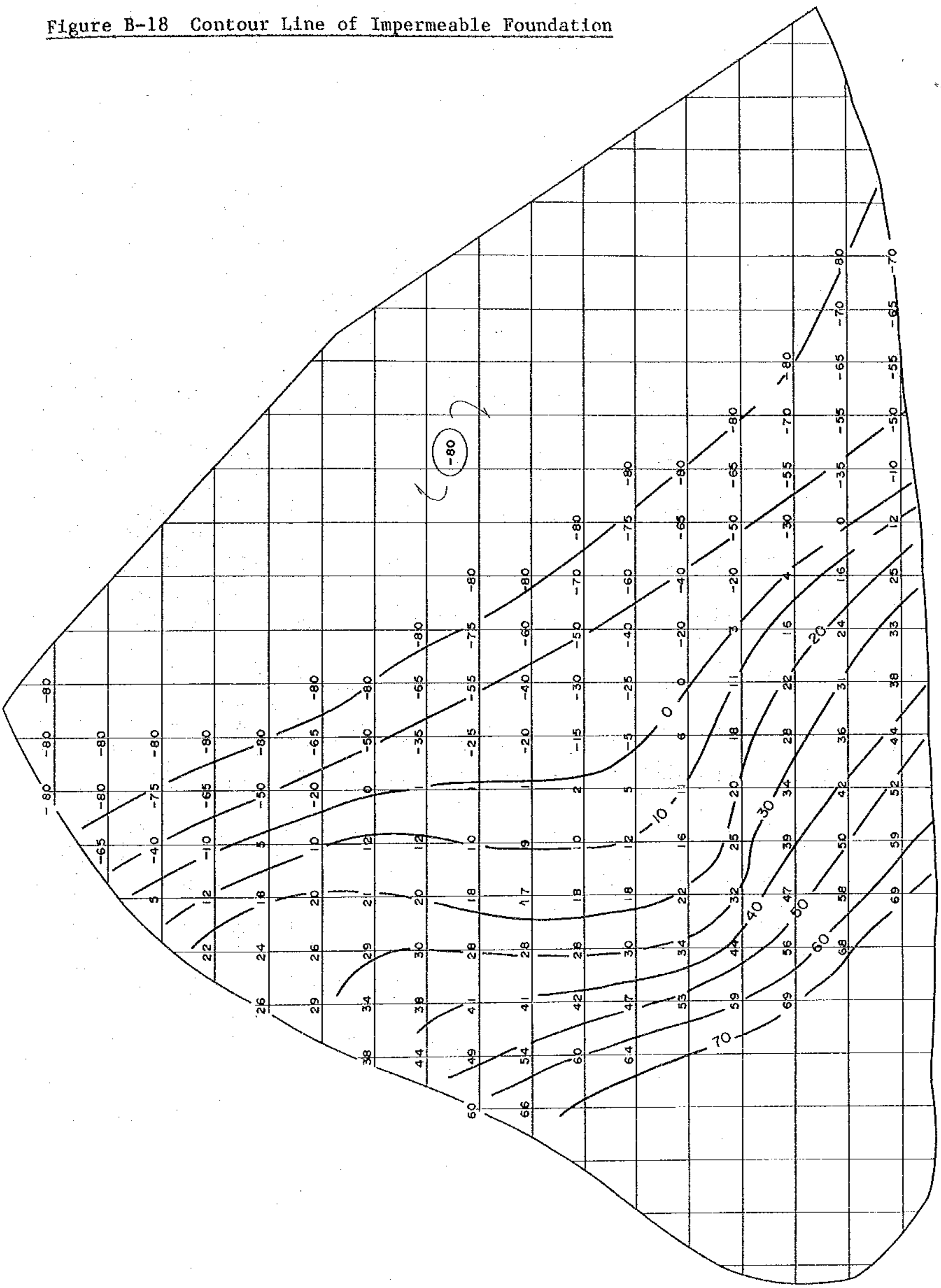


Figure B-19 Thickness of Permeable Layor

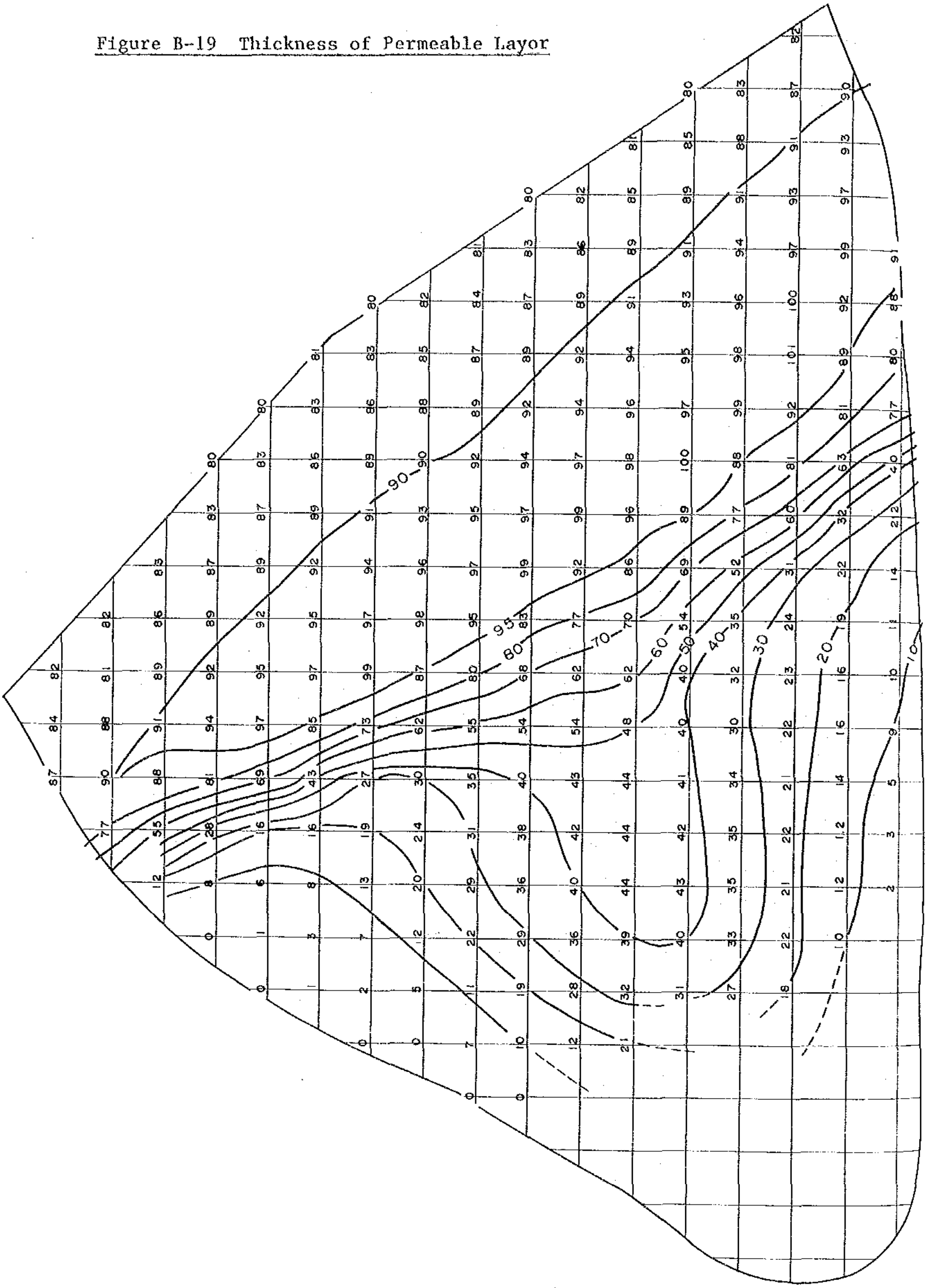
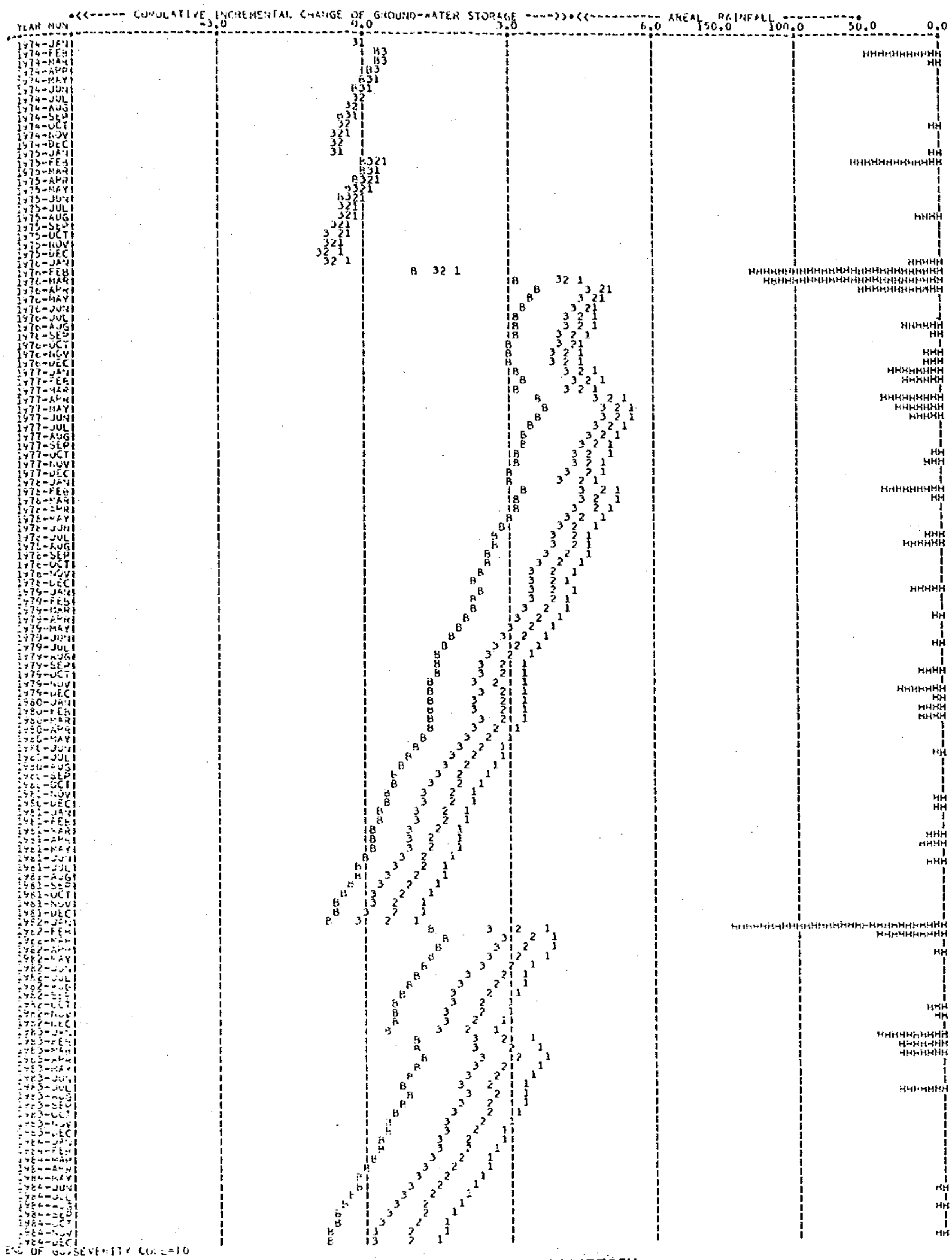
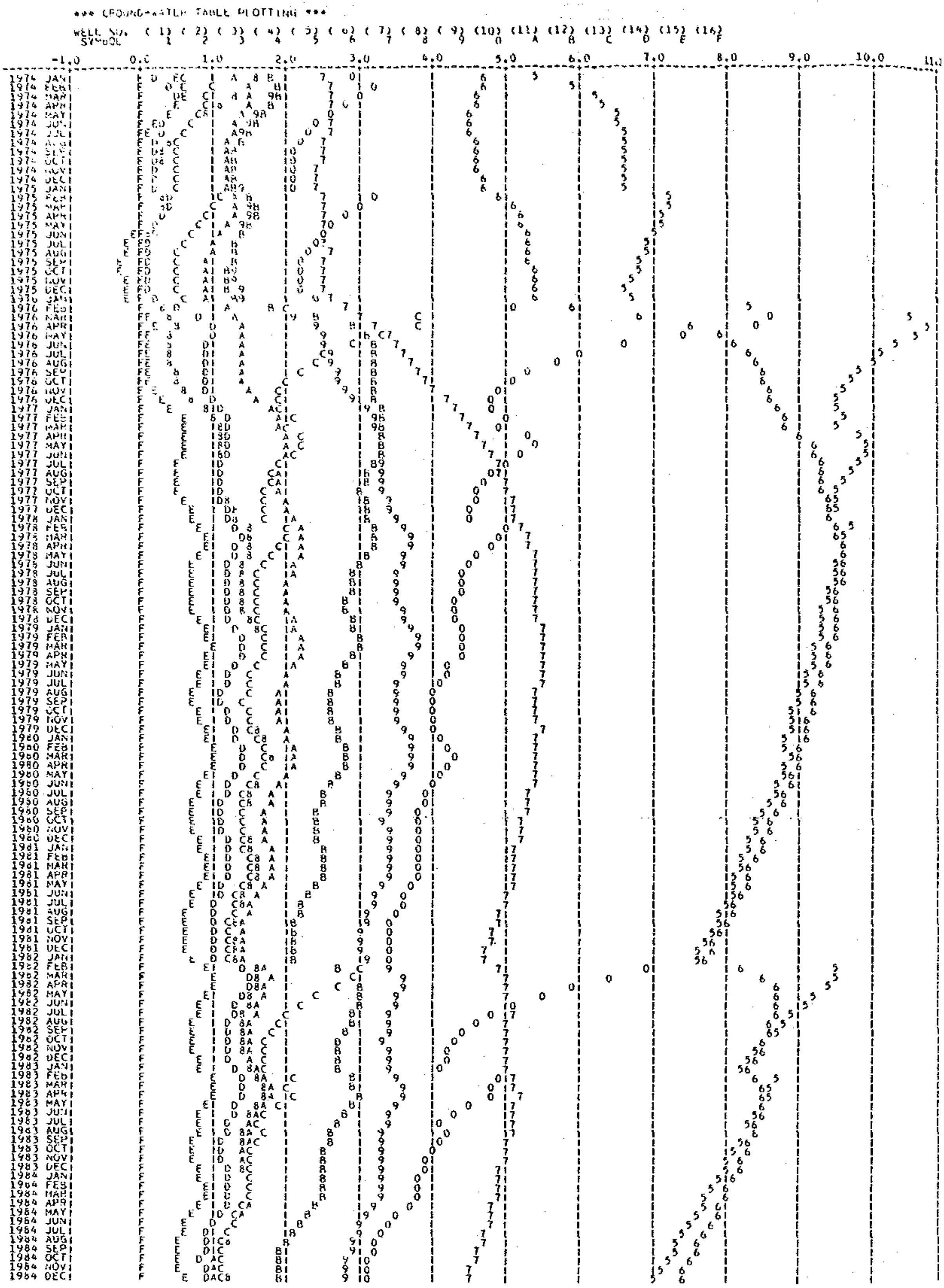


Figure B-20 Simulated Change of Groundwater Storage



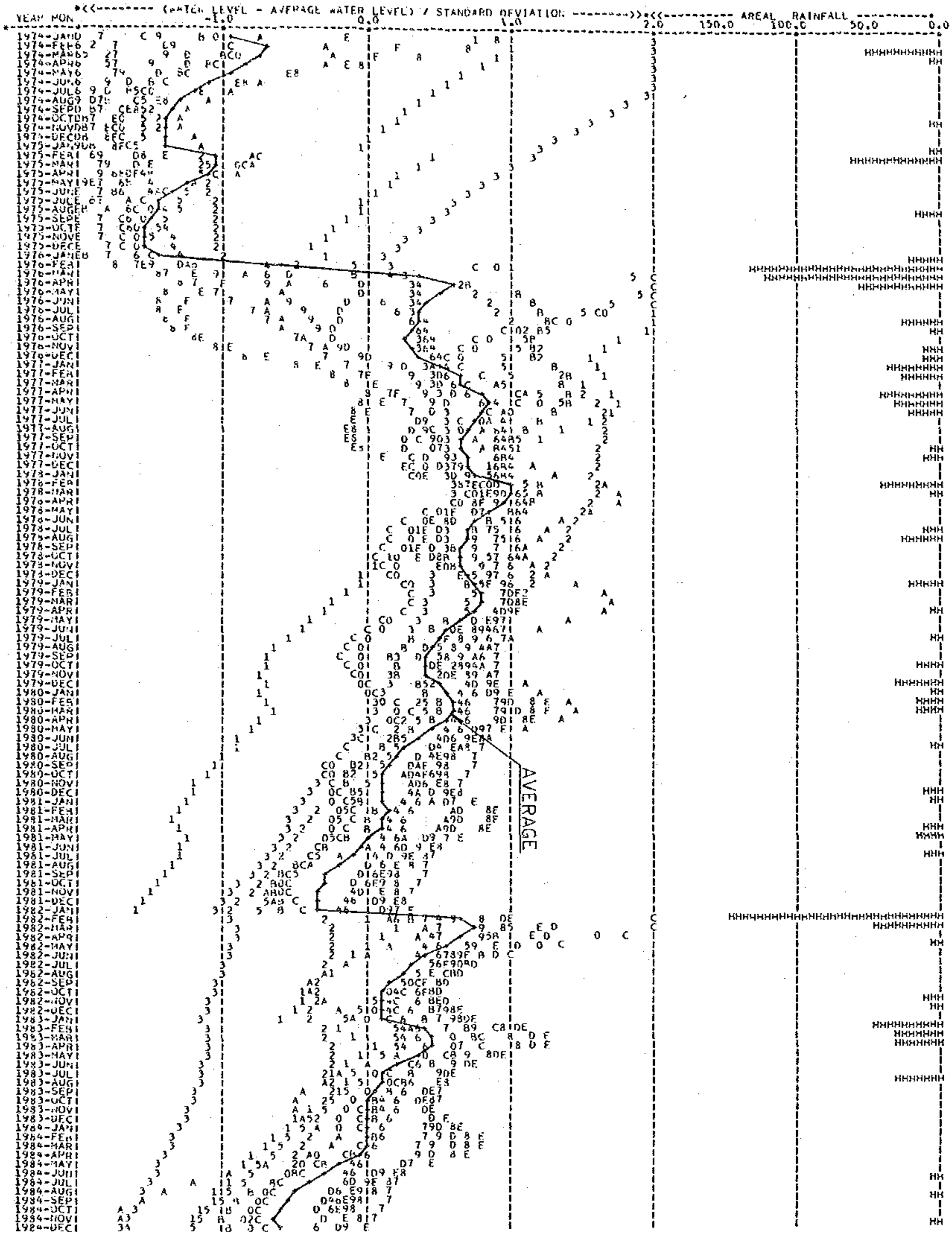
SYMBOL	DESCRIPTION
B	PRESENT CONDITION(BEFORE CONSTRUCTION OF DAM)
1	PROPOSED-1 (EXTRACTION SAME AS PRESENT)
2	PROPOSED-2 (PRESENT WATER USE + 1.26MCM/YEAR)
3	PROPOSED-3 (PRESENT USE + 1.26 + 1.39MCM/YEAR)
H	MONTHLY AREAL RAINFALL(MM)

Figure B-21 Simulated Groundwater Tables (Case 0)



SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	OA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	AE-61	B	AE-91
C	AE-99	D	14-M	E	OA-2	F	SEA

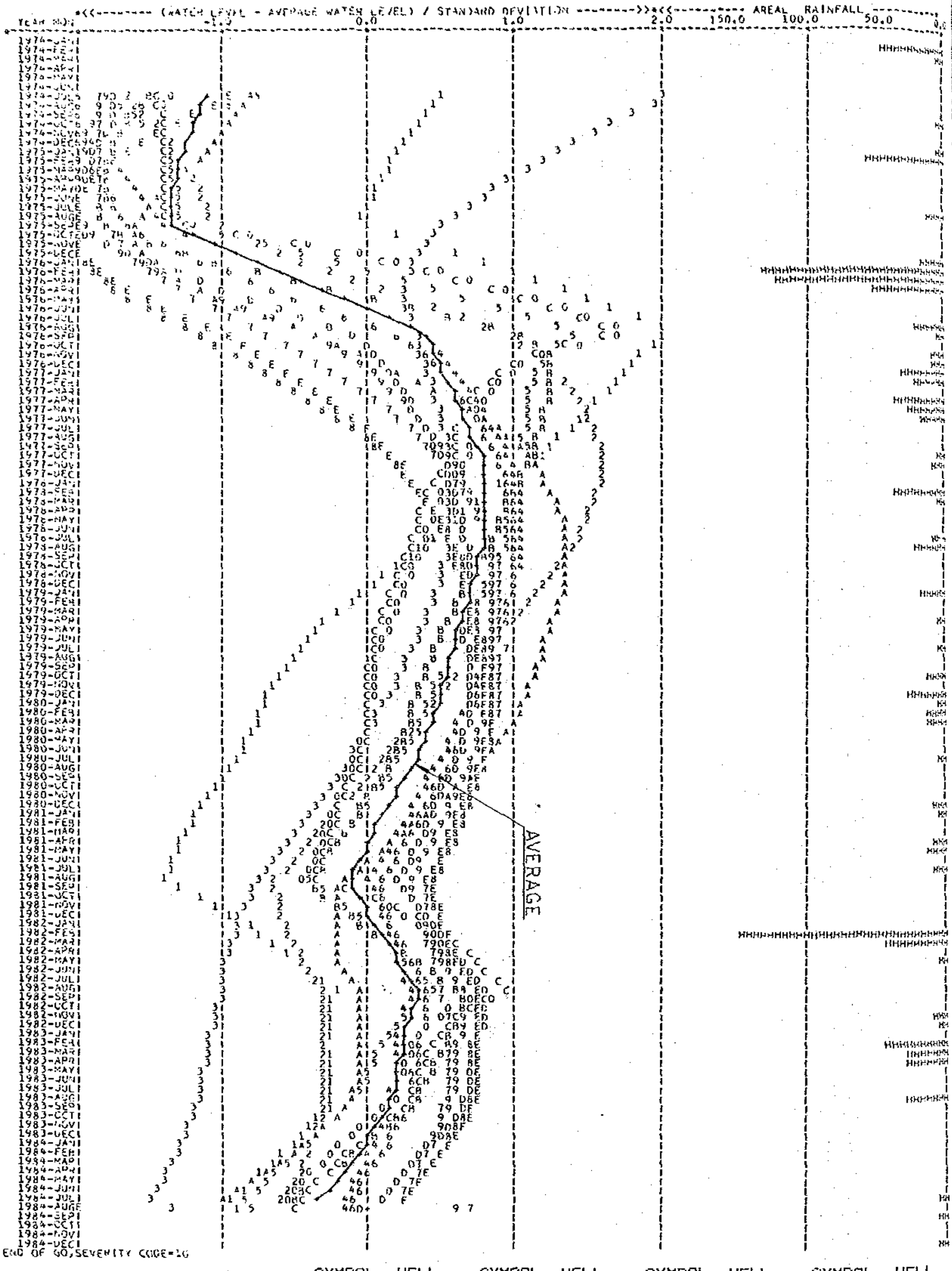
Figure B-22 Deviation of Groundwater Level (Case 0)



SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	OA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	AE-61	B	AE-91
C	AE-99	D	14-M	E	OA-2	F	SEA

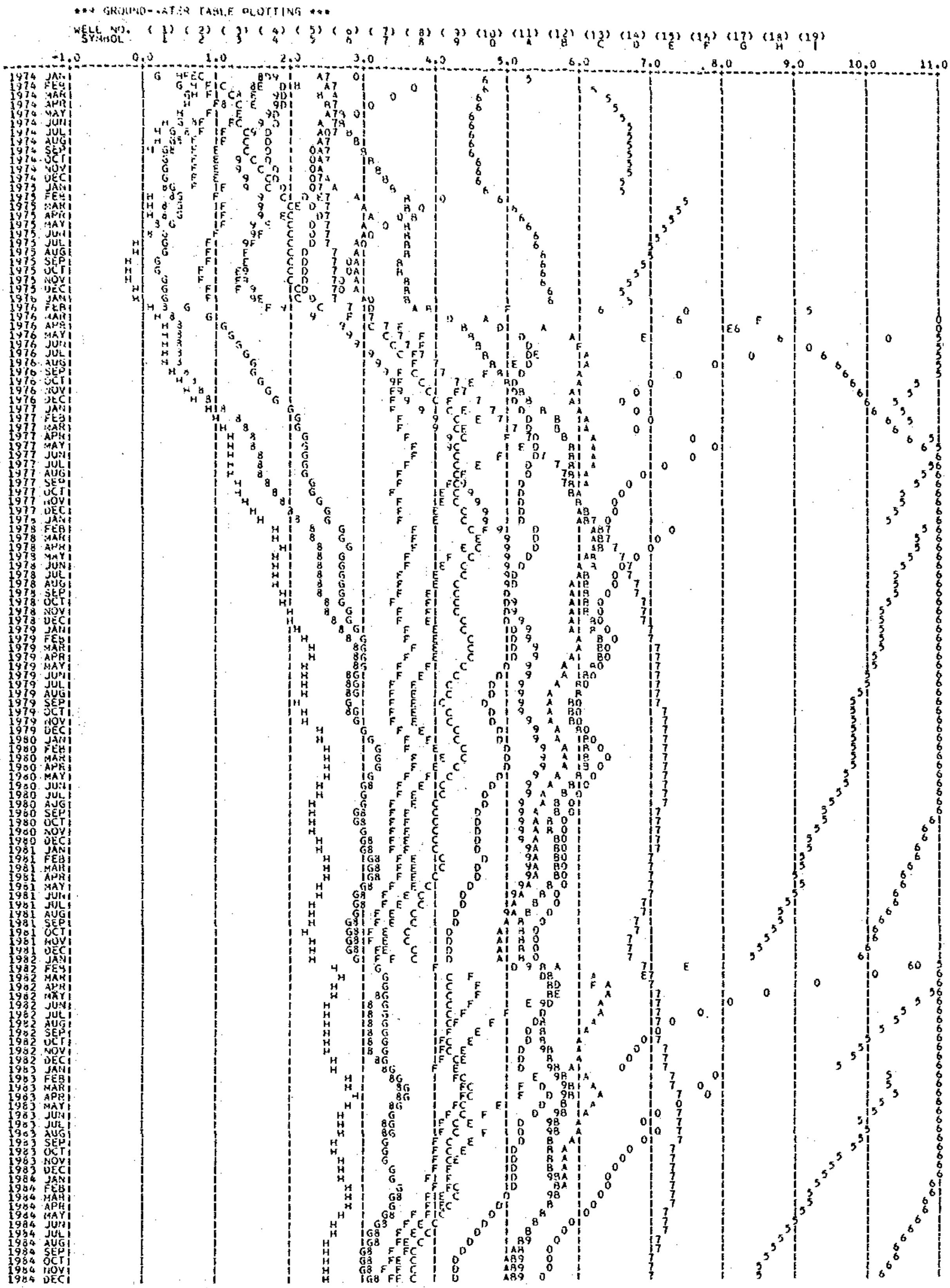
Figure B-23 Deviation of Groundwater Level (Case 0)

(12-months Moving Average)



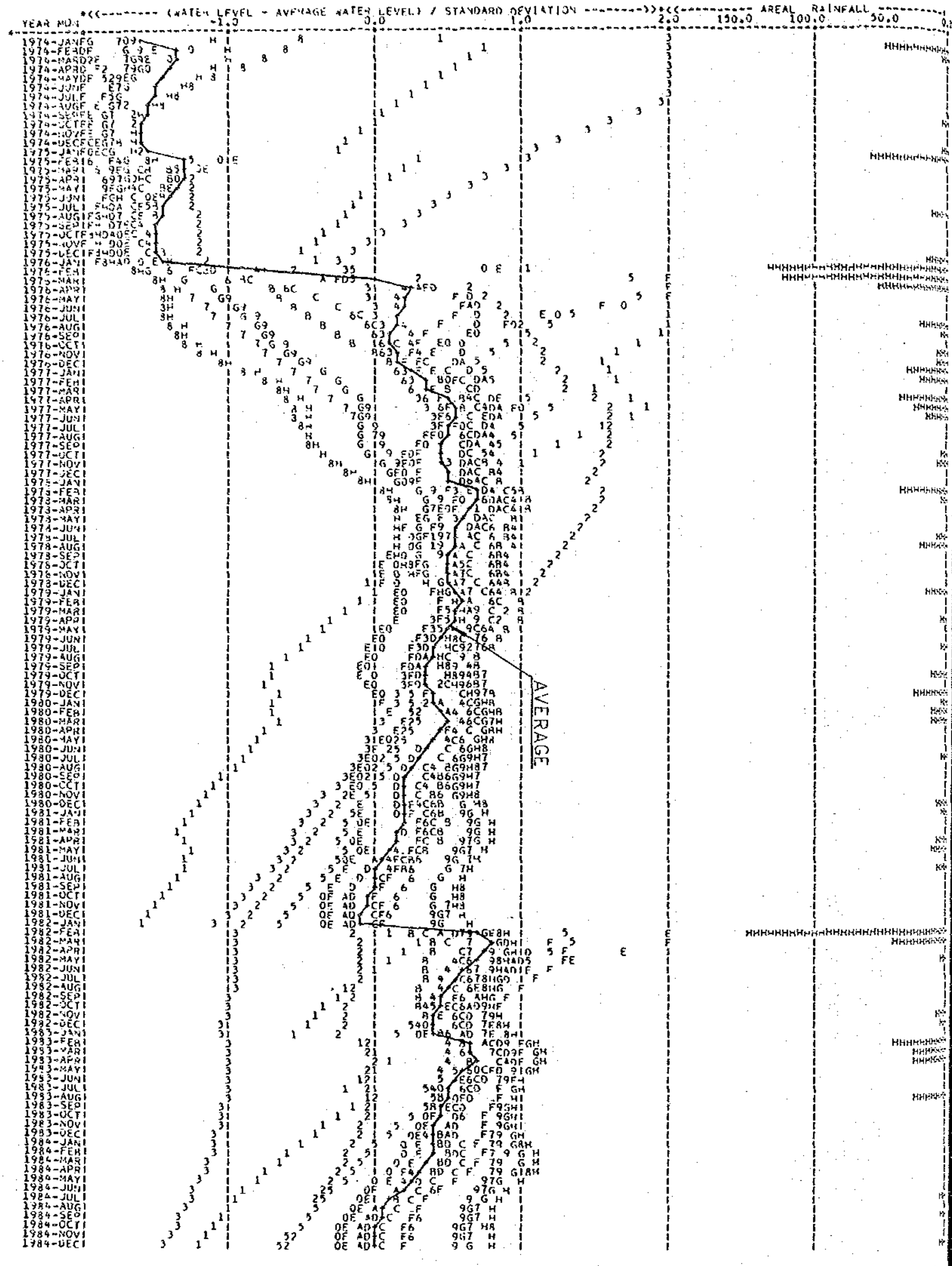
SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	DA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	AE-61	B	AE-91
C	AE-99	D	14-M	E	DA-2	F	SEA

Figure B-24 Simulated Groundwater Tables (Case 1)



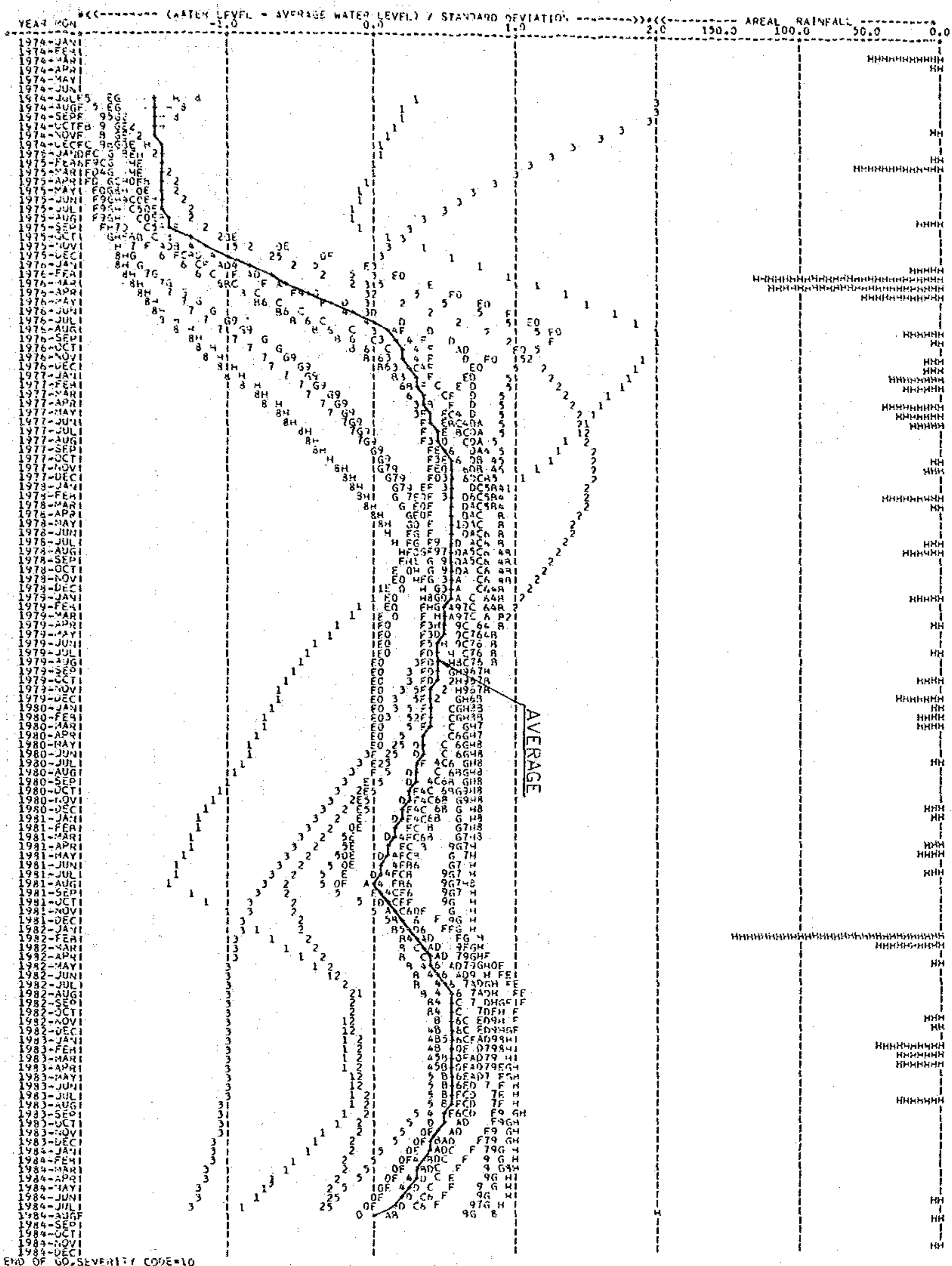
SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	OA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	NJ-1	B	AE-61
C	NJ-4	D	AE-91	E	NJ-3	F	AE-99
G	14-M	H	OA-2	I	SEA		

Figure B-25 Deviation of Groundwater Level (Case 1)



SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	OA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	NJ-1	B	AE-61
C	NJ-4	D	AE-91	E	NJ-3	F	AE-99
G	14-M	H	OA-2	I	SEA		

Figure B-26 Deviation of Groundwater Level (Case 1)



SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL	SYMBOL	WELL
1	OA-1	2	JA-6	3	JA-4	4	JA-5
5	JA-3	6	EA-1	7	JA-1	8	BOS-2
9	WSI-26	0	JA-2	A	NJ-1	B	AE-61
C	NJ-4	D	AE-91	E	NJ-3	F	AE-99
G	14-M	H	OA-2	I	SEA		