12.2 Aquifer Modeling

12.2.1 Previous Study

Numerical simulations of the Quaternary aquifers in El Qaa plain, El Arish area and Romana-Bir El Abd area have been carried out by RIWR in cooperation with EC (BRGM/RIWR, 1993a). El Arish and Romana-Bir El Abd areas (North Sinai) were modeled using MARTHE (a multi-layered quasi-three dimensional groundwater flow model developed by BRGM). The Calibration of the model was completed comparing observed and calculated piezometric heads and the groundwater balance based on the model was estimated.

In El Qaa plain, MODFLOW was applied. A single aquifer was assumed and the simulation area was divided by square mesh of 4 km^2. The distribution of the permeability was based on the results of 22 pumping tests. The source of recharge in the model was mainly the precipitation in the St. Catherine area. The mean rate of total groundwater abstraction was estimated to be approximately 10,000m^3/day. The piezometric head calibration of the model was completed in steady state condition. From the simulation study, it was concluded that the recharge of the El Qaa plain quaternary aquifer could be estimated between 20,000 and 30,000 m^3/day.

Hatem(1996) also, executed a groundwater simulation in El Qaa plain using MODFLOW. Same simulation area by RIWR in cooperation with EC was divided by square mesh of 1 km^2. From the runoff study of representative catchments (wadi Sudr and wadi El Shikh), he proposed rainfall-recharge equations for limestone and basement terrain. Using the rainfall-recharge equations, groundwater recharge was estimated to be 17,300 m^3/day (6.30 million m^3/year). Calibrated recharge with simulation model was obtained to be 21,300 m^3/day (7.77 million m^3/year).

Also, a numerical modeling of the Lower Cretaceous aquifer was tried by RIWR in cooperation with EC (BRGM/RIWR, 1993a). However, the calibration of the model couldn't be completed, because of the lack of exact data in Neguev area and the lack of knowledge of the boundary conditions in North and Western Sinai.

12.2.2 Selected Simulation Site

In South Sinai, the Quaternary aquifers are distributed in El Qaa plain, coastal plains and major wadis. The Pre-Quaternary aquifers appear in the Upper and Lower Cretaceous sedimentary rocks and the Precambrian basement rocks.

The El Qaa plain is largest Quaternary groundwater basin in South Sinai. A number of productive wells and piezometers have been drilled in the plain. Therefore, groundwater data (well log, pumping test, etc.) can be easily collected. Groundwater monitoring in the plain has been carried out, and water levels and chemical analysis data are stored in the databases. The Quaternary aquifer is considerably exploited for the water demand of domestic supply in El Tur and Sharm El Sheik cities, and irrigation use in the plain. In the northwestern part of the plain, salinity of groundwater is relatively high. The sea water intrusion is recognized at the coast near El Tur. Therefore, the El Qaa plain was selected to be groundwater simulation site.

Groundwater simulation in the plain was once carried out, however, groundwater flow was simulated only. To deal with salinity of groundwater or seawater intrusion problem, it is appropriate to introduce new solute transport model.

There are few data on the Quaternary aquifers in the other coastal plains or wadis and modeling of the aquifers in these areas are considerably difficult. The distribution of the Lower Cretaceous aquifer is studied in this project. Monitoring of water level will be continued after this project. The Modeling of the Lower Cretaceous aquifer is the subject for a future study based on the monitoring results.

12.2.3 Concept of Modeling

Results of hydrogeological study in the El Qaa plain is summarized in chapter VIII. Hydrogeological investigation clarified that there exist three aquifers in the study area. The first aquifer is unconfined, and occurs in the limited area around El Tur (Wadi Village and El Gabaal). It is composed of wadi and gravel deposits, and has been developed for rural water supply and irrigation use by dug wells. Also, water from dug wells is transported to Sharm El Sheik by tank trucks. The first aquifer was previously believed to be separated aquifer. As discussed in §8.2.1, the first aquifer is vertically separated from the second aquifer by silty zone of the first aquifer. It is however connected with the second aquifer horizontally. The first aquifer can be regarded as a part of the second aquifer.

The second aquifer consists of sand and gravel layers with interbeded clay. It is considered to be unconfined /semi-confined aquifer, and most of production wells in the plain have been drilled in this aquifer. This aquifer is not distributed in southern part of the El Qaa plain where basement rocks occur at the shallow depth. Based on the geophysical investigation, the second aquifer is bordered by a inferred fault which runs along Wadi Isla. As discussed in the following section, the second aquifer is supposed to be recharged from eastern Precambrian and Cretaceous mountains through fissures, joints and fault planes.

Several tube wells penetrate up to the third aquifer underlain by the second aquifer. The third aquifer is composed of sand and gravel layer with clay and silt. This aquifer is considered to be confined and recharged by leakage through confining layer. However, definition of confining layer is difficult due to lack of data.

As a result of the above discussion, an aquifer system consisting of two aquifers with one confining layer (aquitard) is recommended for the groundwater model in the El Qaa plain. One aquifer is unconfined and represents first and second aquifers, and the other is confined and represents third aquifer. Upper unconfined aquifer and lower confined aquifer are connected with leakage through confining layer (Table 12.2.3-1 and Fig.12.2.3-1).

12.2.4 Data Manipulation

1) Grid design

Considering hydrogeological structure of the study areas, modeled domains were defined as shown in Fig.12.2.4-1. Model area covers all of northern part of the El Qaa plain. Area of the domain is about 1,600 km^2 (24×66km). The domain was divided by grid at an interval of 1 km in both longitudinal and transversal directions.

In vertical direction, space was discretized to represent individual aquifers and aquitard by individual layers of the model. Also, the unconfined aquifer was divided into three (3) layers. The aquitard and confined aquifer were divided into three (3) and two (2) layers, respectively (Fig.12.2.4-2). Elevations of bottom of the unconfined (the first and second) aquifer, the aquitard and the confined (the third) aquifer were digitized for each mesh (See Tables.12.2.4-1 to 12.2.4-4).

2) Boundary Condition

(1) Flow Model

Boundary conditions for groundwater flow model is indicated in Fig.12.2.4-1. The model area is bordered by Pre-Cambrian mountains at the east. From the mountainous area, infiltrated rainwater recharges aquifers in the plain through fissures, joints and fault planes of fractured rocks. So then, this boundary can be defined as flux (recharge) boundary. Recharge rate is examined in the following section, and shown in Table 12.2.4-6. In the MODFLOW, recharge is inputted into top active cell.

In the north and northwest side, Tertiary-Cretaceous mountains are distributed. They are mainly composed of limestone. Recharge form these region is considered to be negligible. Therefore, no flux condition was set on the north and northwest boundary.

Gulf of Suez can be assumed as constant head boundary, assigning sea level to boundary cells of layers 1 to 3. At the south, the model area is bounded by an inferred fault. This boundary can be considered as no flux boundary.

(2) Solute Transport Model

Fig.12.2.4-1 shows boundary conditions for the solute transport model. At the constant flux boundary, quality of recharged water was defined for each wadi, as shown in Table 12.2.4-6. In the unconfined aquifer (layer 1 to 3), constant concentration boundary was set on the border of Gulf of Suez corresponding to the quality of seawater. Then, it was able to calculate the effect of seawater intrusion.

3) Aquifer Constants

In the second aquifer, pumping tests in twenty-three (23) wells were carried out. Eleven (11) tests of these were reanalyzed, then distribution of hydraulic conductivity was drawn in Fig.8.2.1-24. Based on this figure, distribution of hydraulic conductivity was zoned and inputted into the model. Four (4) pumping tests in the third aquifer were reanalyzed and average value was inputted into the model. From the value of leakage factor, hydraulic conductivity of confining layer was estimated.

It was assumed that hydraulic conductivity was isotropic in horizontal and the ratio of horizontal to vertical was 0.1. Initial inputs of hydraulic conductivity distribution of unconfined aquifer (layers 1-3) are shown in Fig.12.2.4-3. Hydraulic conductivity of aquitard (layers 4-6) and confined aquifer (layers 7,8) were assumed to be uniform and 0.15m/day and 1.5m/day were inputted respectively.

Specific storage was estimated at 1×10^{-5} /m and specific yield (effective porosity) was assumed to be 0.15.

There are no dispersivity data. A longitudinal dispersivity of 30 m was used, but the simulation results were relatively insensitive to adjustments of the dispersivity values.

The ratios of longitudinal dispersivity to transverse and vertical dispersivity were assumed to be 0.3 and 0.1, respectively.

4) Groundwater Level

The groundwater levels in the Quaternary aquifer have been periodically measured since 1989. Based on the results of the measurement, water table contour maps (Fig.8.2.1-14(1)-(3)) were drawn.

(1) Water Table

The contour maps indicate that groundwater in the second aquifer flows generally form northeast to southwest in the eastern part of El Tur. Contour lines are almost parallel to the coastline. Near the seashore contour lines are closely distributed. This phenomenon indicates existing of low transmissivity zone along coast, and it coincides with distribution of transmissivity based on the results of the pumping tests. In the northern part of the plain, groundwater flows generally from north to south.

These contour maps ware used to compare with calculated heads in model calibration.

(2) Variation of Water Level

There were seasonal fluctuations in water level. However, water levels have been almost stable on a long-term point of view. Therefore, recharge and discharge is balanced and steady-state can be assumed.

5) Water quality

(1) Groundwater Quality Index

Salinity in the groundwater is main problem in this plain. Therefore salinity or total dissolved solid (TDS) was used for quality index in the model.

(2) Distribution of TDS

Based on the chemical analyses and electrical conductivity measurements, distributions of TDS in the second aquifer were drawn in Figs.8.2.1-17-18. These figures show that fresh water (TDS <500 mg/*l*) flows from the area between Wadi Mir and Wadi Shadk toward the well field of city supply. This distribution of TDS indicates that a large quantity of fresh water is recharged from fractured basement rocks, which extend to the east of the El Qaa plain.

Near the coast of El Tur, high salinity water (TDS >7000 mg/*l*) is observed and it can be considered as seawater intrusion. At north of the line that links El Tur and Wadi Hibran, salinity of groundwater is high (TDS 1000-3000 mg/*l*) and there is a tendency for low salinity water to distribute in the eastern part of the area. It is considered that high salinity in this area is due to evaporites and shales within the aquifer.

These contour maps ware used to compare with calculated heads in model calibration.

(3) Seasonal fluctuation

Distributions of TDS in summer (Sep-96) and winter (Feb-97) indicate considerable difference. Also, variation of TDS (Figs.8.2.1-20) shows seasonal fluctuation, which is supposed to be attributed to seasonal change of recharge. Simulating seasonal fluctuation of TDS is difficult due to lack of seasonal recharge data. So then, annual average of TDS was used in the model calibration.

(4) Chemical Reactions

The increase of groundwater salinity in the northern part of the plain is attributed to the frequent occurrence of evaporites and shales with in the aquifer. The MT3D can include simple chemical reaction process of groundwater and aquifer material. However, complex chemical reactions of chlorides, carbonates and sulfates can not be included. Also, there is no data of evaporite content in the aquifer. At present, chemical reactions were neglected in the model. This problem is one of the subjects to future study.

6) Pumpage

(1) Previous Study

The pumpage from the second aquifer has been reported in several studies (Table 12.2.4-5). However, the values of pumpage are considerably different each other. Also, the development of groundwater extraction during the period 1972 - 1992 is summarized in the following table. The extraction had rapidly increased in 1980's and reached at its peak in 1987. After 1987, amount of pumpage had gradually decreased and the average pumpage rate in 1992 was 8,820 m^3/day. On the other hand, groundwater discharge in 1992 was estimated at 4800 m^3/day based on unit water consumption per capita (Hatem, 1996).

Groundwater Extraction in the El Qaa Plain during the Piriod 1972-1992

(BRGM/RIWR, 1993b)

Year	Number of wells		Average Groundwater Extraction m^3/day		
	Irrigation	Domestic	Irrigation	Domestic	Total
1972	2	3	500	1,700	2,200
1984	14	6	5,350	3,200	8,550
1987	14	9	6,540	6,500	13,040
1990	14	9	2,310	7,960	10,270
1992	10	8	830	7,790	8,820

(2) Present Groundwater Use

There are no flow meters for each well, so then exact figure of groundwater extraction can not be determined. At present, there is no way but to estimate it based on more certain data.

The value of production capacity in statistical data book 1998 is adopted as domestic use in El Tur; i.e. 6,000 m^3/day (2.19 million m^3/year). Nine (9) wells (Sr. no. 24, 26, 28-31, 37, 38, and 47; numbers are corresponded to these in well inventory) are existing for domestic use. Two (2) wells (Sr. no. 31 and 47) are not in use now.

Water to Sharm El Sheik from tubed wells is estimated at 1,000 m^3/day (0.36m^3/year) based on the result of interview to Sharm El Sheik City (See chp.10). There are two (2) wells (Sr. no. 27 and 39) for water transportation to Sharm El Shiek.

Five (5) wells (Sr. no. 17, 19, 34, 40, and 41) are now in use for irrigation. There is no data of extraction rate, so then pumpage for irrigation is estimated in proportion to number of wells. In 1992, pumpage rate was 830 m^3/day by ten (10) wells (Table 12.2.4-3). Irrigation use can be estimated at 415 m^3/day (0.15 million m^3/year).

Total extraction rate from second aquifer becomes 7,415 m^3/day in annual average (2.70 million m^3/year). Seasonal fluctuation of groundwater discharge can not however be estimated.

There are nineteen (19) dug wells in Wadi Village, and twelve (12) dug wells in El Gabaal. Water from dug wells is transported to Sharm El Sheik, and amount of water can be estimated at 1,500 m^3/day based on the result of interview to Sharm El Sheik City (See chp.10). Total amount of pumpage from dug wells is estimated 2,000 to 3,000 m^3/day at present (by WRRI El Tur Office). In this project, 2,000 m^3/day in annual average (0.73 million m^3/year) was adopted as extraction rate of dug wells in moderate estimate.

As a result, total pumpage in the El Qaa plain can be estimated at 9,415 m^3/day in annual average (3.43 million m^3/year).

Depth and water level of dug wells are all different within a mesh. The aquifer model with 1-km width grid can not express subtle changes in topography. So then, a hypothetical well that represents dug wells within a mesh was assumed. The average depth and accumulated pumpage of dug wells was applied for the hypothetical well.

(3) Future Groundwater Use

Future water demand is forecasted from NPDS (National Project for Development in the Siani,1994) ,as discussed in §10.2. In NPDS, population in El Tur is planed to increase to 110 thousand. With increase in population, water demand for domestic use will grow. It is estimated about 30,000m^3/day (10.8 million m^3/year) in 2017.

Additional groundwater requirement will be developed in the unconfined ("second") aquifer. From the salinity constraint, developing area is restricted within the northeastern part of El Tur, where production wells for domestic use have been drilled.

There is no precise development plan for irrigation in NPDS. Requirement of water quality for irrigation is less severe, then development area can extend to outside of the domestic well field

7) Groundwater Recharge

There are several studies on groundwater recharge in the El Qaa plain (Table 12.2.4-5). In Sinai Development Study (Dames and Moore, 1985), amount of recharge was estimated at 66,000 m^3/day (24.1 million m^3/year) from runoff analysis. However, it was not based on a field runoff data. BRGM/RIWR (1993b) estimated the recharge rate at 31,500 m^3/day (11.5 million m^3/year) by means of flow net method. Hatem (1996) built following recharge equations form runoff analysis of representative catchments;

For limestone terrain: $RE = 0.21(RF - 5)$ (12.2.4-1)

For basement terrain: $RE = 0.40(RF - 7)$ (12.2.4-2)

Where, RE: average annual recharge in mm, RF: average annual rainfall in mm.

Using these equations, the recharge was calculated as 17,249 m^3/day (6.29 million m^3/year).

At first, model calibration was executed using these figures (Table 12.2.4-6). In the process of calibration, recharge rates for each wadi were modified as discussed in §12.3.

8) Input Data

Above mentioned aquifer constants, pumpage and recharge were assigned to each cell. Also, initial water level and concentration of TDS were allotted to each cell. Steady state was assumed, so then, arbitrary values were applied to initial conditions.

12.3 Model Calibration

12.3.1 Procedure of Model Calibration

Model calibration was carried out in the following steps;

a. Calibration of the steady-state flow

b. Calibration of the quasi-steady state solute transport

The first step of model calibration is to clarify the aquifer parameters and recharge rate. As mentioned above, variation of water level since 1989 can be neglected. So then, steady state flow was calculated. Calculated head was compared with observed water level, then parameters (mainly hydraulic conductivity) and recharge rate were modified until final agreement between the calculated and the observed water level is achieved.

In the second step, TDS distribution was calibrated to fix the transport parameters. The MT3D calculates the transient solute transport using the results of steady state or transient flow calculation. TDS of groundwater has been almost stable. Therefore, calculation was continued reaching steady state. In practice, calculation period of 1000 years was used. Calculated TDS was compared with observed data, and the transport parameters were revised to complete the coincidence between them. Also, flow parameters were slightly modified at this stage.

12.3.2 Calibrated Model

1) Comparison between Observed and calculated results

(1) Water Level

Computed water level was compared with field observations. Fig.12.3.2-1 shows observed and computed water level configurations of the unconfined aquifer. In this

Figure, dry cell means that elevation of bottom of the cell is higher than water table. The 5-m and 10-m contours of the both are almost coincided. Difference between the two of 20-m contour is slightly large. But, 20-m contour of measured water level was estimated, due to no observation point.

Water level in the confined aquifer is shown in Fig.12.3.2-2. Few observation points exist in this aquifer therefore comparison is difficult.

Fig.12.3.2-3 shows comparison of observed and calculated water level at observation points. If the both values coincide, corresponding point is plotted an a line of 45 ° angle. All point distributes along the line of 45 ° angle. The mean error that is the mean difference between measured and simulated water level is 0.7m. The results indicate good agreement of the two.

(2) Water Quality

Observed and calculated TDS distributions of the unconfined aquifer are shown in Fig.12.3.2-4. The calculated results generally reproduce configuration of TDS distribution. At east of El Tur, computed extension of 500 mg/*l* contour that reaches to domestic well field is smaller than the observed. Near the coast of El Tur, Calculated TDS is lower than the measured.

Fig.12.3.2-5 shows calculated TDS in the confined aquifer. Comparison of observed and calculated TDS is difficult due to scarcity of observation point.

Comparison of observed and computed TDS at observation points is indicated in Fig.12.3.2-6. Almost of points range along the line of 45 ° angle. The mean error is 323 mg/*l*.

Upstream finite difference was used for advection calculation, although it generated numerical dispersion. Hybrid MOC and MOC method gave same results that were more irregular than the upstream results. In this study average distribution of TDS was assumed to be investigate, so then smooth results calculated by upstream method was employed.

This model has several limitations. Density change of water is ignored, so then density dependent flow can not be computed. The chemical reaction of salinity is ignored and the source of salinity is assumed to be in recharged water. Also, sensitivity of TDS change to the change of heads is small as a characteristic of the numerical model.

Considering these limitations, calculated and observed TDS agree well.

2) Fixed Parameters

Hydraulic conductivity, recharge rate and recharge concentration were modified to calibrate the model. Fixed hydraulic conductivity of unconfined aquifer (layers 1-3) is shown in Fig.12.3.2-7. At the domestic well field, hydraulic conductivity was doubled. Near Safariat, hydraulic conductivity was decreased to agree with the second aquifer thinning.

Calibrated recharge rate and recharge concentration is presented in Table 12.2.4-6. The recharge rate increased to about 160% of initial value, and it was intermediate value between the results of BRGM/RIWR(1993b) and Hatem(1996).

3) Water Balance in Calibration Period

Water balance in calibration period is given in the below table

		m^3/day	million m^3/year	%
Recharge		16,175	5.9	100
Discharge	Pumpage	9,415	3.44	58
	Flow to Sea	6,790	2.47	42
	Total	16,205	5.91	100
(Recharge)-(Discharge)		-30	0.00	-0.2

*{(Recharge)-(Discharge)}/(Recharge)

12.4 Model Prediction

12.4.1 Development Plan

1) Discharge

According to the future groundwater requirement, the following cases of development plan were studied:

Study Case for Development Plan

Case	Additional Pumpage m³/day	Additional Pumpage 10^6 m³/y	Number of Wells	Total Pumpage m³/day	Total Pumpage 10^6 m³/y	Recharge m³/day	Recharge 10^6 m³/y
1	10,585	3.86	11	20,000	7.30		
2	5,585	2.04	6	15,000	5.47		
3	3,085	1.13	3	12,500	4.56	16,175	5.90
4	585	0.21	1	10,000	3.65		
5	5,585	2.04	6	15,000	5.47		

Case 1 to 4 are assumed to exploit domestic water, and they aim to investigate the effect of development to the domestic well field. Additional groundwater discharge is extracted from the unconfined aquifer. From the salinity constrain, developing area is restricted within the northeastern part of El Tur, where production wells for domestic use have been drilled. Extraction rate of an additional well was supposed to be 1,000m³/day.

Case 5 is assumed to develop to irrigation water at the north of the domestic well field. It aims to examine the effect varying developing area purpose of water use.

2) Prediction Period

Prediction period for each is 20 years for the development and 100 years after the development (Fig.12.4.1-1). The object aquifer is unconfined, therefore the effect of development should be observed for a long duration.

3) Recharge

Recharge rate will vary in prediction period, but there is no precise recharge analysis to obtain probability of recharge at present. Calibrated recharge rate was used as average of long period.

12.4.2 Constraints on Groundwater Development

1) Water Balance Constraint

Groundwater abstraction should be within the possible amount of recharge on arbitrary spatial and time scales. In this study time scale is determined to be one year, therefore annual amount of pumpage must not exceed that of the possible recharge. In other words, the groundwater level must recover to the initial level at the end of one hydrologic year. Following table shows the criteria for the annual residual drawdown.

Criteria for Water Balance Constraint

Rank	Annual Residual Drawdown (m)	Description
A	0.00 - 0.02	Not surely safe, but allowable if there is no alternative plan
B	0.03 - 0.01	The aquifer storage will be possibly depleted in future
C	0.11 -	The aquifer storage will be probably depleted in near future

2) Water Quality Constraint

In cases 1 to 4, the use of groundwater is domestic, so then the drinking water quality standards is applicable. Limitation of TDS in the Egyptian standards is 1,500mg/*l*. Criteria on the water quality are shown in the following table. For case 4, irrigation water is developed, then 3,000mg/*l* of TDS is allowable.

Criteria for Environmental Constraint

Rank	TDS	Description
A	- 1,000	Good : Good quality
B	1,000 -1,500	Allowable : Slightly poor quality, but not exceed drinking water quality standards
C	1,501 -	Not allowable : Exceed drinking water quality standards

3) Environmental Constraint

Influence on existing dug wells is a possible environmental constraint to be considered. Depth and water level of dug wells are all different. Average water depth is 2.0 m, however there are wells with water depth less than 0.5 m. Therefore, influence on the existing dug wells is examined as follows:

Criteria for Environmental Constraint

Rank	Total Residual Drawdown (m)	Description
A	0.00 - 0.50	Allowable: No problems for practical use
B	0.51 - 2.00	Undesirable : Partly damaged
C	2.01 -	Not allowable : Damaged

4) Economical Constraint

This constraint comes from the limits of pump lift. Expected drawdown of existing wells were designed at 5-20 m (REGWA, 1982). As a result, the total drawdown should be within about 10-m. Otherwise the pump must be altered to suit deeper water level, which will cost more. Criteria for the economic constraint are shown in the following table.

Criteria for Economic Constraint

Rank	Total Drawdown (m)	Description
A	0-10	Good : No problems in practical use
B	10-20	Allowable : Well yield may decrease
C	20>	Undesirable : Pump should be changed

12.4.3 Model Prediction

1) Case 1

Groundwater abstraction from unconfined aquifer amount to 20,000 m^3/day and it exceeds the recharge rate largely. Therefor, serious drawdown of groundwater table will be experienced at 120 years after the development started. Fig.12.4.3-1 show water level of the unconfined aquifer at the time.

Maximum drawdown at the domestic well field exceeds 5.5 m (Fig.12.4.3-2). Water level decline will continue after the calculation end (Fig.12.4.3-16). Such drawndown corresponds to rank B (undesirable) of water balance constraint, but rank A (good) of economic constraint. Drawdown at Wadi Village reached 4 m, and this corresponds to rank C (not allowable) in environmental point of view.

TDS distribution of unconfined aquifer after 120 years is shown in Fig.12.4.3-3. Extent of 500 mg/*l* contour to the domestic well field is considerably reduced. TDS concentration of existing domestic well increases and exceeds 1,000 mg/*l* (Fig.12.4.3-17). This corresponds to rank B (undesirable) of water quality constraint.

At the coastal area, increase of TDS is inconspicuous. Drawdown at the coastal area is smaller than that at the domestic well field. Water level is still higher than sea level, then seawater intrusion is restrained.

2) Case 2

Total pumping in this case amount to 15,000 m^3/day, which is less than is less than recharge rate. Fig.12.4.3-4 shows water level of unconfined aquifer at 120 years after development started. Drawdown at the domestic well field reaches 3 m in maximum (Fig.12.4.3-5). Form water balance constraint, this case is classified into rank B (undesirable). Although, drawdown at Wadi Village reaches 2 m and this corresponds rank C (not allowable).

Fig.12.4.3-6 shows TDS distribution of unconfined aquifer after 120 years. Change of concentration is not remarkable comparing with Case 1. TDS increase of existing domestic well is moderate, and the concentration is less than 1,000 m^3/l (Fig.12.4.3-17). This figure is satisfactorily good range

3) Case 3

Pumping rate from unconfined aquifer is 12,500 m^3/day in this case. Water level of unconfined aquifer at 120 years after development started is indicated in Fig.12.4.3-7. Drawdown at the domestic well field is 1.2 m in maximum, and this corresponds to rank A (allowable) for water balance constraint (Fig.12.4.3-8). Drawdown at Wadi Village becomes 0.8 m maximum. From environmental constoraint, such drawdown is matched with rank B (undersirable).

TDS distribution at 120 years after is shown in Fig.12.4.3-9. There is little difference between this case and the present in configuration of TDS distribution. Increase of TDS concentration in existing domestic well is small, and the concentration is less than 700 mg/l (Fig.12.4.3-17). This figure corresponds to rank A (good) for quality constraint.

4) Case 4

Groundwater extraction is 10,000 m^3/day, which shows little increase from the present extraction. Fig.12.4.3-10 show water level of unconfined aquifer at 120 years after development started. Configuration of water level is almost same at the present. No contour can drawdown map (Fig.12.4.3-11). At existing domestic well, variation of water level is insignificant (Fig.12.4.3-16). This situation corresponds to rank A (allowable) for water balance and environmental constraints.

TDS distribution after 120 years (Fig.12.4.3-12) shows almost same configuration of the present distribution. TDS concentration of existing domestic well is stable (Fig.12.4.3-17). This case classified into rank A (good) for water quality constraint.

5) Case 5

Pumpage rate is same as Case 2, but additional is assumed to use for irrigation. Fig.12.4.3-13 shows water level after 120 years of the unconfined aquifer. Water level declines at new irrigation well field at the north of the domestic well field. Drawdown reaches 4.5 m at the irrigation well field, while one at the domestic well field is 2 m. At Wadi Village drawdown comes up to 2 m (Fig.12.4.3-14). Water balance constraint is classified into rank B (undesirable), but environmental constraint corresponds to rank C (not allowable).

Fig.12.4.3-15 show TDS distribution after 120 years. High salinity groundwater is pumped at new well field, so then 500 mg/*l* contour extends remarkably to the north. Also, TDS concentration of existing domestic well is slightly reduced (Fig.12.4.3-17). Water quality constraint for this case is classified into rank A (good).

12.5 Groundwater Abstraction Potential

12.5.1 Summary of Model Simulation

1) Water Balance

Result of water balance in prediction period are summarized in Table 12.5.1-1. In this table "storage" means water released from groundwater storage due to water level decline.

2) Result of Simulation

Results of the model simulation in the El Qaa plain are summarized as bellow:

Relation to Expected Constraints

Case	Water Balance	Water Quality	Environmental Impact	Economical
1	UD	A	NA	G
2	UD	G	NA	G
3	A	G	UD	G
4	A	G	A	G
5	UD	G	NA	G

Remarks; G=Good, A=Allowable, UD=Underirable, NA=Not Allowable

12.5.2 Groundwater Abstraction Potential

From the above results, both development plan of Case 3 (total pumpage from the El Qaa plain is 12500 m^3/day) and Case 4 (total pumpage is 1000 m^3/day) are acceptable.

In the case of development plan of Case 3, additional groundwater abstraction amount to about 3000 m^3/day. This value will be a groundwater abstraction potential in the study area. However, expected drawdown reaches 0.8 m at Wadi Village, it can be expected that the development causes slight influence on the existing dug wells.

Case 2 or Case 5, of which additional groundwater extraction is about 5,500 m^3/day, is less allowable, due to lack of water balance. In Case 2, increase of TDS is not remarkable. However, sensitivity of concentration to head change is low in solute transport computation, so then probable change of quality can be supposed.

Comparing with Case 2, results of Case 5 indicate that extraction for irrigation at the north of domestic well field is effective preserve the existing domestic wells.

12.5.3 Future Improvement Simulation

The modeling of the El Qaa groundwater basin is based on the knowledge at present. Therefore, modification of model will be expected in future investigations.

Groundwater extraction is relatively easy to investigate. To examine accurate extraction of groundwater, installation of flowmeter for each well is recommended.

Continues observation of water level over long period is important to estimate recharge rate and storage change of aquifer. It is recommended to install self-recording water level recorder for observation wells.

Periodical investigation of water quality is important to check the relation between quality and water level. Also, it is useful in preservation of groundwater. Measurement of EC once a month is recommended.

Almost of pumping is withdrawn in the domestic well field, but no observation well exists there. That may decrease accuracy of simulation model. It is recommended to construct new observation well, which useful in management of groundwater.

Model calibration should be improved based on future studies above mentioned. Sensitivity of solute concentration to water level change should be investigated using long term observation data.

The MT3D used for solute transport computation can not include density dependent flow.

To deal with seawater intrusion exactly, a model including change of water density is required.

Table 12.2.3-1 Hydrogeologic Description and Model Units in the El Qaa Plain

Geologic Age		Stratigraphic Formation	Explanation	Hydrogeologic Units	Model Units
Quaternary	Holocene	Sabkha Deposits	Distributed in the limited area at northwest of El Tur. Not exploitable.		-
		Wadi Deposits	Distributed in some wadi valleys. Mainly sand with small gravel. No aquifer in general but the area around El Tur ("1st aquifer").	Unconfined Aquifer	Unconfined (Sub) Aquifer
	Pleistocene	Gravel Deposits Upper	Distributed in the whole El Qaa. Coarse gravel with sand. An aquifer occurs at the shallow in the limited area around El Tur ("1st aquifer"). Generally high electric resistivity value (>100ohm-m) indicates no water.	Unconfined Aquifer	
		Middle	Coarse to medium sand and gravel, with interbeded clay. "2nd aquifer" occurs. Most of production wells extract groundwater from the aquifer. No distribution in southern El Qaa, where basement likely occurs at the shallow depth. Showing medium electric resistivity value (10<50 ohm-m).	Unconfined/ Semi-confined Aquifer	Unconfined (Main) Aquifer
		Lower	Sand and gravel with clay and silt. "3rd aquifer" occurs. Less productivity than 2nd aquifer. Showing low electric resistivity value (<10 ohm-m) in general. Probably low quality of water.	Confined Aquifer/ Aquitard	Aquitard Confined Aquifer
Tertiary-Cretaceous			Distributed in the mountainous region on the north and northwest side of El Qaa. Remnant occurs in Gebel Safariat. Mainly limestone. No drilled borehole except oil exploration holes reached the formations in the plain.	Hydrogeologic Basement	Basement
Precambrian			Metamorphic rocks, Igneous rocks. Distributed in the mountainous region on the eastern side of El Qaa and in the southern area of the plain in places. Results of geophysical prospecting indicate the basement rocks appear at the shallow depth in the south El Qaa.		

Table 12.2.4-1 Elevation of Ground Surface for Each Cell ASLm

	17	33	200	160	240	280	320	300	260	220	195	185	160	158	153	165	180	200	220	270	320	440	420	450
	20	40	140	400	360	270	380	360	240	205	185	170	150	145	147	158	175	200	240	300	330	400	420	460
	40	50	160	170	220	320	400	300	220	190	170	162	150	139	140	150	160	175	200	280	360	460	460	350
i=	60	80	100	140	220	280	340	320	220	185	168	150	140	130	135	140	150	160	200	300	360	400	320	300
5	75	75	120	160	260	230	310	250	240	175	160	140	133	125	130	135	140	157	175	240	260	260	320	400
	90	78	140	170	240	210	300	255	240	165	147	133	125	119	123	125	135	147	158	260	340	350	420	480
	100	90	170	190	220	195	260	240	240	160	140	127	97	115	115	120	128	135	143	200	380	250	360	480
	130	100	160	200	200	195	300	280	220	150	135	120	110	108	110	115	118	123	137	200	200	240	340	460
	150	100	140	180	170	240	330	290	220	145	127	115	106	104	105	108	110	115	125	140	300	240	260	440
10	90	90	130	140	180	240	360	340	220	142	124	110	104	103	103	102	104	110	117	130	300	400	300	420
	75	80	120	160	190	220	400	380	220	145	123	108	100	99	99	99	103	105	113	130	230	350	400	470
	70	90	140	160	200	240	400	380	220	145	125	110	98	96	98	98	105	113	120	135	200	220	300	420
	140	120	120	140	100	240	400	370	200	145	127	110	95	93	93	98	108	115	126	145	165	185	300	350
	200	320	180	130	180	290	450	380	210	144	120	108	94	86	94	98	108	116	130	145	163	190	240	300
15	400	400	230	140	180	280	400	400	200	140	120	104	90	84	90	97	105	115	130	148	162	185	210	250
	500	450	260	170	180	240	400	380	190	145	120	100	90	82	86	90	100	115	130	145	165	190	220	260
	500	450	280	215	180	220	350	370	180	140	116	100	90	81	83	87	95	110	125	147	175	205	238	300
	450	450	280	235	205	240	300	360	175	140	120	100	88	80	80	82	90	108	125	150	175	208	243	300
	400	350	280	250	220	260	300	340	180	135	115	100	88	79	75	76	85	104	123	145	173	204	240	280
20	500	420	300	265	240	240	300	340	190	140	115	100	85	78	72	70	80	95	112	133	165	193	228	265
	350	400	315	280	250	260	280	340	200	140	117	95	83	72	70	69	75	85	100	120	144	170	200	260
	450	420	320	285	255	280	300	320	200	138	110	97	78	69	68	69	69	75	88	107	127	145	170	240
	530	400	310	275	250	280	320	320	190	132	108	92	75	69	68	68	69	74	84	98	115	143	180	225
	540	420	270	250	240	300	320	300	180	135	104	96	72	68	68	70	73	80	87	95	117	145	180	220
25	530	400	260	230	230	280	360	300	160	120	97	81	72	67	70	75	79	87	95	104	120	145	175	210
	480	380	240	200	220	270	340	280	150	112	93	80	68	66	73	77	85	96	103	114	130	150	175	205
	500	400	230	180	200	260	360	250	140	110	88	73	66	74	76	78	83	97	112	127	142	160	186	217
	420	380	220	170	180	240	340	240	125	100	80	70	65	75	77	79	90	105	120	137	153	168	195	220
	420	320	200	150	160	230	320	210	107	90	78	65	67	70	74	78	90	105	120	138	157	165	193	230
30	420	340	190	140	150	210	300	200	100	78	70	60	65	67	73	75	93	100	118	138	160	220	240	230
	360	300	160	130	130	200	280	160	100	75	60	59	60	85	67	75	83	100	115	140	180	190	230	240
	320	280	140	120	115	180	240	130	90	70	58	56	59	60	65	70	80	100	118	150	165	180	240	260
	100	240	140	100	120	170	240	115	83	62	57	54	57	59	65	75	80	97	118	140	163	180	210	230
	40	240	140	90	160	160	220	110	78	58	54	57	58	60	67	77	85	98	118	137	158	175	197	220
35	0	240	100	70	160	150	200	110	73	55	53	57	60	67	75	82	95	103	120	135	150	168	183	205
	0	240	90	55	160	140	200	95	65	52	55	58	60	65	75	83	93	104	122	136	152	168	183	205
	0	240	40	50	150	130	180	87	60	50	55	58	59	65	75	80	93	104	123	138	160	178	200	220
	0	100	30	50	150	130	170	80	55	60	50	57	60	63	70	80	92	104	123	140	158	180	210	240
	0	0	20	100	140	120	160	75	50	50	45	53	58	60	67	77	90	103	120	138	155	175	200	230
40	0	0	10	200	140	100	140	70	47	50	44	48	55	57	65	75	88	100	117	130	147	165	180	205
	0	0	0	200	180	110	120	63	43	40	41	45	48	53	60	70	82	97	112	123	140	155	175	200
	0	0	0	100	240	100	80	65	50	43	37	40	45	50	58	65	77	90	105	117	135	155	180	260
	0	0	0	30	100	100	80	60	53	45	37	37	43	47	55	60	72	83	100	117	140	165	200	260
	0	0	0	0	180	140	60	55	48	40	35	37	40	44	53	58	68	80	100	120	145	180	220	260
45	0	0	0	0	220	100	60	38	35	35	30	35	42	48	53	58	70	87	103	125	157	185	215	260
	0	0	0	0	180	70	50	30	30	25	30	35	43	50	57	65	77	90	110	127	155	182	215	255
	0	0	0	0	180	100	35	20	25	30	30	37	45	55	65	75	85	97	110	130	157	182	218	255
	0	0	0	0	20	60	20	15	30	40	38	43	48	59	68	83	95	106	120	137	160	184	218	255
	0	0	0	0	3	5	10	27	35	43	45	52	58	67	78	90	100	113	130	147	165	190	215	250
50	0	0	0	3	3	3	23	30	40	45	55	58	64	72	82	97	108	120	137	155	176	199	220	260
	0	0	0	3	3	5	25	37	43	50	57	64	70	79	88	100	115	130	148	164	185	210	240	275
	0	0	0	0	3	10	20	40	43	53	60	70	77	82	95	108	123	135	152	170	195	217	245	280
	0	0	0	0	3	15	32	40	48	57	65	75	83	90	102	113	127	140	157	175	195	214	240	275
	0	0	0	0	4	11	28	40	47	58	68	74	85	98	110	120	130	153	163	180	197	218	247	280
55	0	0	0	0	10	28	40	45	53	62	72	80	90	105	115	128	138	148	167	182	195	220	250	295
	0	0	0	5	12	30	40	50	56	65	75	87	97	110	122	133	147	160	170	185	204	220	245	290
	0	0	3	10	17	30	42	50	60	70	80	95	105	117	128	140	153	167	178	192	210	230	260	300
	0	0	5	8	20	32	44	53	52	75	87	100	113	125	135	145	160	173	190	205	220	245	265	310
	0	0	9	10	22	32	43	55	65	78	95	107	118	130	142	155	170	180	200	218	240	255	283	335
60	0	0	5	9	23	35	47	57	68	80	97	110	125	137	150	165	175	195	210	230	250	275	295	340
	0	0	0	3	22	40	50	62	73	83	98	112	130	142	158	173	190	208	225	240	265	285	310	360
	0	0	0	0	20	38	50	62	75	88	100	115	130	145	160	180	197	217	235	255	280	305	330	380
	0	0	0	6	13	30	48	61	77	93	102	120	133	144	160	180	198	220	240	257	285	315	335	380
	0	0	0	9	17	28	40	55	73	92	103	120	133	147	162	180	200	220	245	273	300	330	355	380
65	0	0	3	9	18	28	42	53	70	90	110	130	144	158	170	182	198	220	250	278	305	338	363	400
	0	0	0	11	20	30	40	60	73	90	110	130	150	165	175	187	200	222	255	280	313	343	380	410
					j=5					10					15					20				

Table 12.2.4-2 Elevation of Bottom of Unconfied Aquifer for Each Cell ASLm

	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
i=	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
5	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
	55	55	55	55	55	55	55	55	55	55	55	50	50	50	50	50	50	55	60	60	60	60	60	60
	45	45	45	45	45	45	45	45	45	45	45	40	40	40	40	40	40	45	50	50	50	50	50	50
10	40	40	40	40	40	40	40	40	40	40	35	30	25	20	20	20	30	35	40	40	40	40	40	40
	30	30	30	30	30	30	30	30	30	30	20	15	10	10	10	15	15	25	30	30	30	30	30	30
	15	15	15	15	15	15	15	15	15	15	10	5	0	-5	-5	5	10	15	20	30	30	30	30	30
	5	5	5	5	5	5	5	5	5	5	5	0	-10	-10	-10	-10	0	10	20	30	40	40	40	40
	0	0	0	0	0	0	0	0	0	0	-5	-10	-15	-20	-20	-20	-10	0	10	25	35	40	50	50
15	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-15	-20	-30	-30	-35	-30	-10	5	15	25	30	40	40
	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-20	-30	-30	-40	-43	-43	-40	-20	0	10	15	25	30	30
	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-35	-40	-45	-45	-47	-45	-43	-30	-20	0	10	15	20	20
	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-45	-45	-50	-53	-53	-50	-45	-40	-30	-20	-10	0	10	10
	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-55	-55	-57	-57	-55	-50	-45	-40	-30	-25	-10	0	0
20	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-57	-53	-47	-45	-40	-30	-25	-20	-20
	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-60	-57	-53	-50	-45	-42	-35	-30	-25	-25
	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-65	-65	-63	-60	-55	-50	-45	-43	-42	-40	-30	-25	-25
	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-65	-65	-63	-60	-55	-45	-35	-30	-30	-30	-25	-25	-25
	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-63	-63	-63	-60	-55	-45	-40	-30	-20	-20	-20	-20	-20	-20
25	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-60	-60	-60	-50	-45	-43	-40	-25	-10	-5	0	0	0	0
	-35	-35	-35	-35	-35	-35	-35	-35	-35	-37	-40	-50	-40	-40	-40	-40	-30	-20	0	10	20	20	20	20
	-35	-35	-35	-35	-35	-35	-35	-35	-35	-35	-37	-37	-37	-35	-33	-30	-20	-10	0	20	30	35	40	40
	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-30	-27	-23	-15	-5	10	30	40	50	50	50
	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-27	-23	-20	-10	0	20	35	45	50	50	50
30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-27	-23	-20	-10	5	20	40	50	50	50	50
	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33	-30	-25	-22	-10	10	25	40	50	50	50	50
	-35	-35	-35	-35	-35	-35	-35	-35	-35	-35	-37	-37	-37	-35	-27	-20	-5	20	40	45	45	50	50	50
	-37	-37	-37	-37	-37	-37	-37	-37	-37	-37	-40	-40	-40	-35	-30	-10	15	30	40	45	45	50	50	50
	-37	-37	-37	-37	-37	-37	-37	-37	-37	-40	-45	-45	-45	-30	-20	0	20	30	40	45	45	45	45	50
35	-40	-40	-40	-40	-40	-40	-40	-40	-40	-45	-50	-50	-50	-40	-20	15	25	30	40	40	40	40	40	40
	-45	-45	-45	-45	-45	-45	-45	-45	-47	-50	-55	-55	-50	-40	-20	10	20	20	25	25	25	30	30	30
	-50	-50	-50	-50	-50	-50	-50	-50	-53	-55	-60	-60	-60	-50	-40	-30	-10	0	0	0	10	20	20	20
	-55	-55	-55	-55	-55	-55	-55	-55	-57	-60	-61	-61	-61	-60	-50	-50	-45	-40	-30	-20	-20	-20	0	0
	-60	-60	-60	-60	-60	-60	-60	-60	-60	-61	-63	-63	-63	-63	-63	-55	-55	-60	-60	-50	-45	-30	-30	-30
40	-63	-63	-63	-63	-63	-63	-63	-63	-63	-63	-65	-65	-65	-65	-65	-65	-65	-65	-63	-50	-50	-45	-40	-40
	-65	-65	-65	-65	-65	-65	-65	-65	-65	-65	-66	-66	-67	-67	-70	-70	-70	-70	-65	-60	-50	-45	-40	-40
	-66	-66	-66	-66	-66	-66	-66	-66	-66	-66	-66	-67	-68	-70	-75	-75	-75	-75	-70	-60	-50	-40	-30	-30
	-67	-67	-67	-67	-67	-67	-67	-67	-67	-67	-67	-68	-68	-70	-75	-80	-80	-80	-80	-60	-45	-30	-20	-20
	-67	-67	-67	-67	-67	-67	-67	-67	-67	-67	-67	-67	-68	-70	-75	-80	-90	-85	-80	-55	-40	-20	-10	-10
45	-68	-68	-68	-68	-68	-68	-68	-68	-68	-68	-66	-66	-66	-65	-70	-75	-85	-90	-80	-50	-30	-15	0	0
	-75	-75	-75	-75	-75	-75	-75	-75	-70	-68	-66	-65	-65	-60	-60	-70	-85	-90	-80	-50	-20	-10	10	10
	-80	-80	-80	-80	-80	-80	-80	-77	-75	-70	-67	-63	-60	-55	-55	-65	-80	-85	-80	-40	-15	0	20	20
	-90	-90	-90	-90	-90	-90	-85	-80	-80	-75	-70	-63	-55	-53	-50	-60	-70	-80	-70	-40	-10	10	20	20
	-100	-100	-100	-100	-100	-95	-90	-85	-80	-75	-65	-60	-55	-45	-40	-50	-60	-80	-70	-20	-10	10	30	30
50	-100	-100	-100	-100	-100	-95	-90	-85	-80	-75	-65	-60	-50	-40	-35	-45	-55	-60	-50	-15	0	20	35	35
	-100	-100	-100	-100	-100	-95	-90	-85	-80	-75	-65	-55	-45	-35	-30	-40	-45	-50	-30	-10	5	25	40	40
	-100	-100	-100	-100	-100	-95	-90	-85	-80	-70	-60	-50	-40	-25	-25	-30	-40	-40	-20	-5	10	30	40	40
	-100	-100	-100	-100	-100	-95	-90	-85	-75	-65	-60	-45	-30	-20	-20	-25	-30	-30	-5	0	10	30	45	45
	-95	-95	-95	-95	-95	-90	-85	-80	-70	-60	-50	-40	-25	-15	-15	-20	-25	-20	-10	0	20	35	45	45
55	-95	-95	-95	-95	-95	-90	-85	-75	-70	-65	-45	-30	-20	-10	-5	-10	-15	-15	-5	10	20	40	50	50
	-95	-95	-95	-95	-90	-85	-80	-75	-65	-50	-40	-25	-10	0	5	0	-5	-10	-5	10	25	40	60	60
	-95	-95	-95	-90	-87	-83	-75	-70	-60	-50	-35	-20	-5	5	7	5	5	0	0	20	30	50	60	60
	-90	-90	-90	-87	-85	-80	-75	-65	-60	-45	-30	-10	0	10	15	10	7	5	10	25	40	55	65	65
	-90	-90	-90	-85	-80	-75	-70	-65	-50	-40	-30	-10	10	17	17	17	15	15	20	30	50	60	70	70
60	-85	-85	-85	-83	-77	-73	-65	-60	-45	-30	-20	0	15	22	25	25	22	20	30	40	55	70	80	80
	-80	-80	-80	-80	-75	-70	-65	-50	-40	-25	-10	10	20	25	30	30	35	30	40	50	65	80	85	95
	-75	-75	-75	-75	-70	-65	-60	-45	-30	-20	0	15	25	30	35	40	45	45	55	65	75	85	90	90
	-70	-70	-70	-70	-65	-63	-50	-40	-25	-10	5	20	25	35	40	45	55	60	65	75	85	90	100	100
	-70	-70	-70	-67	-63	-55	-45	-30	-20	-5	10	25	30	35	45	55	60	70	75	85	90	100	110	110
65	-70	-70	-70	-65	-60	-50	-40	-30	-10	0	20	25	35	40	50	60	70	75	80	90	100	110	120	120
	-65	-65	-65	-55	-45	-30	-20	-10	10	20	30	35	45	55	65	75	80	90	100	110	120	130	130	130
					j=5					10					15					20				

Table 12.2.4-3 Elevation of Bottom of the Aquitard for Each Cell ASLm

	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15
	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15
i=	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
5	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
	3	3	3	3	3	3	3	3	3	3	3	0	0	0	0	0	0	3	5	5	5	5	5	5
	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-8	-10	-10	-10	-10	-10	-10	-8	-5	-5	-5	-5	-5	-5
10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-13	-15	-18	-20	-20	-20	-15	-13	-10	-10	-10	-10	-10	-10
	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-25	-28	-30	-30	-30	-28	-28	-23	-20	-20	-20	-20	-20	-20
	-28	-28	-28	-28	-28	-28	-28	-28	-28	-28	-30	-33	-35	-38	-38	-33	-30	-28	-25	-20	-20	-20	-20	-20
	-38	-38	-38	-38	-38	-38	-38	-38	-38	-38	-38	-40	-45	-45	-45	-45	-40	-35	-30	-25	-20	-20	-20	-20
	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-43	-45	-48	-50	-50	-50	-45	-40	-35	-28	-23	-20	-15	-15
15	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-53	-55	-60	-60	-63	-60	-50	-43	-38	-33	-25	-20	-20
	-58	-58	-58	-58	-58	-58	-58	-58	-58	-58	-60	-65	-65	-70	-72	-72	-70	-60	-50	-45	-38	-33	-30	-30
	-65	-65	-65	-65	-65	-65	-65	-65	-65	-65	-68	-70	-73	-73	-74	-73	-72	-65	-60	-50	-45	-43	-40	-40
	-75	-75	-75	-75	-75	-75	-75	-75	-75	-75	-78	-78	-80	-82	-82	-80	-78	-75	-70	-65	-60	-50	-45	-45
	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-85	-88	-88	-89	-89	-88	-85	-83	-80	-75	-73	-60	-55	-55
20	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-95	-95	-95	-94	-92	-89	-88	-85	-75	-73	-70	-70
	-97	-97	-97	-97	-97	-97	-97	-97	-97	-97	-97	-97	-97	-102	-100	-99	-97	-95	-93	-86	-83	-80	-78	-78
	-97	-97	-97	-97	-97	-97	-97	-97	-97	-97	-101	-102	-103	-102	-100	-98	-95	-93	-92	-91	-90	-85	-83	-83
	-102	-102	-102	-102	-102	-102	-102	-102	-102	-102	-102	-102	-108	-107	-105	-103	-98	-93	-90	-90	-85	-83	-93	-83
	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-107	-108	-107	-105	-103	-98	-95	-90	-85	-85	-85	-85	-85	-85
25	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-105	-110	-110	-105	-103	-102	-100	-93	-85	-83	-80	-80	-80	-80
	-98	-98	-98	-98	-98	-98	-98	-98	-98	-99	-100	-110	-105	-105	-105	-105	-100	-95	-80	-70	-60	-60	-60	-60
	-103	-103	-103	-103	-103	-103	-103	-103	-103	-103	-104	-108	-109	-108	-107	-105	-100	-90	-80	-60	-50	-45	-40	-40
	-107	-107	-107	-107	-107	-107	-107	-107	-107	-107	-107	-108	-112	-110	-107	-103	-95	-85	-70	-50	-40	-30	-30	-30
	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-107	-103	-100	-90	-80	-60	-45	-35	-30	-30	-30
30	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-107	-103	-100	-90	-75	-60	-40	-30	-30	-30	-30
	-113	-113	-113	-113	-113	-113	-113	-113	-113	-113	-113	-113	-113	-110	-105	-102	-90	-70	-55	-40	-30	-30	-30	-30
	-115	-115	-115	-115	-115	-115	-115	-115	-115	-115	-117	-117	-117	-115	-107	-100	-75	-60	-40	-35	-35	-30	-30	-30
	-117	-117	-117	-117	-117	-117	-117	-117	-117	-117	-120	-120	-120	-115	-110	-90	-85	-50	-40	-35	-35	-30	-30	-30
	-117	-117	-117	-117	-117	-117	-117	-117	-117	-120	-125	-125	-125	-110	-100	-80	-60	-50	-40	-35	-35	-35	-35	-30
35	-120	-120	-120	-120	-120	-120	-120	-120	-120	-125	-130	-130	-130	-120	-100	-65	-55	-50	-40	-40	-40	-40	-40	-40
	-125	-125	-125	-125	-125	-125	-125	-125	-127	-130	-135	-135	-130	-120	-100	-70	-60	-60	-55	-55	-55	-50	-50	-50
	-130	-130	-130	-130	-130	-130	-130	-130	-131	-135	-140	-140	-140	-130	-120	-110	-90	-80	-80	-80	-70	-60	-60	-60
	-135	-135	-135	-135	-135	-135	-135	-135	-137	-140	-141	-141	-141	-140	-130	-130	-125	-120	-110	-100	-100	-100	-80	-80
	-140	-140	-140	-140	-140	-140	-140	-140	-140	-141	-143	-143	-143	-143	-143	-135	-135	-140	-140	-130	-123	-110	-110	###
40	-142	-142	-142	-142	-142	-142	-142	-142	-142	-143	-145	-145	-145	-145	-145	-145	-145	-145	-143	-140	-130	-125	-120	###
	-143	-143	-143	-143	-143	-143	-143	-143	-143	-145	-146	-146	-147	-147	-150	-150	-150	-150	-145	-140	-130	-125	-120	###
	-143	-143	-143	-143	-143	-143	-143	-143	-143	-146	-146	-147	-148	-150	-155	-155	-155	-155	-150	-140	-130	-120	-110	###
	-144	-144	-144	-144	-144	-144	-144	-144	-144	-147	-147	-148	-148	-150	-155	-160	-160	-160	-150	-140	-125	-110	-100	###
	-144	-144	-144	-144	-144	-144	-144	-144	-144	-147	-147	-147	-148	-150	-155	-160	-170	-165	-160	-135	-120	-100	-90	-90
45	-139	-139	-139	-139	-139	-139	-139	-139	-144	-148	-146	-146	-146	-145	-150	-155	-165	-170	-160	-130	-110	-95	-80	-80
	-138	-138	-138	-138	-138	-138	-138	-143	-140	-144	-146	-145	-145	-140	-140	-150	-163	-165	-155	-130	-100	-90	-70	-70
	-140	-140	-140	-140	-140	-140	-140	-139	-138	-140	-144	-141	-140	-135	-135	-145	-155	-156	-155	-120	-95	-80	-60	-60
	-145	-145	-145	-145	-145	-145	-143	-140	-140	-138	-140	-136	-133	-133	-130	-140	-150	-155	-145	-120	-90	-70	-60	-60
	-135	-140	-140	-140	-140	-143	-140	-138	-140	-138	-133	-135	-133	-125	-122	-130	-140	-150	-140	-100	-90	-70	-50	-50
50	-135	-135	-140	-140	-140	-138	-140	-138	-135	-138	-133	-130	-125	-120	-115	-125	-133	-135	-125	-95	-80	-60	-45	-45
	-135	-135	-135	-140	-140	-138	-135	-133	-135	-133	-128	-127	-122	-115	-110	-120	-123	-125	-110	-90	-75	-55	-40	-40
	-135	-135	-135	-135	-135	-138	-135	-133	-130	-130	-125	-120	-115	-105	-105	-110	-120	-120	-100	-85	-70	-50	-40	-40
	-130	-135	-135	-135	-135	-133	-130	-130	-128	-123	-120	-117	-110	-100	-100	-105	-110	-110	-85	-80	-70	-50	-35	-35
	-128	-128	-133	-133	-133	-130	-128	-125	-125	-120	-115	-110	-100	-95	-95	-100	-105	-100	-90	-80	-60	-45	-35	-35
55	-128	-128	-128	-128	-133	-130	-128	-123	-120	-118	-113	-105	-100	-90	-85	-90	-95	-95	-85	-70	-60	-40	-30	-30
	-128	-128	-128	-128	-125	-128	-125	-123	-118	-110	-105	-98	-90	-80	-75	-80	-85	-90	-85	-70	-55	-40	-20	-20
	-128	-129	-128	-125	-124	-122	-123	-120	-115	-110	-103	-95	-85	-75	-73	-75	-75	-80	-80	-60	-50	-30	-20	-20
	-120	-125	-125	-124	-123	-120	-118	-113	-110	-103	-95	-85	-80	-70	-65	-70	-73	-75	-70	-55	-40	-25	-15	-15
	-120	-120	-120	-118	-120	-118	-115	-113	-105	-100	-95	-85	-70	-63	-63	-63	-65	-65	-60	-50	-30	-20	-10	-10
60	-118	-118	-118	-117	-114	-112	-108	-110	-103	-95	-90	-80	-65	-58	-55	-55	-58	-60	-50	-40	-25	-10	0	0
	-115	-115	-115	-115	-113	-110	-108	-100	-95	-88	-80	-70	-60	-55	-50	-50	-45	-50	-40	-30	-15	0	5	5
	-113	-113	-113	-113	-110	-108	-105	-98	-90	-85	-75	-65	-55	-50	-45	-40	-35	-35	-25	-15	-5	5	10	10
	-110	-110	-110	-110	-108	-107	-100	-95	-88	-80	-73	-60	-55	-45	-40	-35	-25	-20	-15	-5	5	10	20	20
	-110	-110	-110	-109	-107	-103	-98	-90	-85	-78	-70	-55	-50	-45	-35	-25	-20	-10	-5	5	10	20	30	30
65	-110	-110	-110	-108	-105	-100	-95	-90	-80	-75	-60	-55	-45	-40	-30	-20	-10	-5	0	10	20	30	40	40
	-108	-108	-106	-103	-98	-90	-85	-80	-70	-60	-50	-45	-35	-25	-15	-5	0	10	20	30	40	50	50	50
					j=5					10					15					20				

Table 12.2.4-4 Elevation of Bottom of Confied Aquifer for Each Cell ASLm

	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20	-20
	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30
i=	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30	-30
5	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50
	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50
	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50	-50
10	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60
	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60	-60
	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70
	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70	-70
	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-90	-80
15	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80	-80
	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-90	-80	-80	-80
	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-90	-90	-90	-90
	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100	-100
	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-100	-100	-100
20	-110	-110	-110	-110	-110	-110	-110	-110	-110	-110	-120	-120	-120	-120	-120	-120	-120	-120	-120	-120	-120	-110	-110	-110
	-120	-120	-120	-120	-120	-120	-120	-120	-120	-120	-120	-120	-130	-130	-130	-130	-130	-130	-130	-130	-120	-120	-120	-120
	-130	-130	-130	-130	-130	-130	-130	-130	-130	-130	-130	-130	-130	-140	-140	-140	-140	-140	-140	-130	-130	-130	-130	-130
	-130	-130	-130	-130	-130	-130	-130	-130	-130	-130	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140
	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-150	-150	-150	-150	-150	-150	-150	-150	-140	-140	-140	-140
25	-140	-140	-140	-140	-140	-140	-140	-140	-140	-140	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150
	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-150	-160	-160	-160	-160	-160	-160	-160	-160	-150	-160	-160	-160	-160
	-160	-160	-160	-160	-160	-160	-160	-160	-160	-160	-160	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170
	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170	-170	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180
	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-180	-190	-190	-190	-190	-190	-190	-190	-180	-180	-180	-180	-180
30	-190	-190	-190	-190	-190	-190	-190	-190	-190	-190	-190	-200	-200	-200	-200	-200	-200	-200	-200	-190	-190	-190	-190	-190
	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200
	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-200	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210
	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-220	-220	-220	-220	-220	-220	-220	-220	-210	-210	-210	-210
	-210	-210	-210	-210	-210	-210	-210	-210	-210	-210	-220	-220	-220	-230	-230	-230	-230	-230	-230	-220	-220	-210	-210	-210
35	-210	-210	-210	-210	-210	-210	-210	-210	-210	-220	-220	-220	-230	-230	-230	-230	-230	-230	-230	-230	-220	-220	-220	-220
	-210	-210	-210	-210	-210	-210	-210	-210	-220	-220	-220	-230	-230	-240	-240	-240	-240	-240	-240	-230	-230	-220	-220	-220
	-220	-220	-220	-220	-220	-220	-220	-220	-220	-220	-230	-230	-240	-240	-250	-250	-250	-250	-240	-240	-230	-230	-220	-220
	-220	-220	-220	-220	-220	-220	-220	-220	-220	-230	-230	-240	-240	-250	-250	-250	-250	-250	-250	-240	-230	-230	-220	-220
	-220	-220	-220	-220	-220	-220	-220	-220	-220	-230	-230	-240	-250	-250	-250	-250	-250	-250	-250	-250	-240	-230	-220	-220
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					j=5					10					15					20				

Table 12 2.4-5 Summary of Existing Study on Groundwater Discharge and Recharge in the El Qaa Plain

		Sinai Development Study[1]		Sainai Water Res. Study[2]		Master Plan[3]		Hatem(1996)[4]		Data Book 1996[5]		Data Book 1998[6]	
	Year	1982		1992		1992		1992		1995		1997	
	Unit	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year
Discharge	Domestic	-	-	7790	2.84	950	0.35	950	0.35	1013	0.37	4013	1.46
	Tourism	-	-	-	-	640	0.23	50	0.02	102	0.04	119	0.04
	Other	-	-	-	-	1000	0.37	950	0.35	1950	0.71	1740	0.64
	Sub total	-	-	7790	2.84	2590[*3]	0.95	1970	0.72	3065[*5]	1.12	5872[*5]	2.14
	Irrigation	-	-	830	0.30	15876	5.79	830	0.30	-	-	-	-
	To Sharm El Sheik	-	-	-	-	-	-	2000	0.73	-	-	-	-
	Total	2800	1.02	8520[*1]	3.14	18466	6.74	4900	1.75	-	-	-	-
Recharge	Northern Part	-	-	19800	7.23	-	-	9456	3.45	-	-	-	-
	Southern Part	-	-	11700	4.27	-	-	7793	2.84	-	-	-	-
	Total	66000	24.09	31500[*2]	11.5	-	-	17249[*4]	6.29	-	-	-	-
Remarks		Recharge was estimated from runoff analysis		Recharge was estimated by flow net method				Recharge equations were built from runoff analysis of representative chatchments					

1) Dames & Moore(1985b)
2)BRGM/RIWR (1993b)
3)Water Master Plan of Southern Sinai Governarate
4)Hatem(1996)
5)Statistical Data Book, Jan. 1996, South Sinai Governorate
6)Statistical Data Book, Jan. 1998, South Sinai Governorate
*1 Including Water to Sharm El Sheik
*2 Excluding Recycled Irrigation Water
*3 Excluding Water to Sharm El Sheik
*4 Estimated by recharge equations, from Table5-9 in Hatem(1996)
*5 Water Consumption

Table 12 2.4-6 Recharge Rate and TDS Concentration

Area	Wadi Name	Estimated Recharge* 10^0 m^3/year	Estimated Recharge* m^3/day	Calibrated Recharge 10^0 m^3/year	Calibrated Recharge m^3/day	C/E %	Initial TDS mg/l	Calibrated TDS mg/l
Limestone Area	Abu reymate	64409	176.5	64409	176.5	100.0	6000	6000
	Elrawd	99709	273.2	99709	273.2	100.0	4000	4000
	Thogra	38540	105.6	38540	105.6	100.0	4000	4000
	Umm gurdy	48801	133.7	48801	133.7	100.0	2000	2000
	Abora	158248	433.6	158248	433.6	100.0	2000	2000
	Subtotal	409707	1122.5	409707	1122.5	100.0	-	-
Basement Area I	Geba	321544	880.9	289390	792.8	90.0	800	800
	Warka	234615	642.8	211153	578.5	90.0	800	800
	Subtotal	556159	1523.7	500543	1371.4	90.0	-	-
Basement Area II	Hibran	684182	1874.5	410509	1124.7	60.0	500	500
	Mir	1177907	3227.1	823196	2255.3	69.9	400	400
	Mnhy	109588	300.2	603774	1654.2	550.9	400	400
	Wagran	133852	366.7	1288379	3529.8	962.5	400	400
	Shadk	379911	1040.9	1372160	3759.3	361.2	400	1000
	Emlaha	291544	798.8	495635	1357.9	170.0	800	1000
	Subtotal	2776984	7608.2	4993653	13681.2	179.8	-	-
Total		3742850	10254.4	5903903	16175.1	157.7	-	-

* from Hatem(1996), Table 5-9

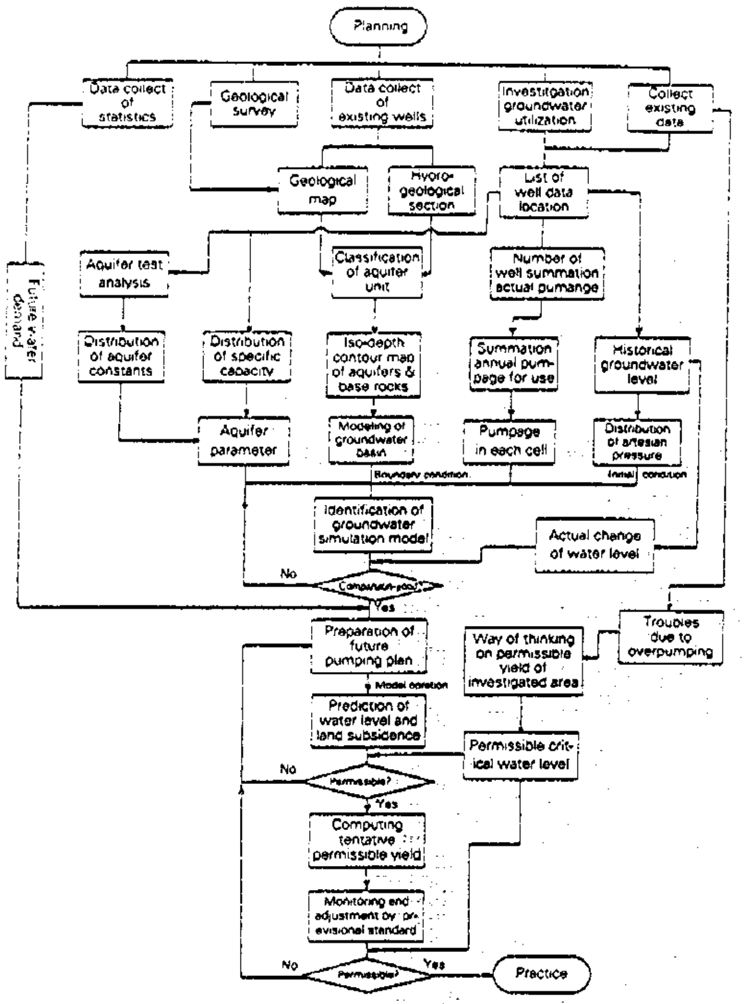
C/E: Calibrated / Estimated

Table 12 5.1-1 Water Balance in Prediction Period

Case	Input						Output						Input - Output	
	Recharge		Storage*		Total		Pumpage		Flow to Sea		Total			
	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	10^6m^3/year	m^3/day	%**
1	16175	5.90	9199	3.36	25374	9.26	20000	7.30	5755	2.10	25755	9.40	-381	-1.5
2	16175	5.90	5514	2.01	21689	7.92	15000	5.48	6399	2.34	21399	7.81	290	1.3
3	16175	5.90	3026	1.10	19201	7.01	12500	4.56	6825	2.49	19325	7.05	-124	-0.6
4	16175	5.90	984	0.36	17159	6.26	10000	3.65	7179	2.62	17179	6.27	-20	-0.1
5	16175	5.90	5728	2.09	21903	7.99	15000	5.48	6623	2.42	21623	7.89	280	1.3

*water released from storage due to drawdown

**(Input - Output) / Input



General procedure of groundwater simulation for groundwater basin management
(after Research Group for Water Balance, 1976)

Fig. 12.1.1-1 General Procedure of Groundwater Simulation

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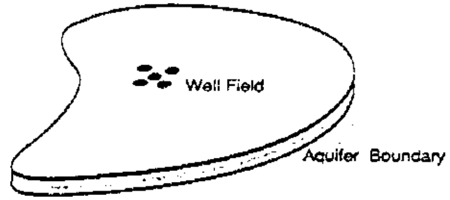
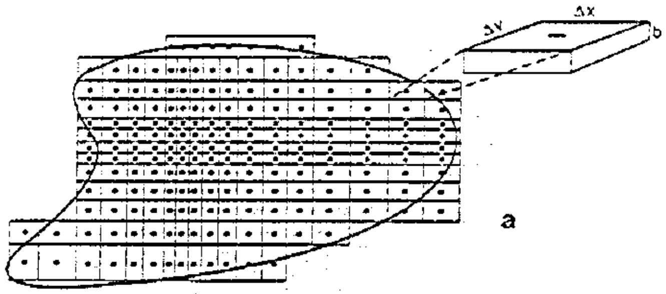
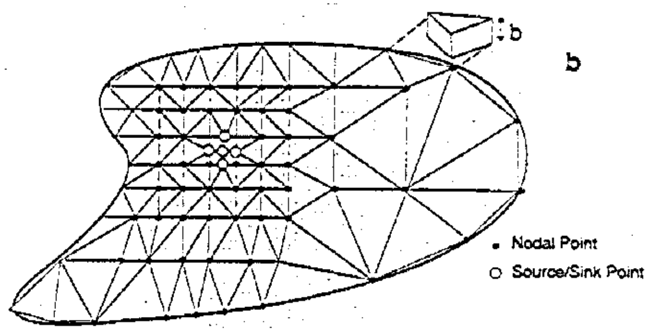


Map view of aquifer showing well field and boundaries



Finite difference grid for aquifer study, where Δx is the x-direction spacing, Δy the y-direction spacing, and b the aquifer thickness.



Finite element configuration for aquifer study.

Fig. 12.1.2-1 Finite Difference and Finite Element Grid Network (Mercer and Faust, 1981)

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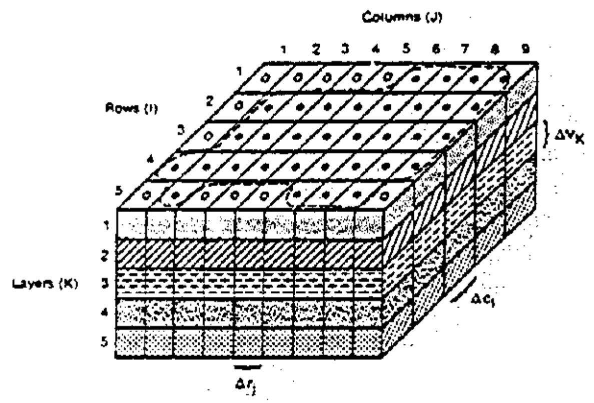

Explanation

- ---- Aquifer Boundary
- ● Active Cell
- o Inactive Cell
- Δr_j Dimension of Cell Along the Row Direction. Subscript (J) Indicates the Number of the Column
- Δc_i Dimension of Cell Along the Column Direction. Subscript (I) Indicates the Number of the Row
- Δv_K Dimension of the Cell Along the Vertical Direction, Subscript (K) Indicates the Number of the Layer

Fig. 12.1.2-2 A discretized Hypothetical Aquifer System (McDonald and Harbaugh, 1988)

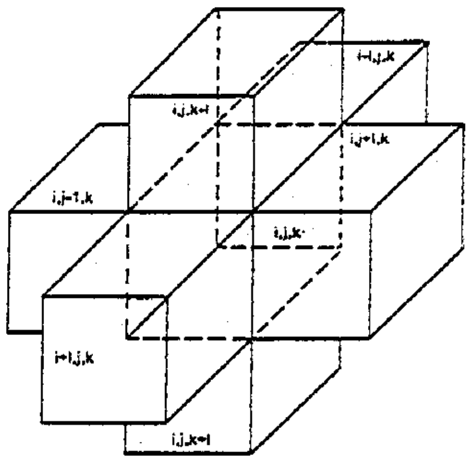

Fig. 12.1.2-3 Cell i,j,k and Indices for the Six Adjacent Cells (McDonald and Harbaugh, 1988)

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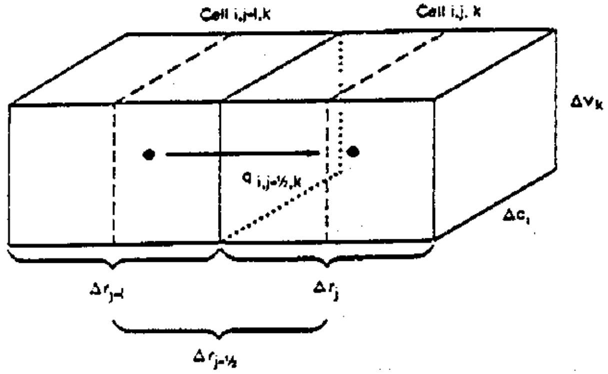

Fig. 12.1.2-4 Flow into Cell i,j,k from Cell i,j-1,k (McDonald and Harbaugh, 1988)

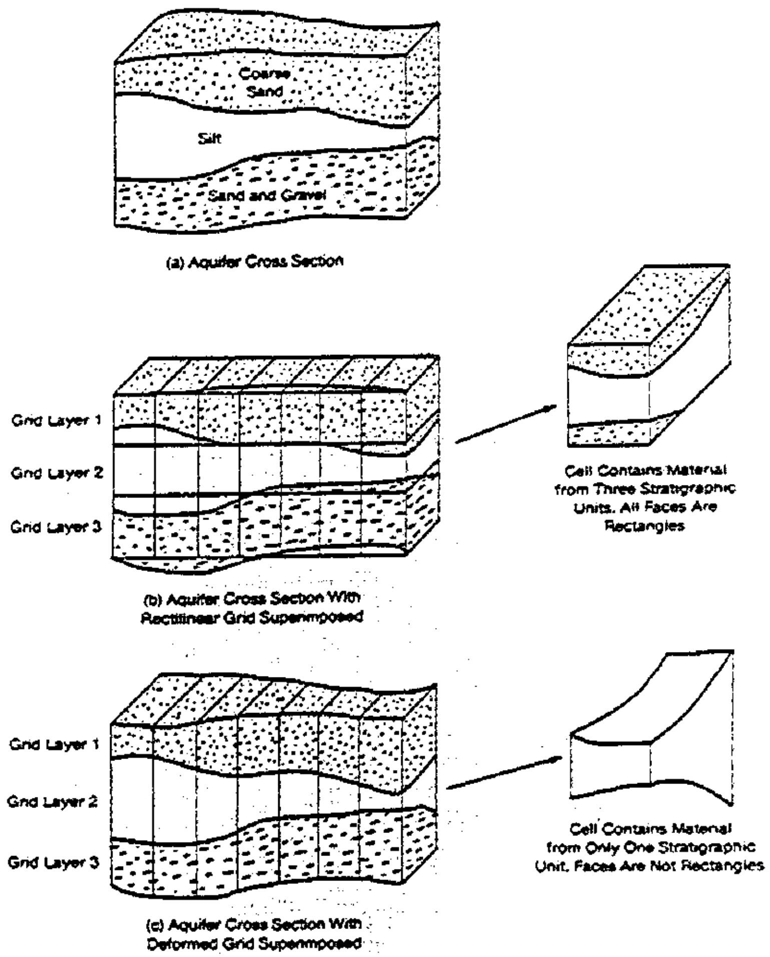

Fig. 12.1.2-5 Schemes of Vertical Discretization (McDonald and Harbaugh, 1988)

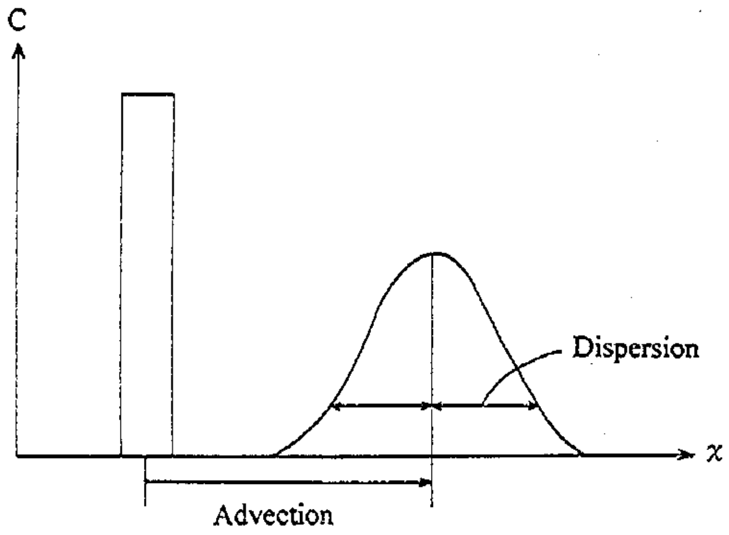

Fig. 12.1.2-6 Advection and Dispersion

MOC

$$C_m^{n^*} = \frac{1}{NP}\sum_{l=1}^{NP} C_l^n$$

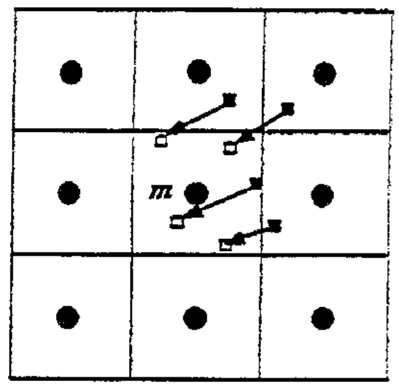

Illustration of the Method Of Characteristics (MOC). A set of moving particles are tracked forward during each time period. An intermediate concentration, equal to the average of the concentrations of all particles in the cell, is assigned to each cell. This intermediate concentration accounts for the effect of advection during the period since the preceding time level, and is used to calculate changes in concentration due to dispersion and other processes over that period.

Fig. 12.1.2-7 Illustration of the MOC (Zheng, 1990)

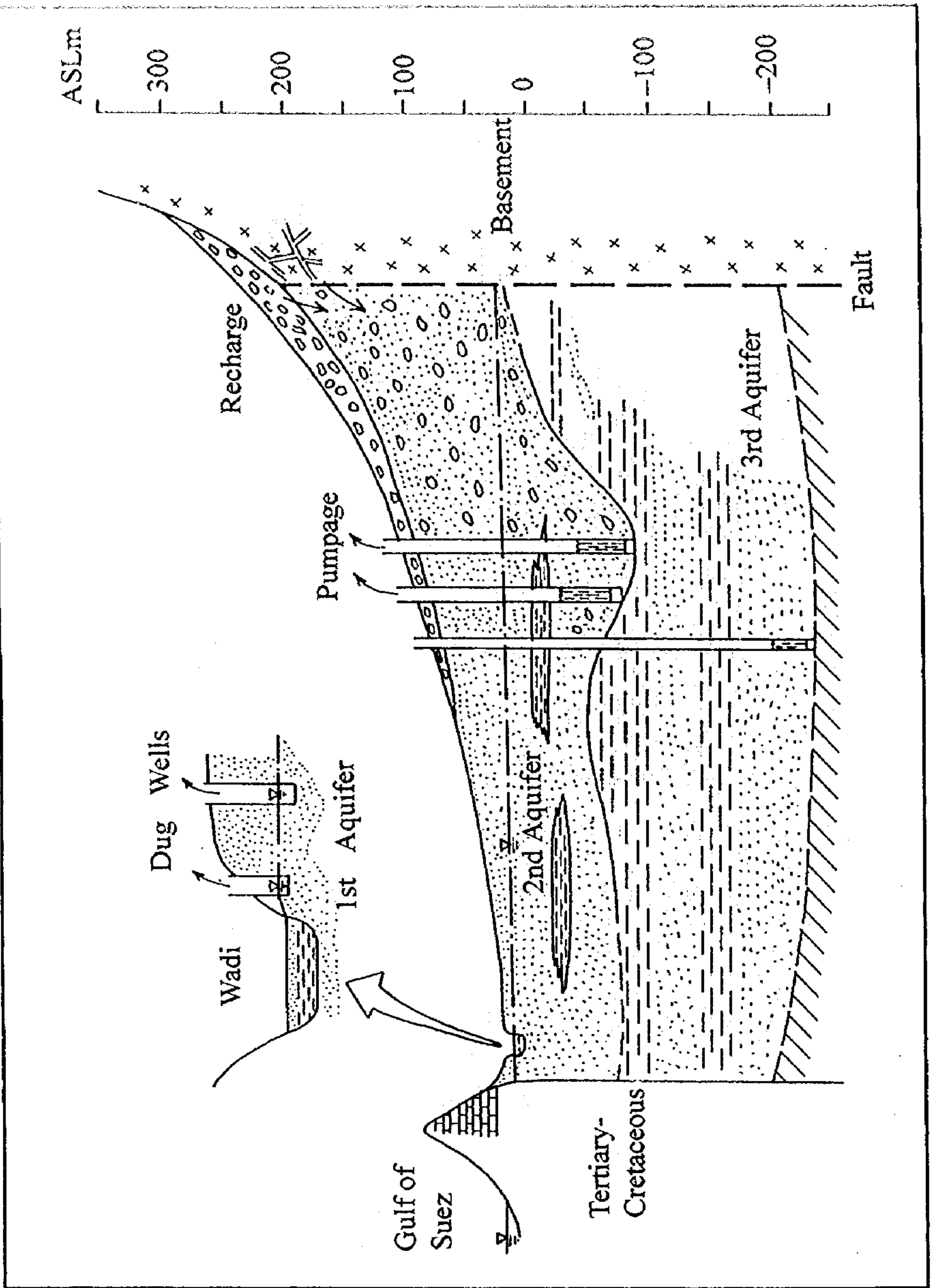

Fig. 12.2.3-1 Schematic Cross Section of the El Qaa plain

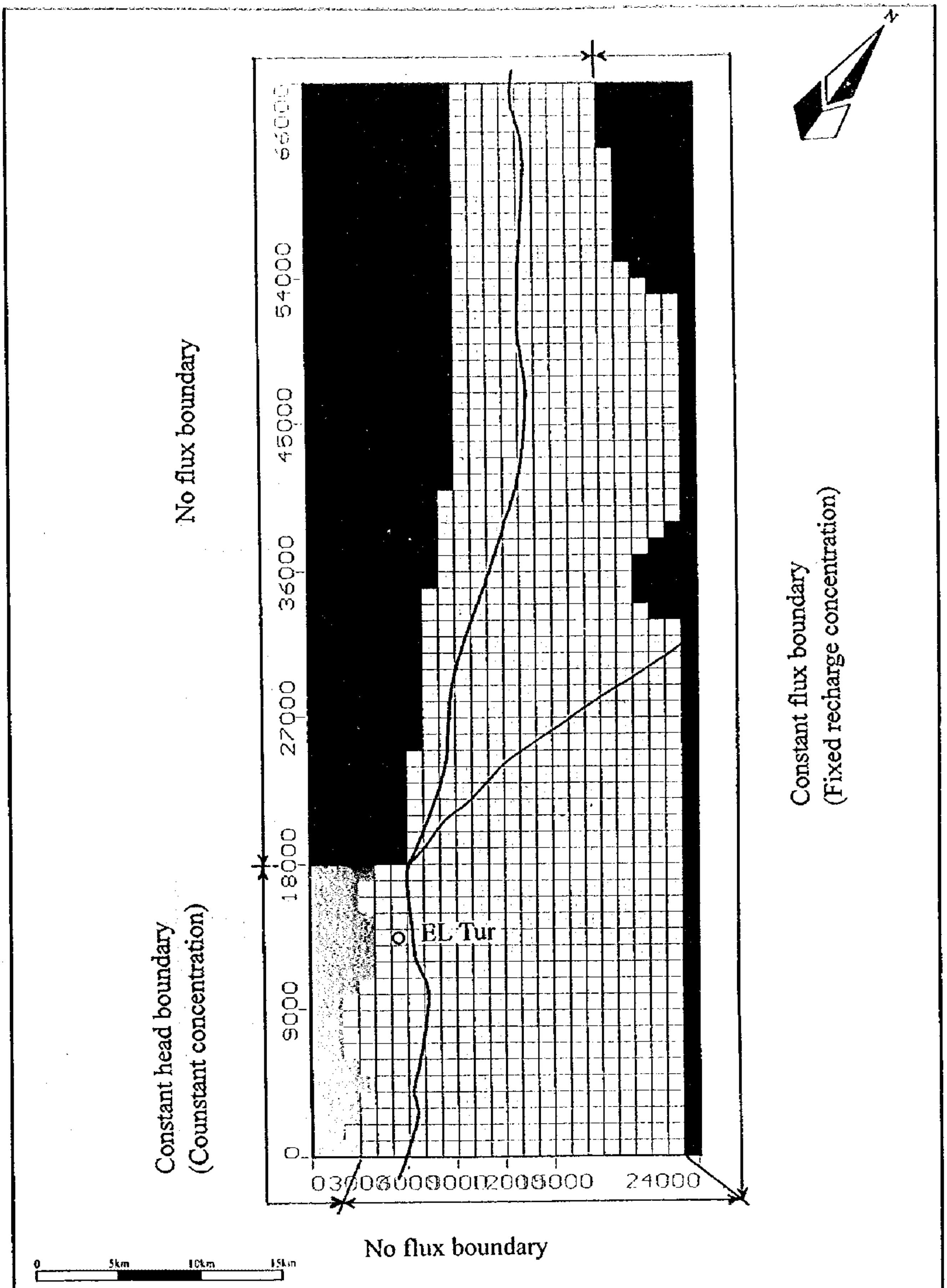

Fig. 12.2.4-1 Model Area and Boundary Conditions

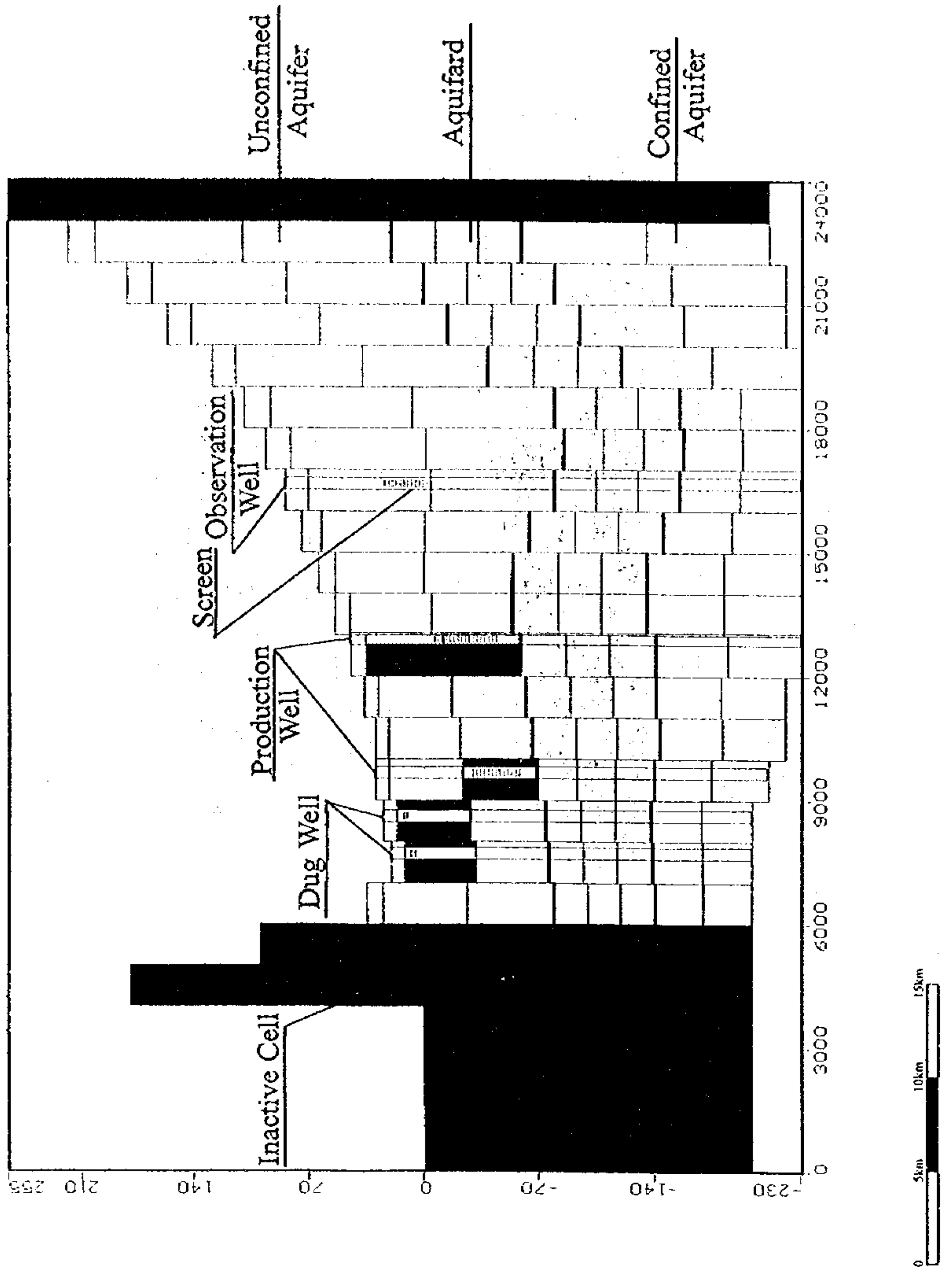

Fig. 12.2.4-2 Vertical Discretization of the Model

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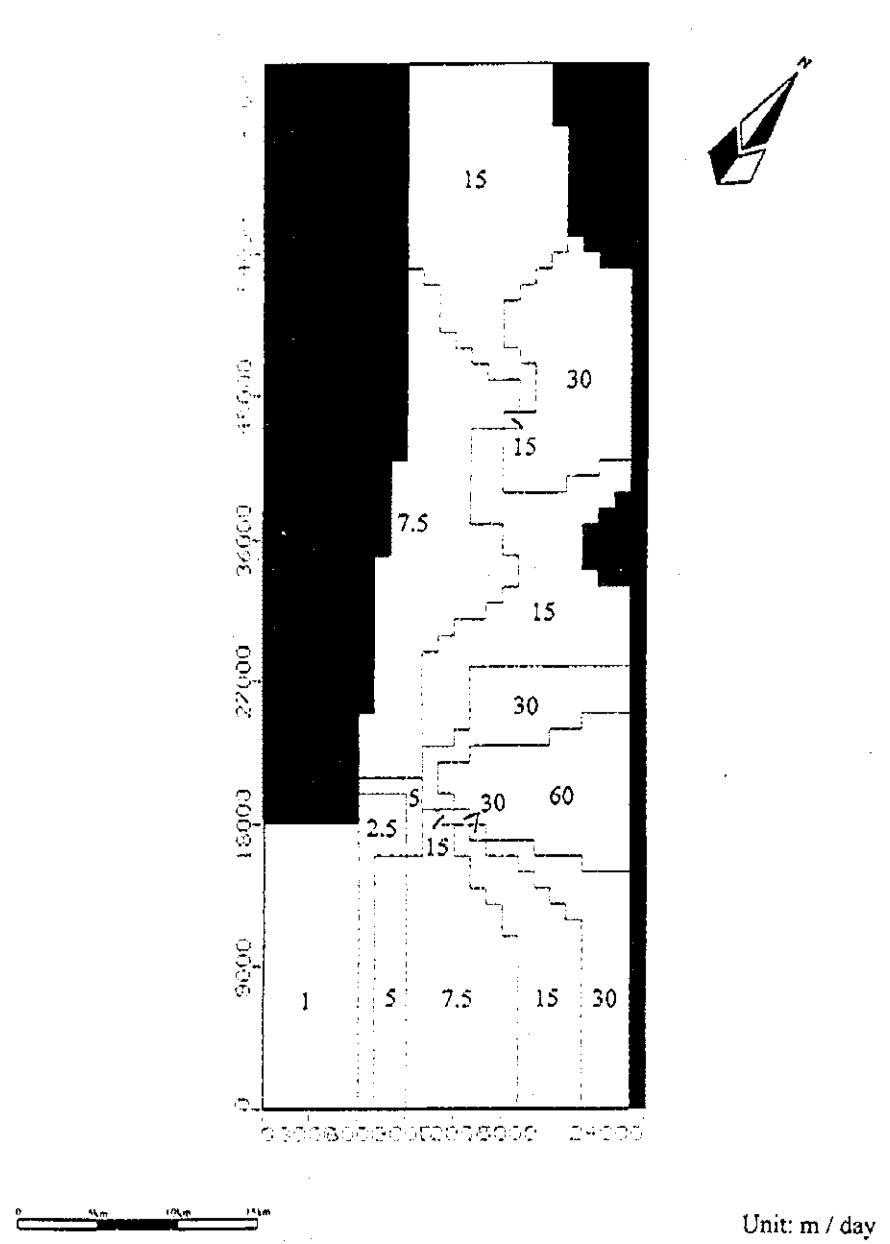

Unit: m / day

Fig. 12.2.4-3 Initial Hydraulic Conductivity of Unconfined Aquifer (Layers 1-3)

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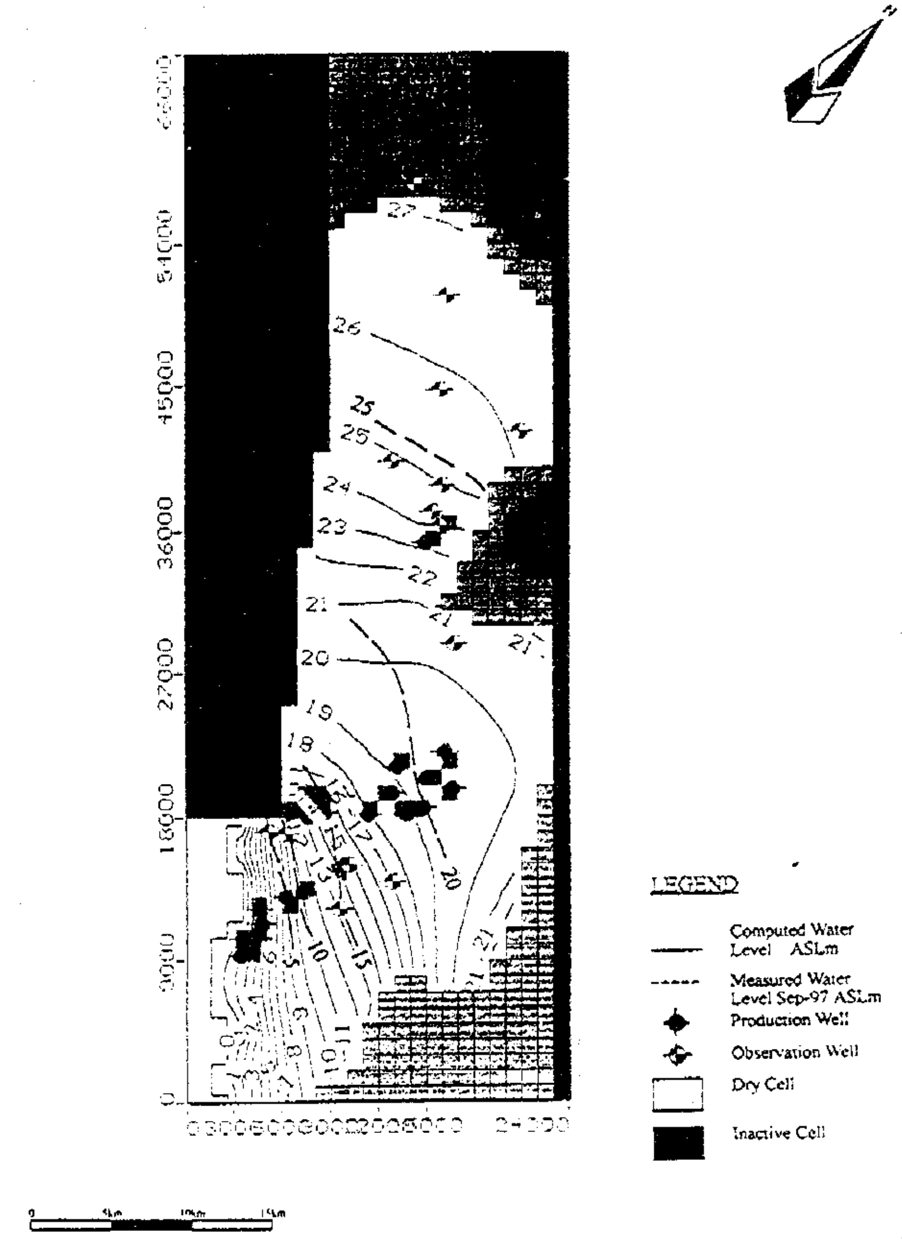

Fig. 12.3.2-1 Observed and Calculated Water Level Configuration of the Unconfined Aquifer

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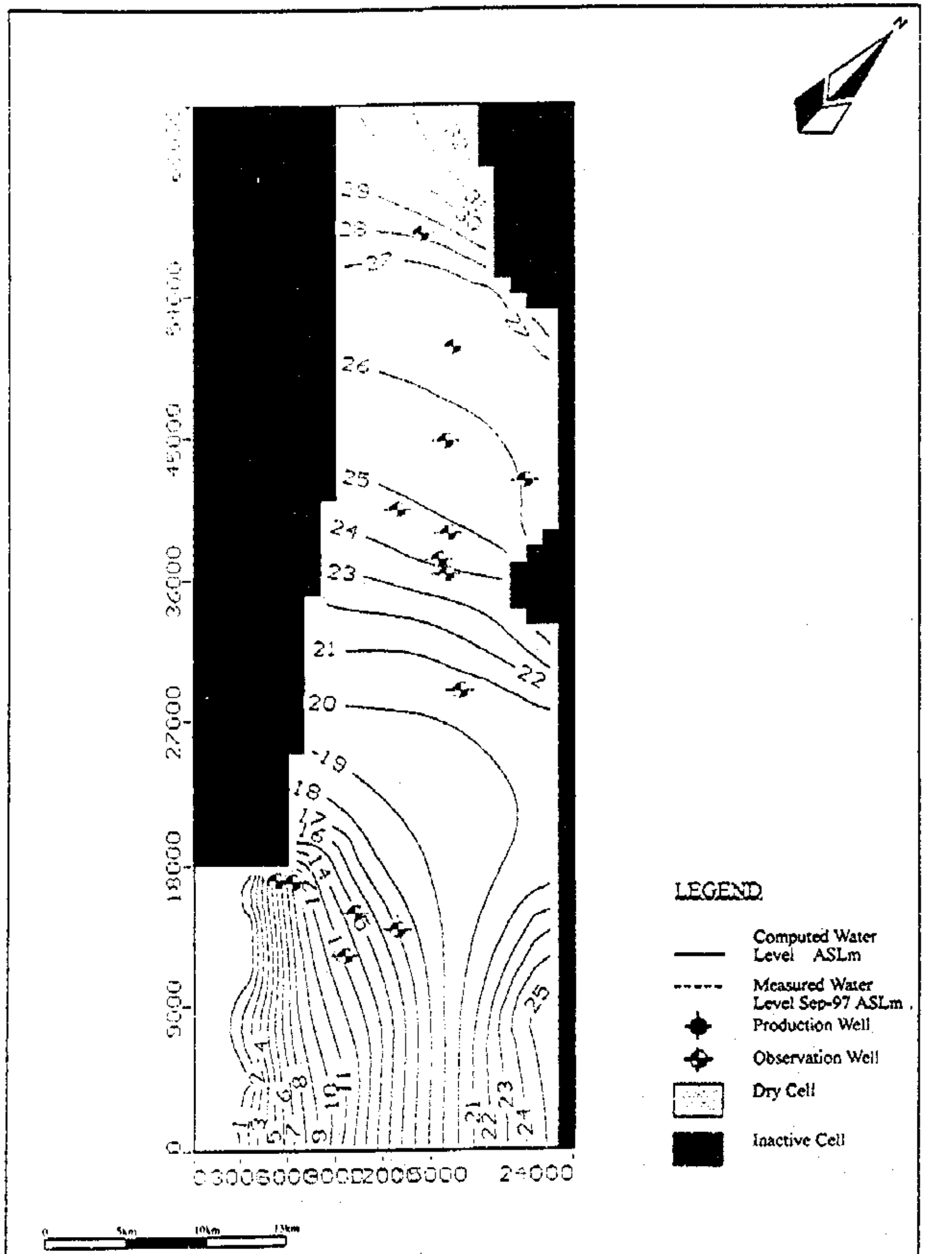

Fig. 12.3.2-2 Calculated Water Level Configuration of Confined Aquifer

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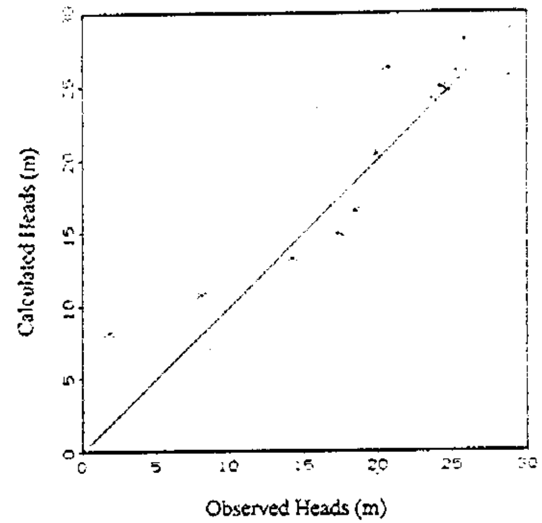

Fig. 12.3.2-3 Plot of Simulated versus Measured Water Level

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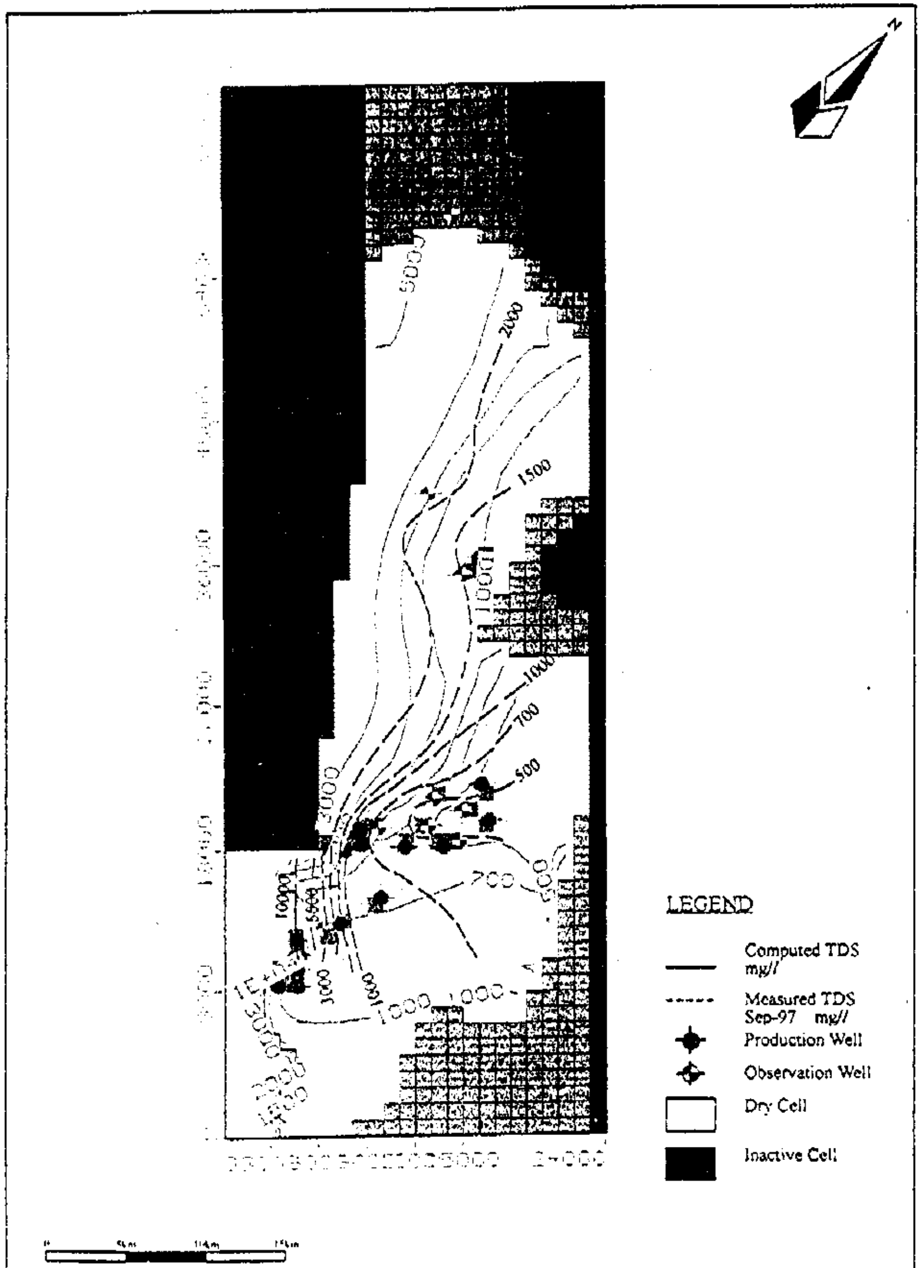

Fig. 12.3.2-4 Observed and Calculated TDS Configuration of the Unconfined Aquifer

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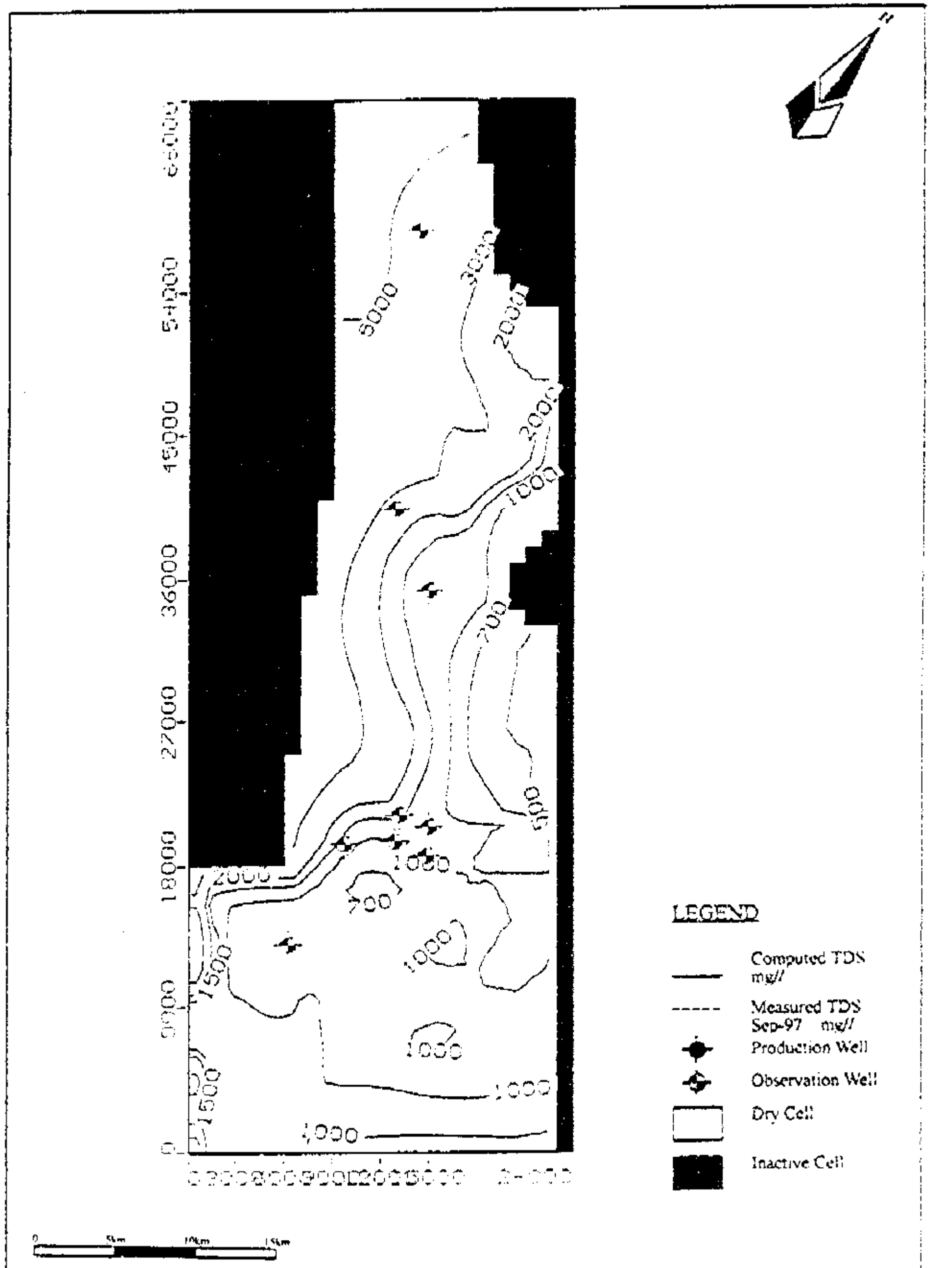

Fig. 12.3.2-5 Calculated TDS Configuration of Confined Aquifer

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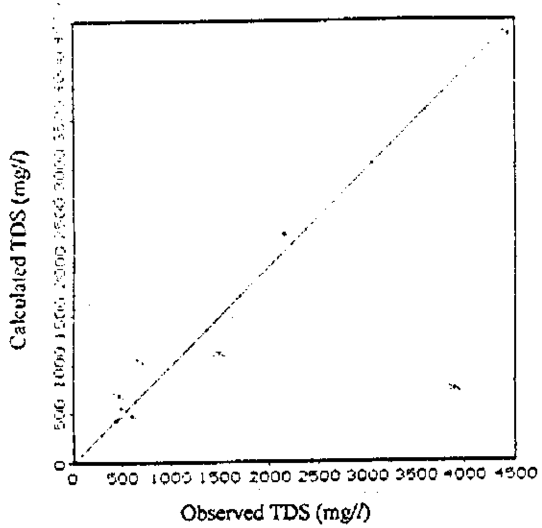

Fig. 12.3.2-6 Plot of Simulated versus Measured TDS

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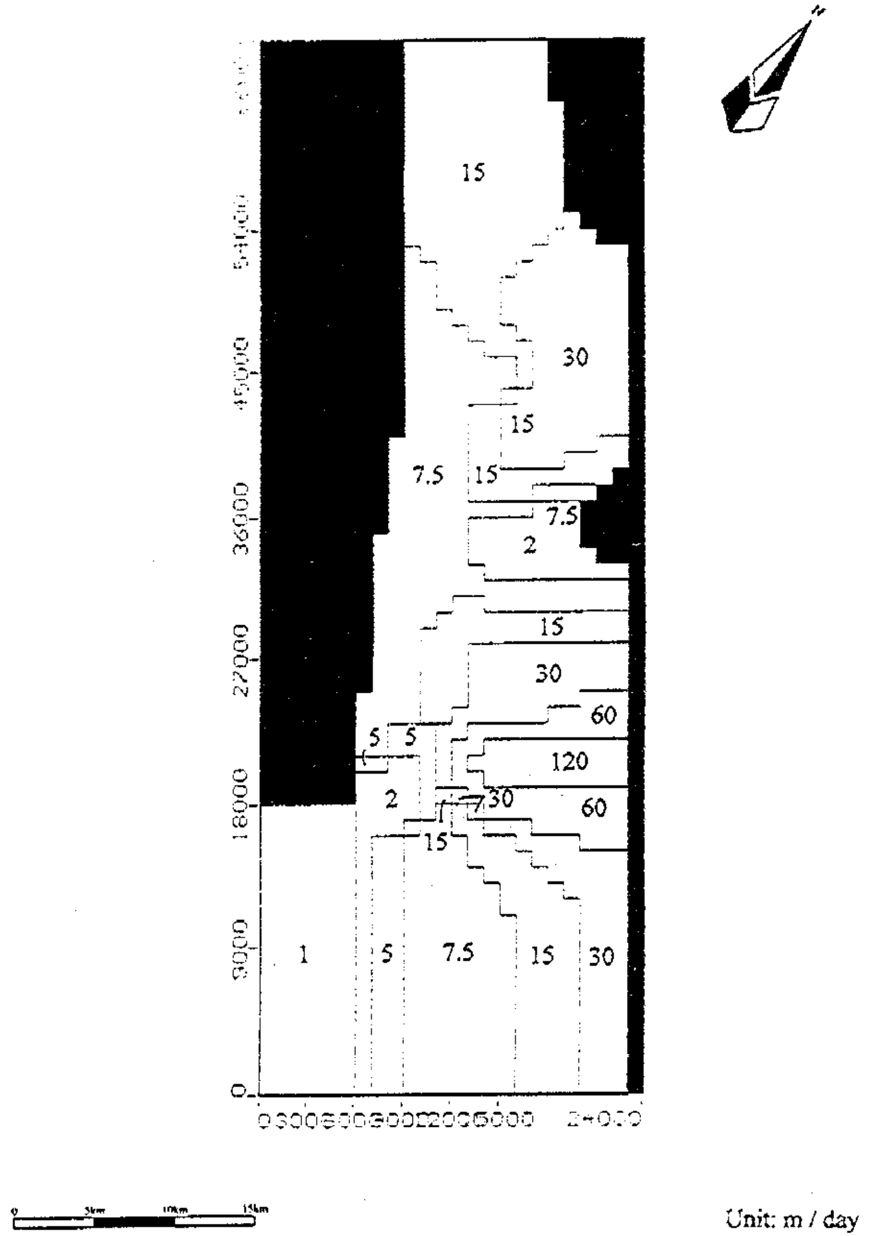

Fig. 12.3.2-7 Fixed Hydraulic Conductivity of Unconfined Aquifer (Layers 1-3)

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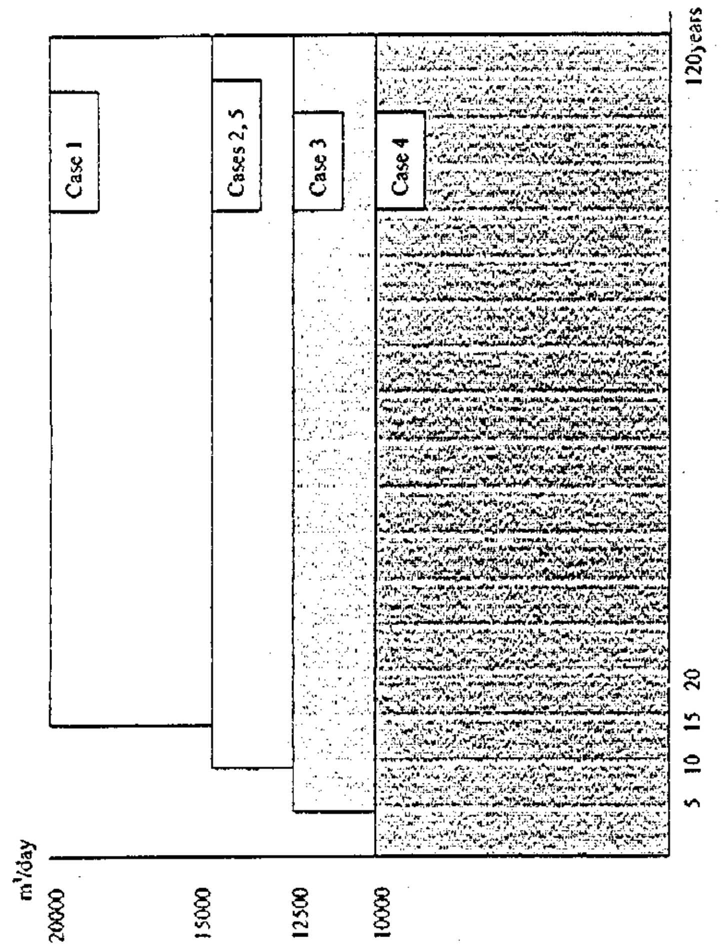

Fig. 12.4.1-1 Study Cases of Development Plan

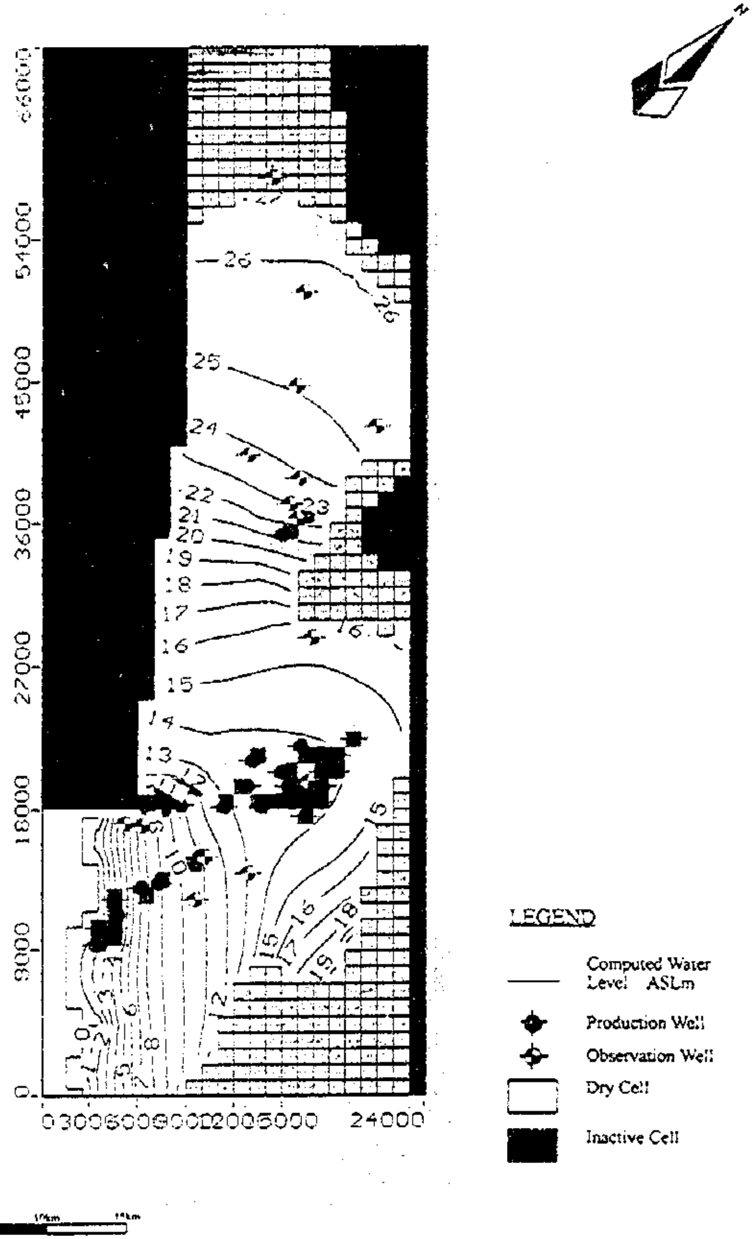

Fig. 12.4.3-1 Calculated Water Level of Unconfined Aquifer, Case 1 (120 years after)

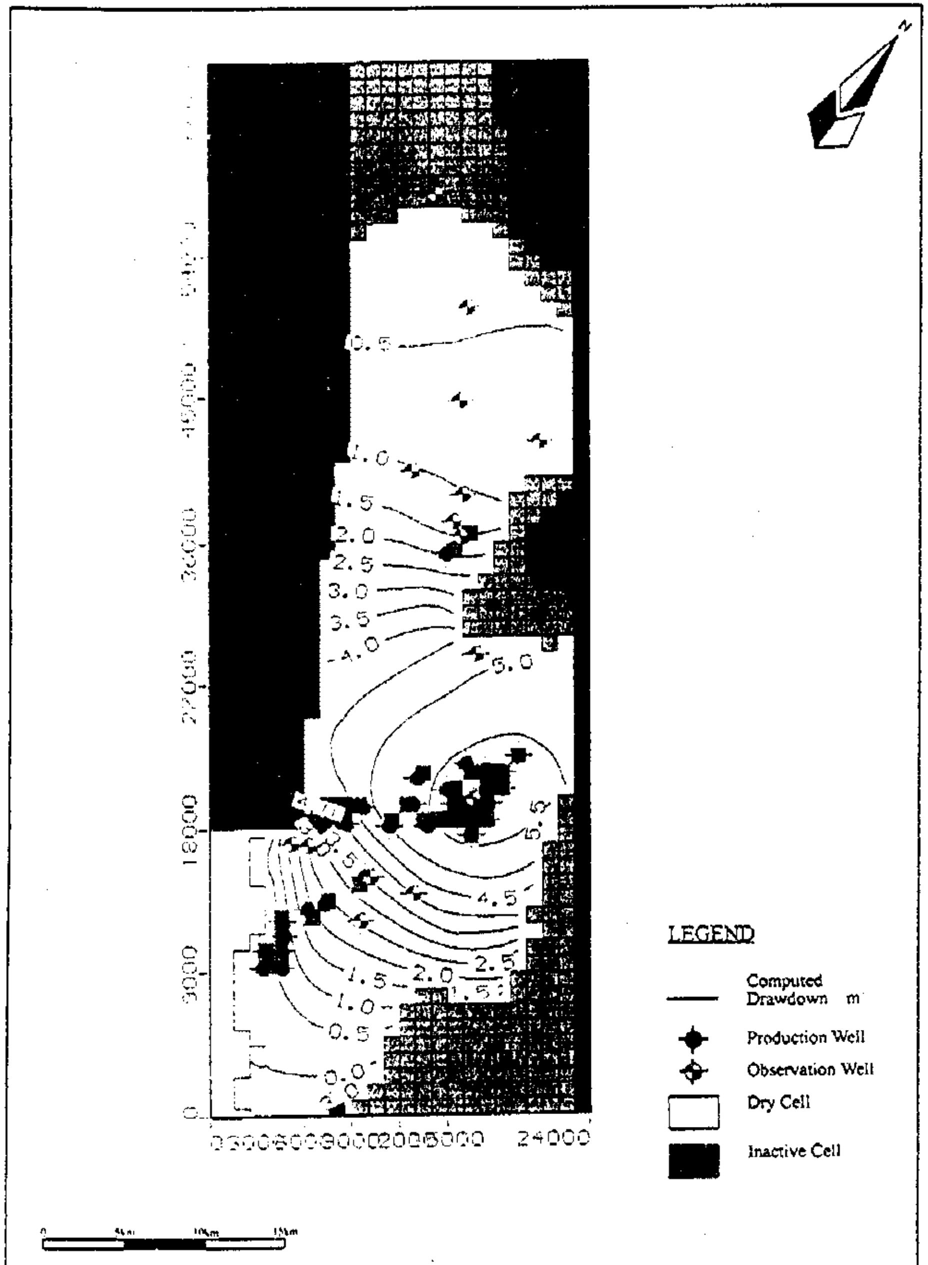

Fig. 12.4.3-2 Calculated Drawdown of Unconfined Aquifer. Case 1 (120 years after)

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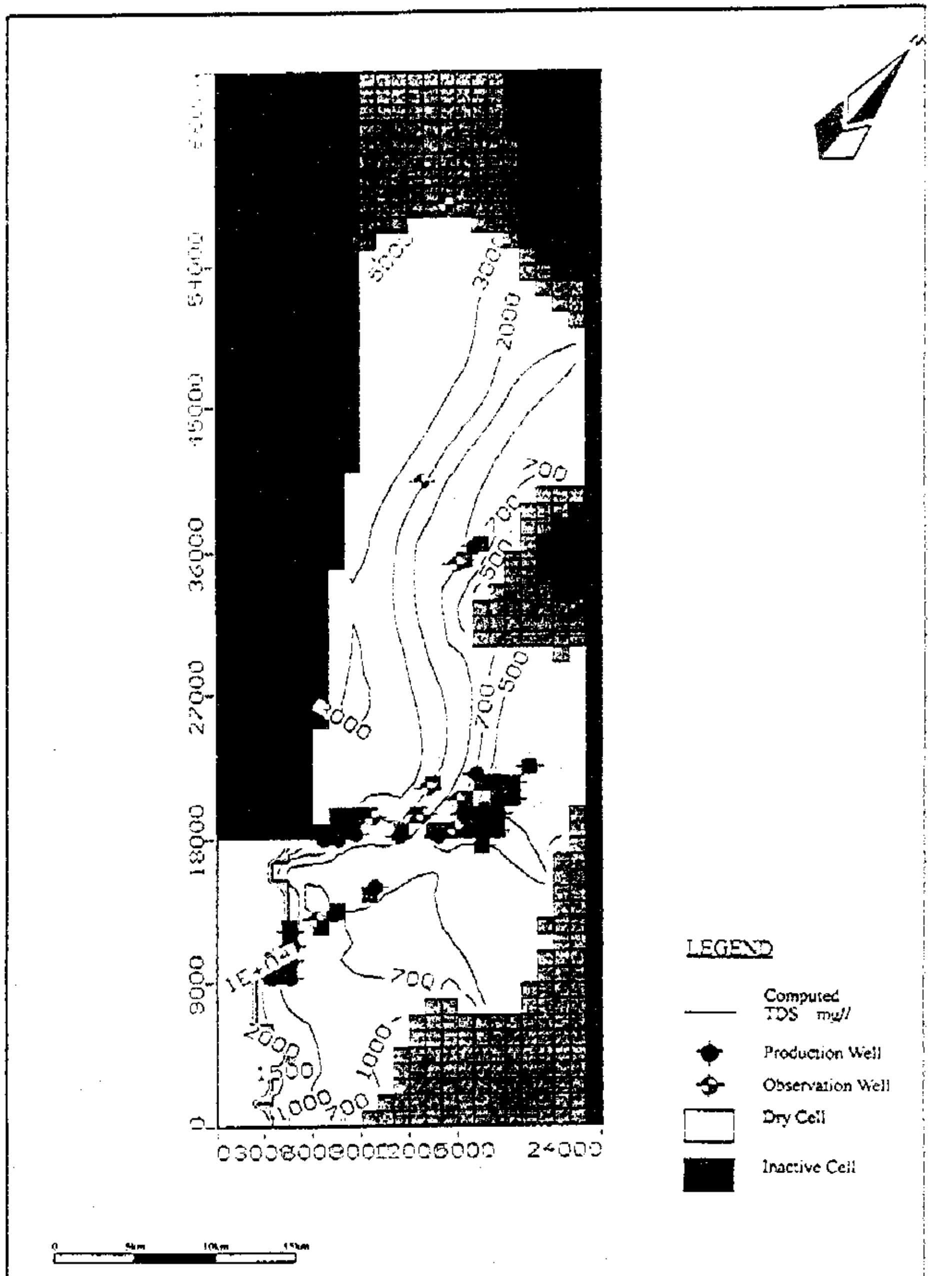

Fig. 12.4.3-3 Calculated TDS of Unconfined Aquifer. Case 1 (120 years after)

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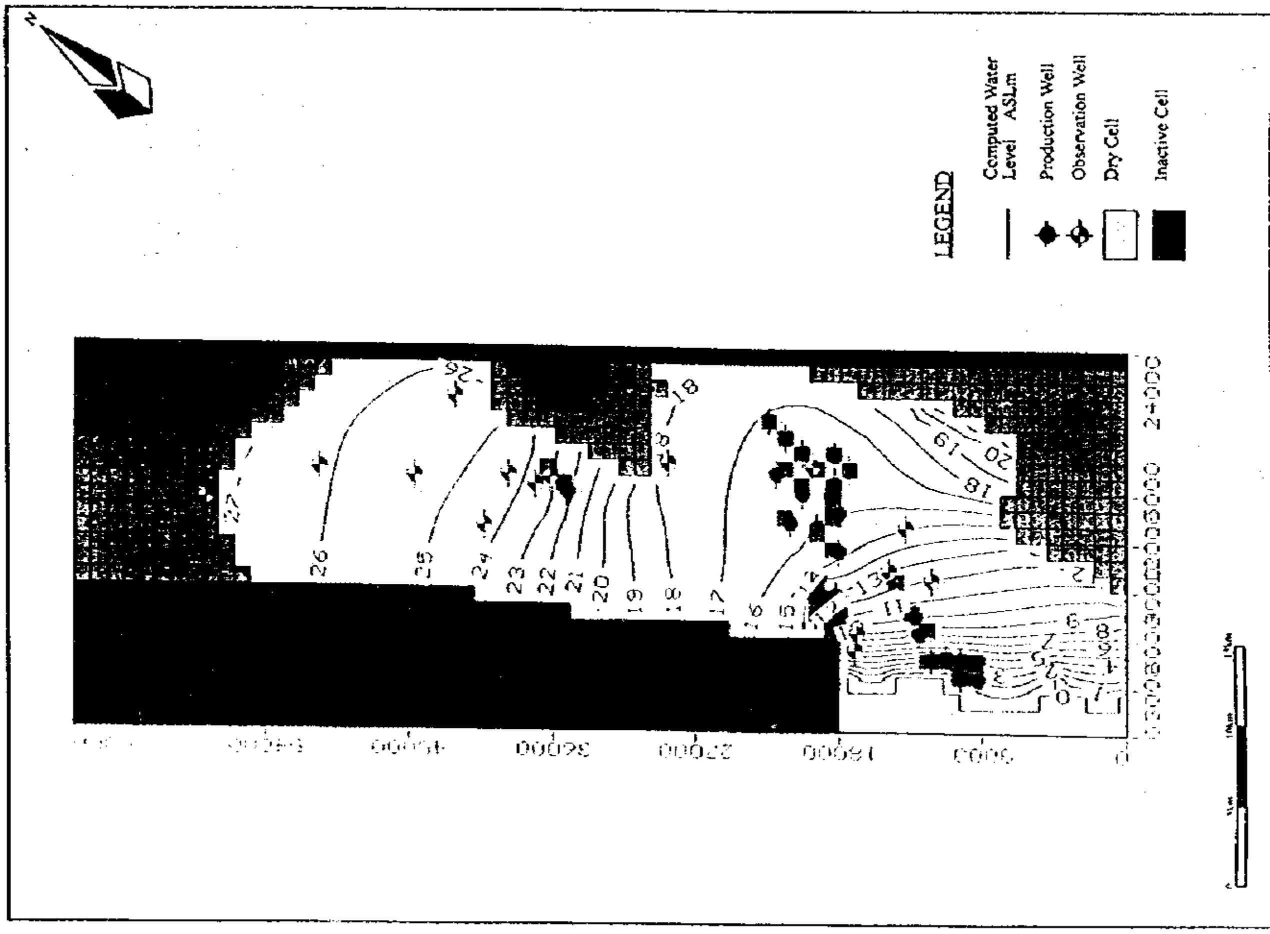

Fig. 12.4.3-4 Calculated Water Level of Unconfined Aquifer, Case 2 (120 years after)

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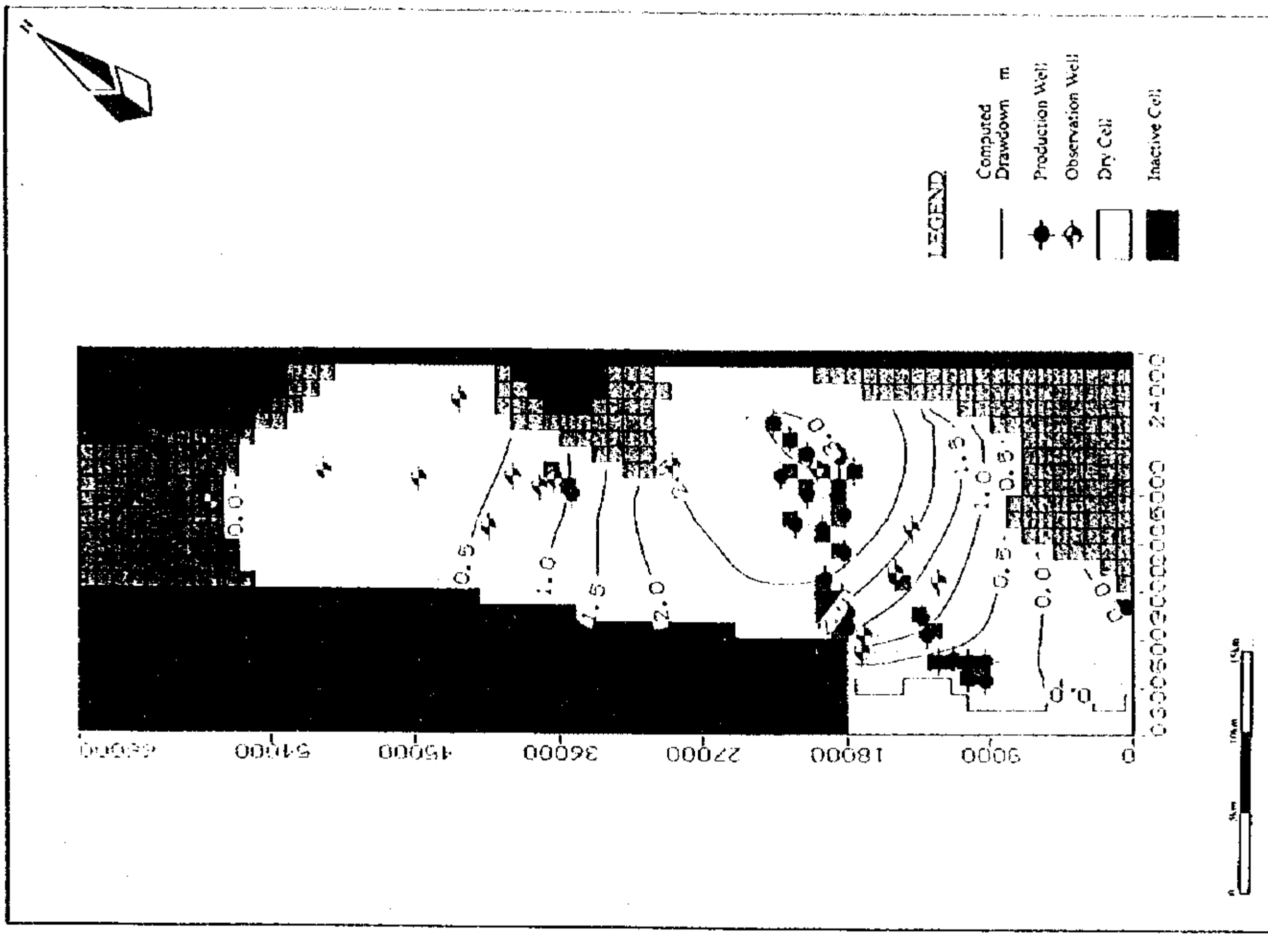

Fig. 12.4.3-5 Calculated Drawdown of Unconfined Aquifer, Case 2 (120 years after)

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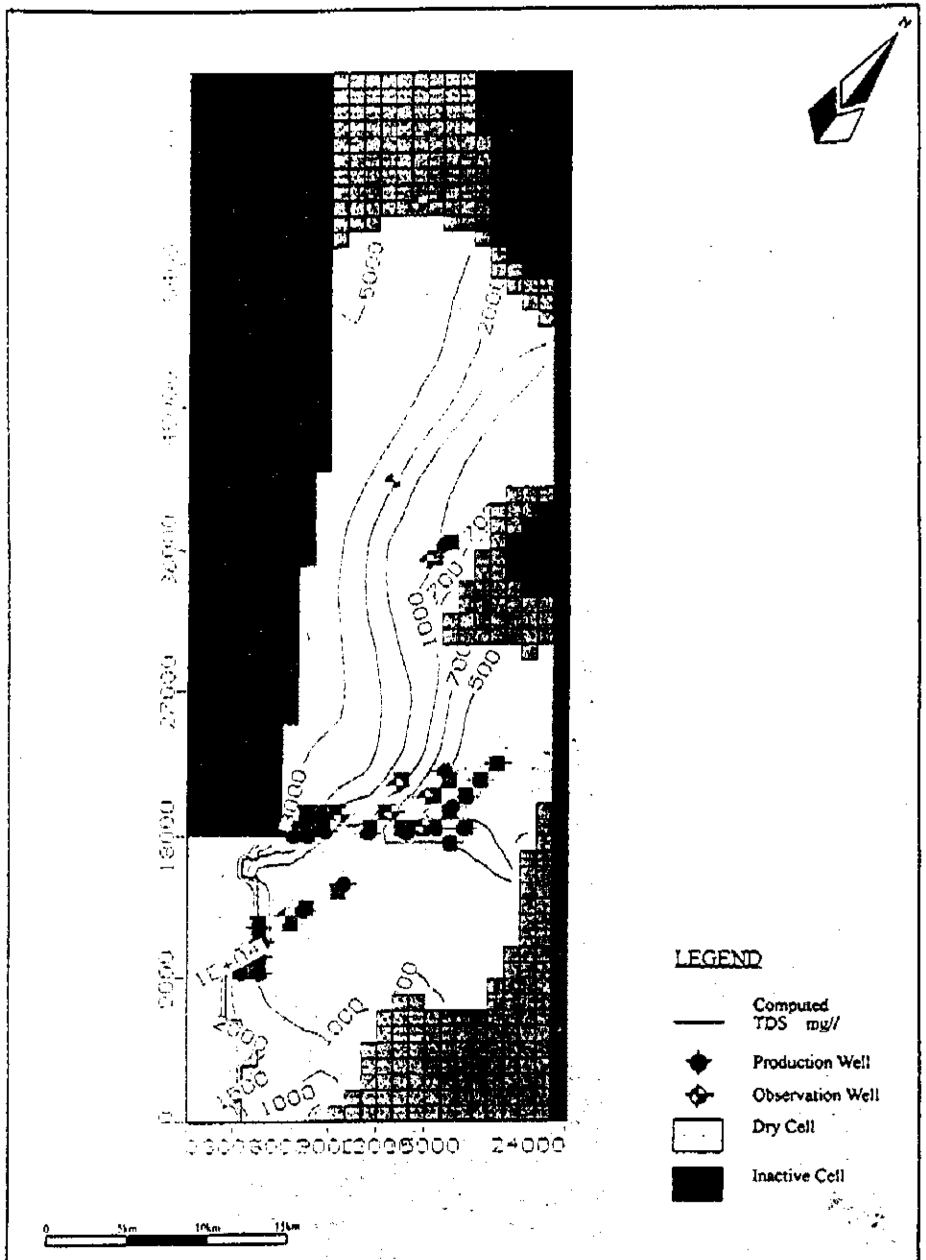

Fig. 12.4.3-6 Calculated TDS of Unconfined Aquifer, Case 2 (120 years after)

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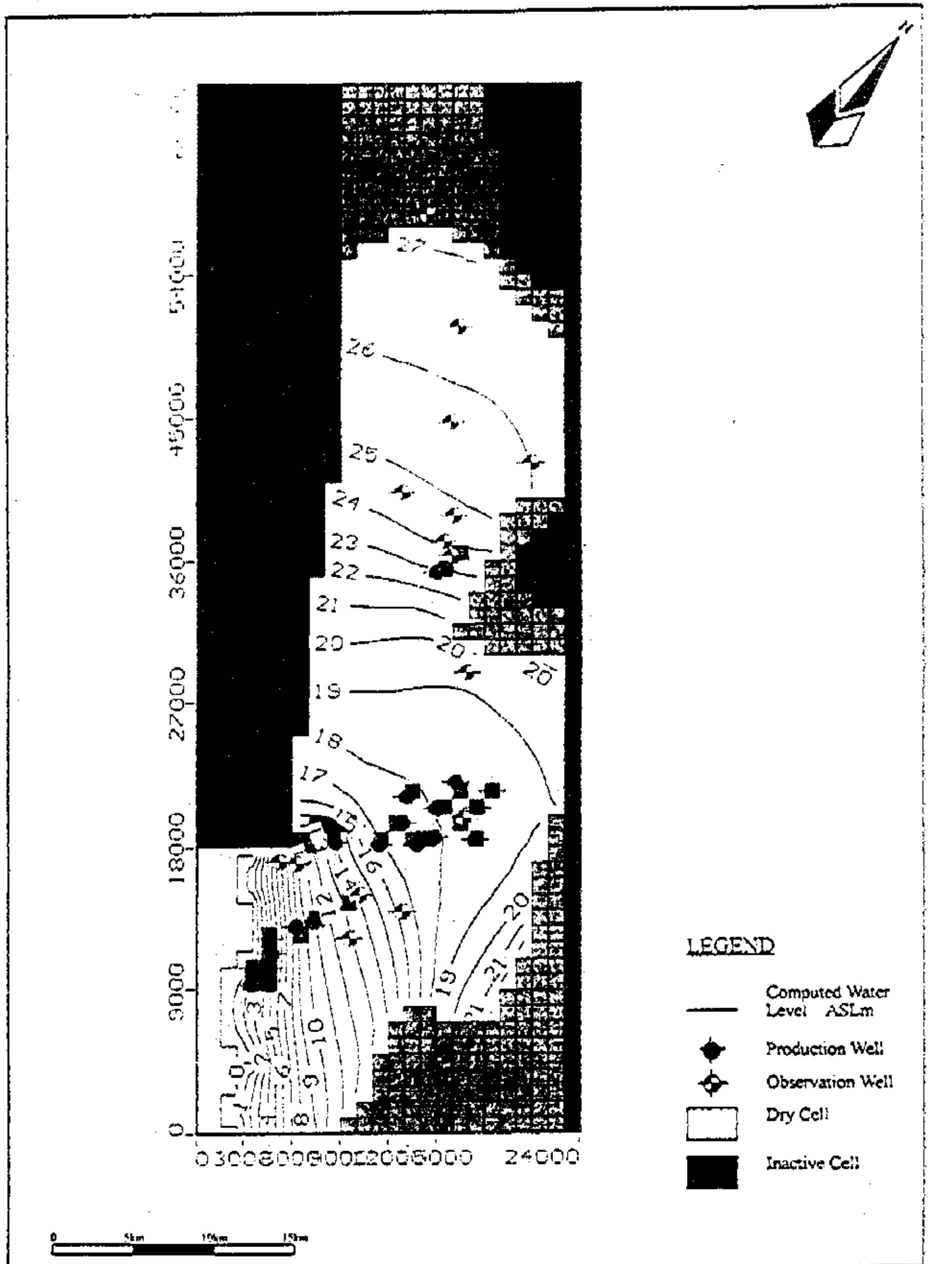

Fig. 12.4.3-7 Calculated Water Level of Unconfined Aquifer, Case 3 (120 years after)

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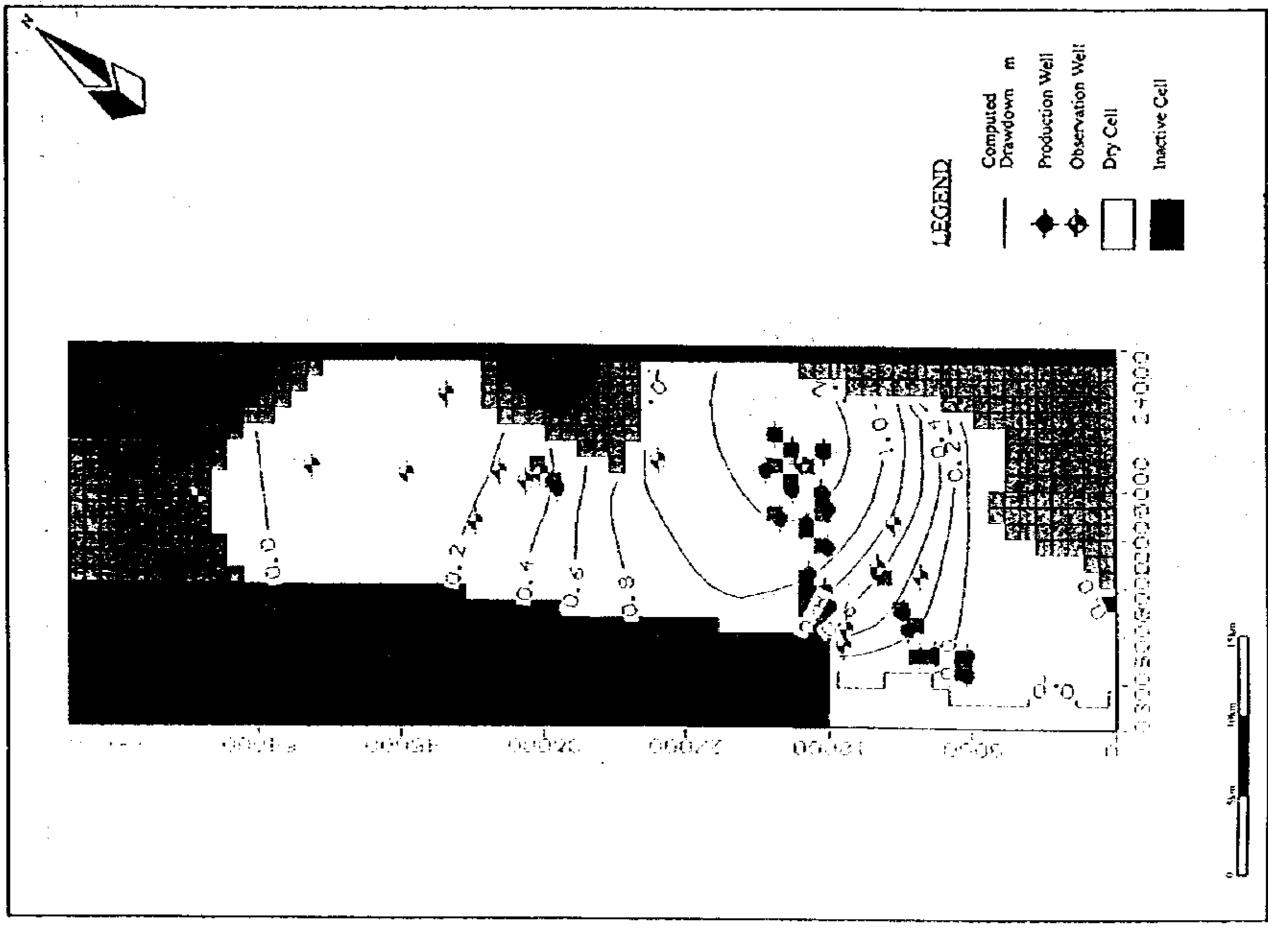

Fig. 12.4.3-8 Calculated Drawdown of Unconfined Aquifer, Case 3 (120 years after)

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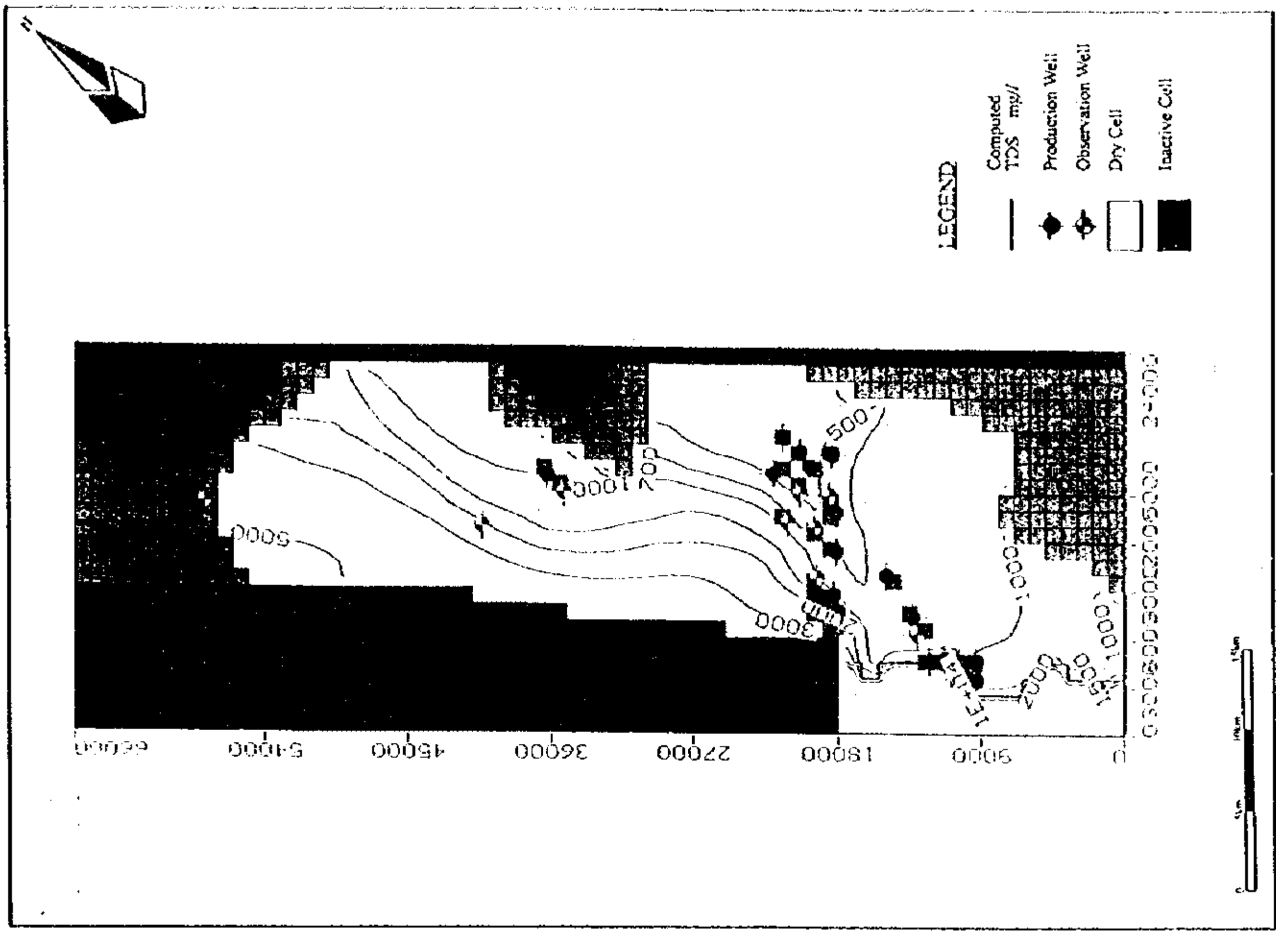

Fig. 12.4.3-9 Calculated TDS of Unconfined Aquifer, Case 3 (120 years after)

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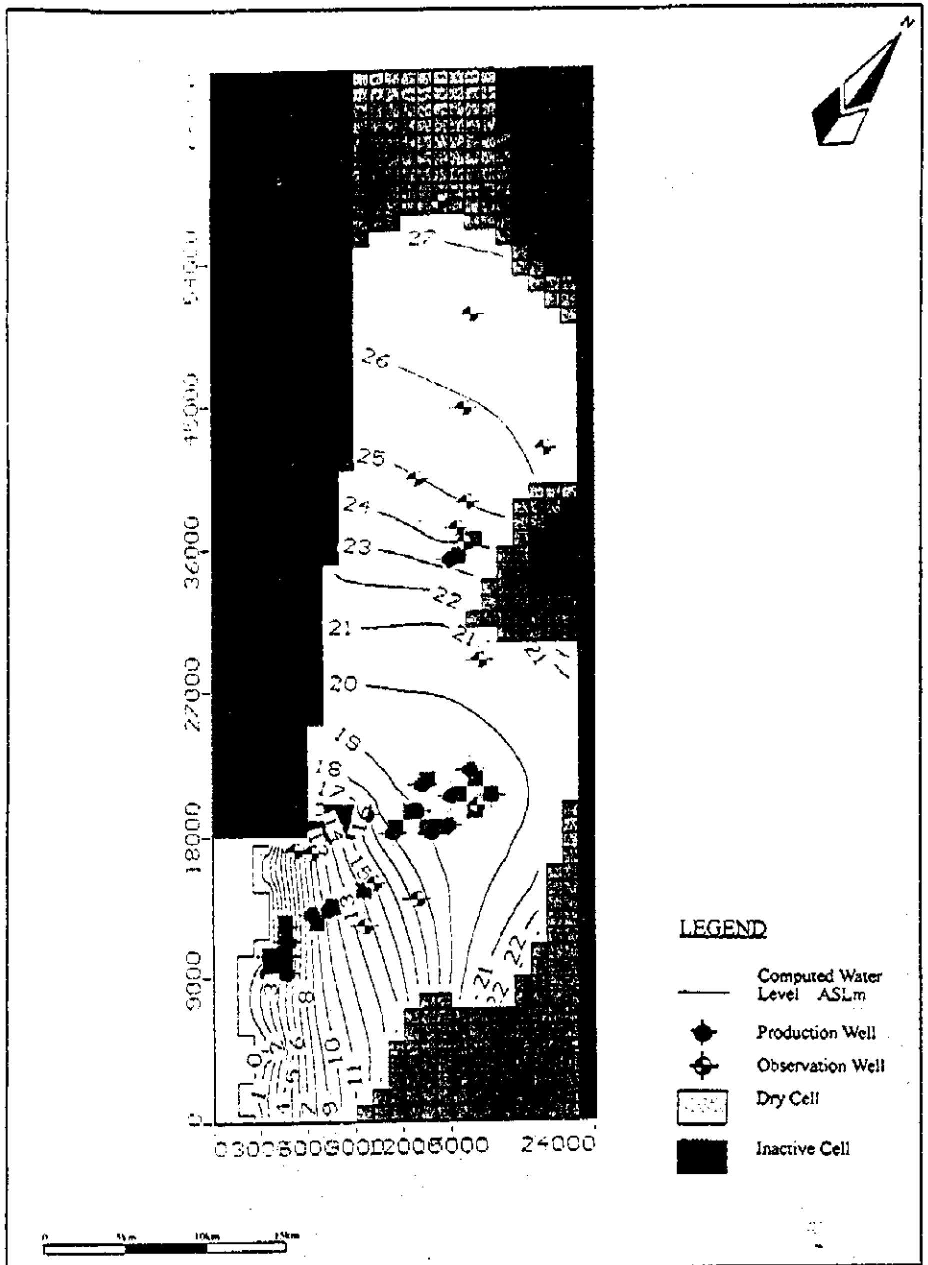

Fig. 12.4.3-10 Calculated Water Level of Unconfined Aquifer, Case 4 (120 years after)

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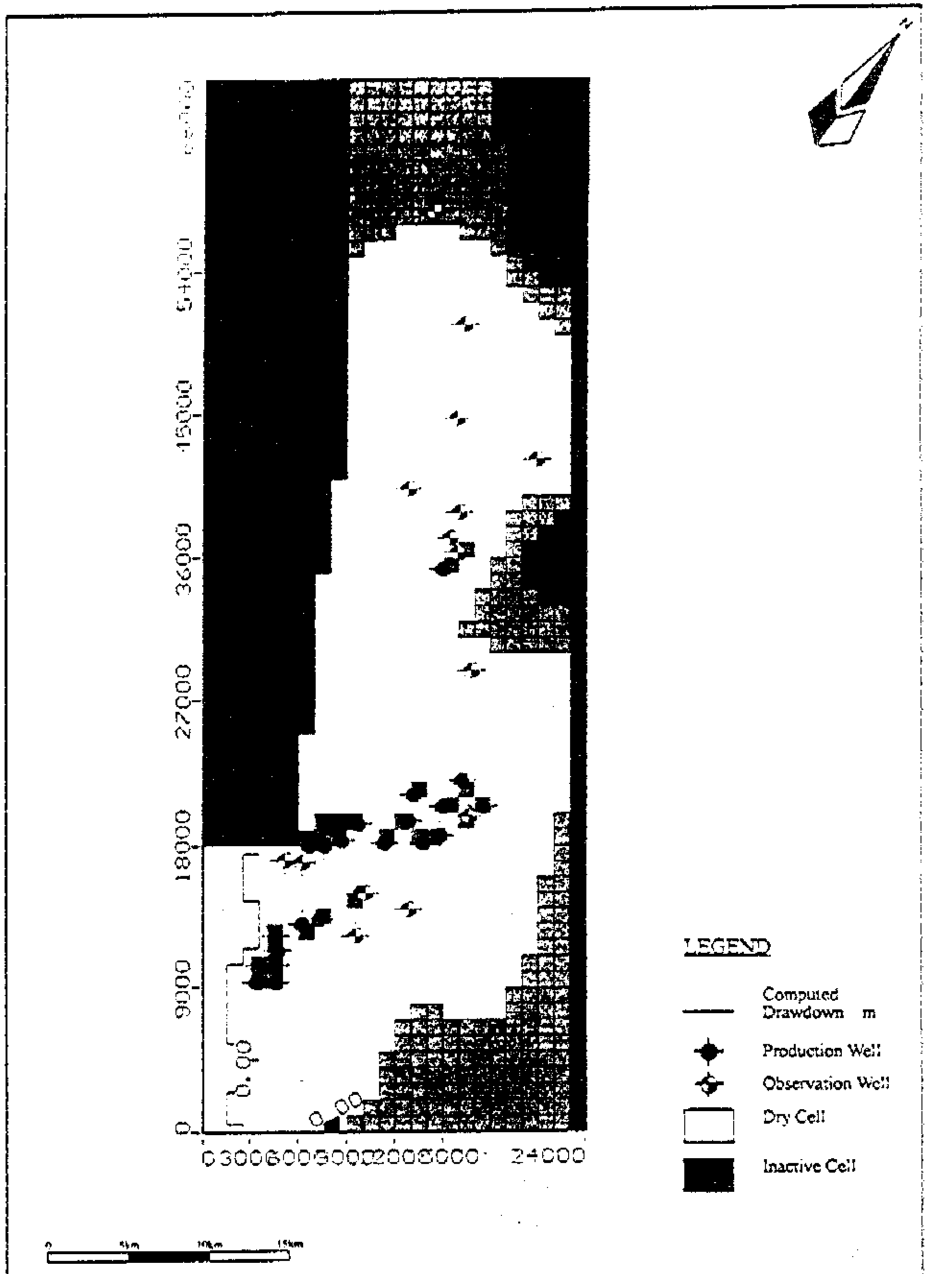

Fig. 12.4.3-11 Calculated Drawdown of Unconfined Aquifer, Case 4 (120 years after)

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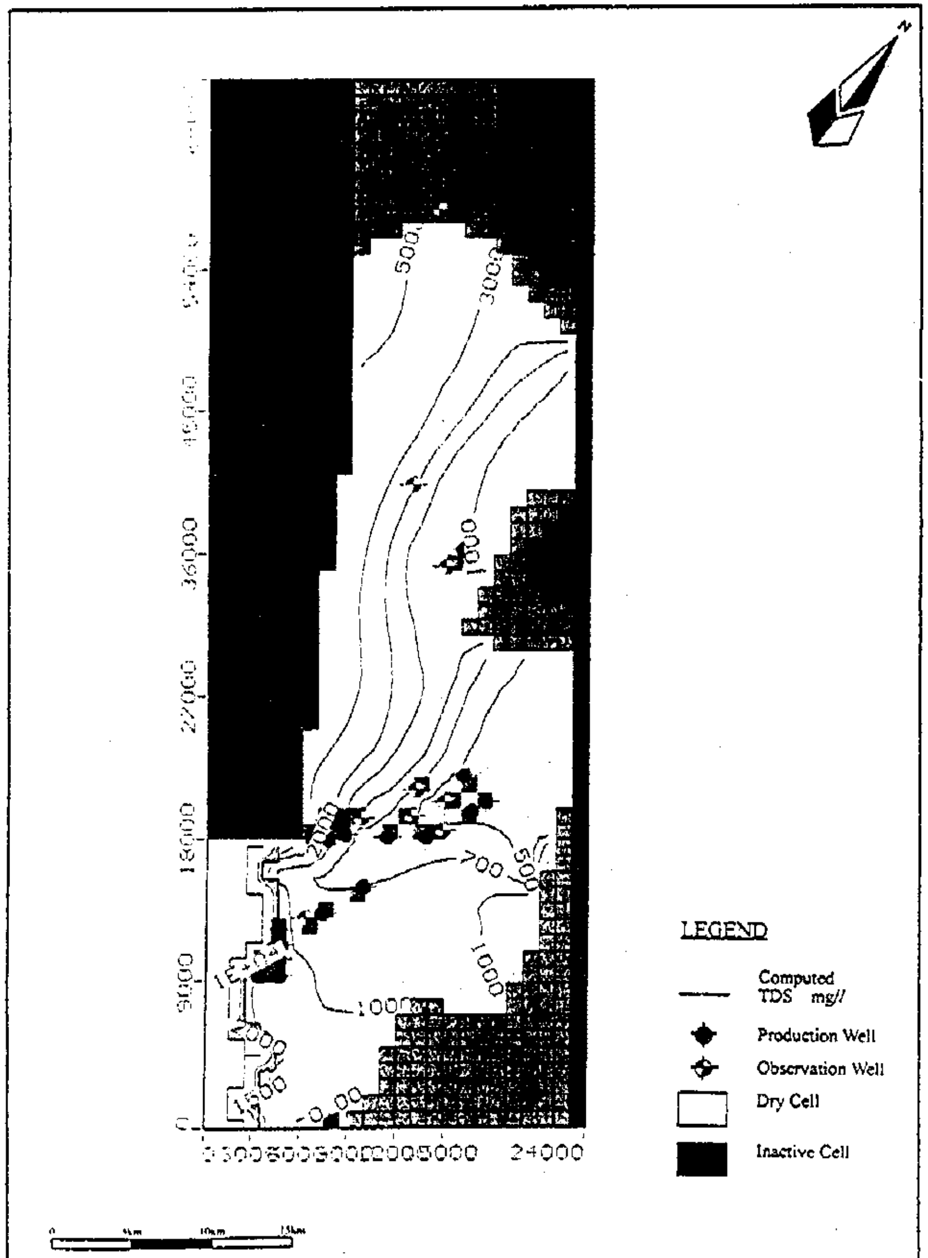

Fig. 12.4.3-12 Calculated TDS of Unconfined Aquifer, Case 4 (120 years after)

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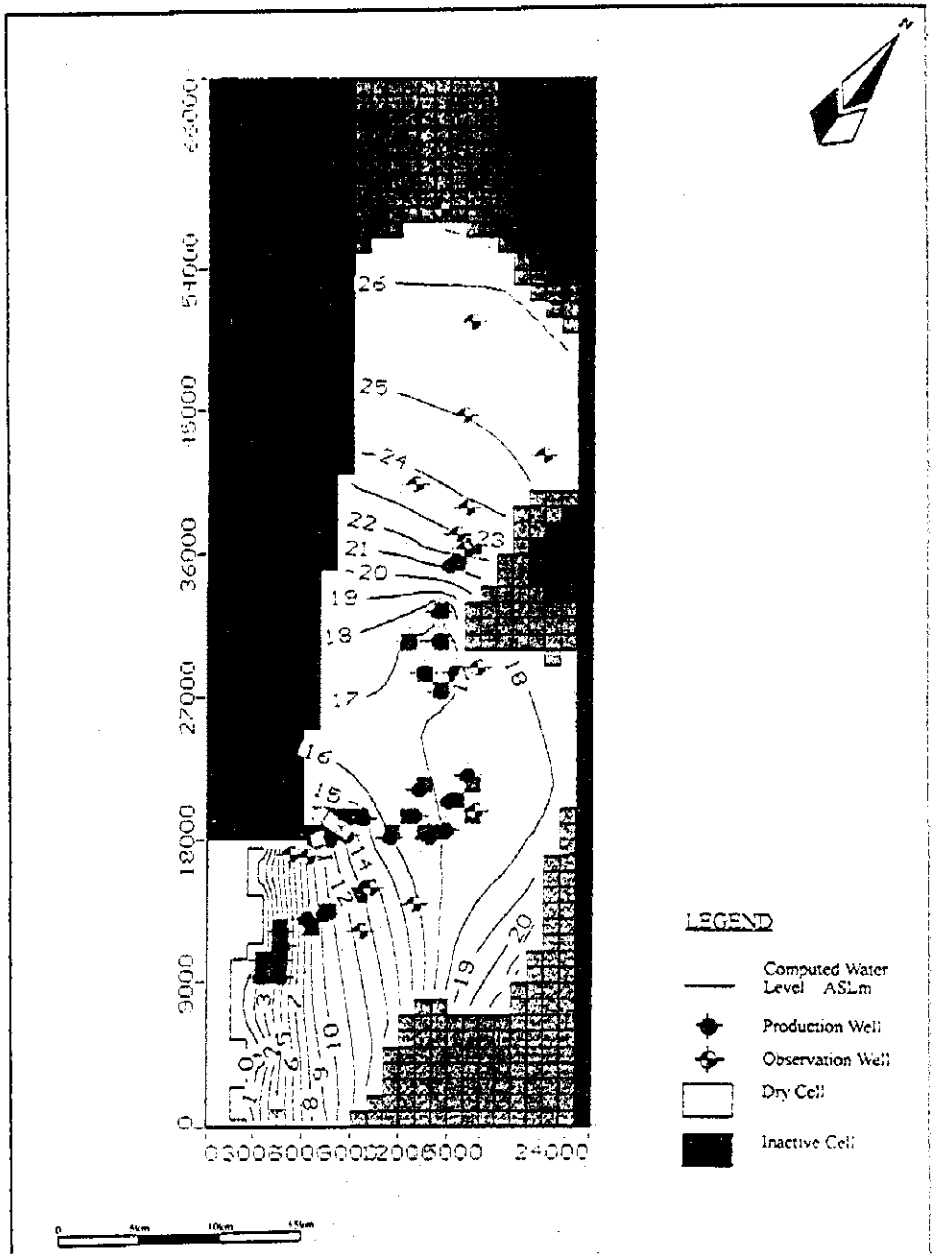

Fig. 12.4.3-13 Calculated Water Level of Unconfined Aquifer, Case 5 (120 years after)

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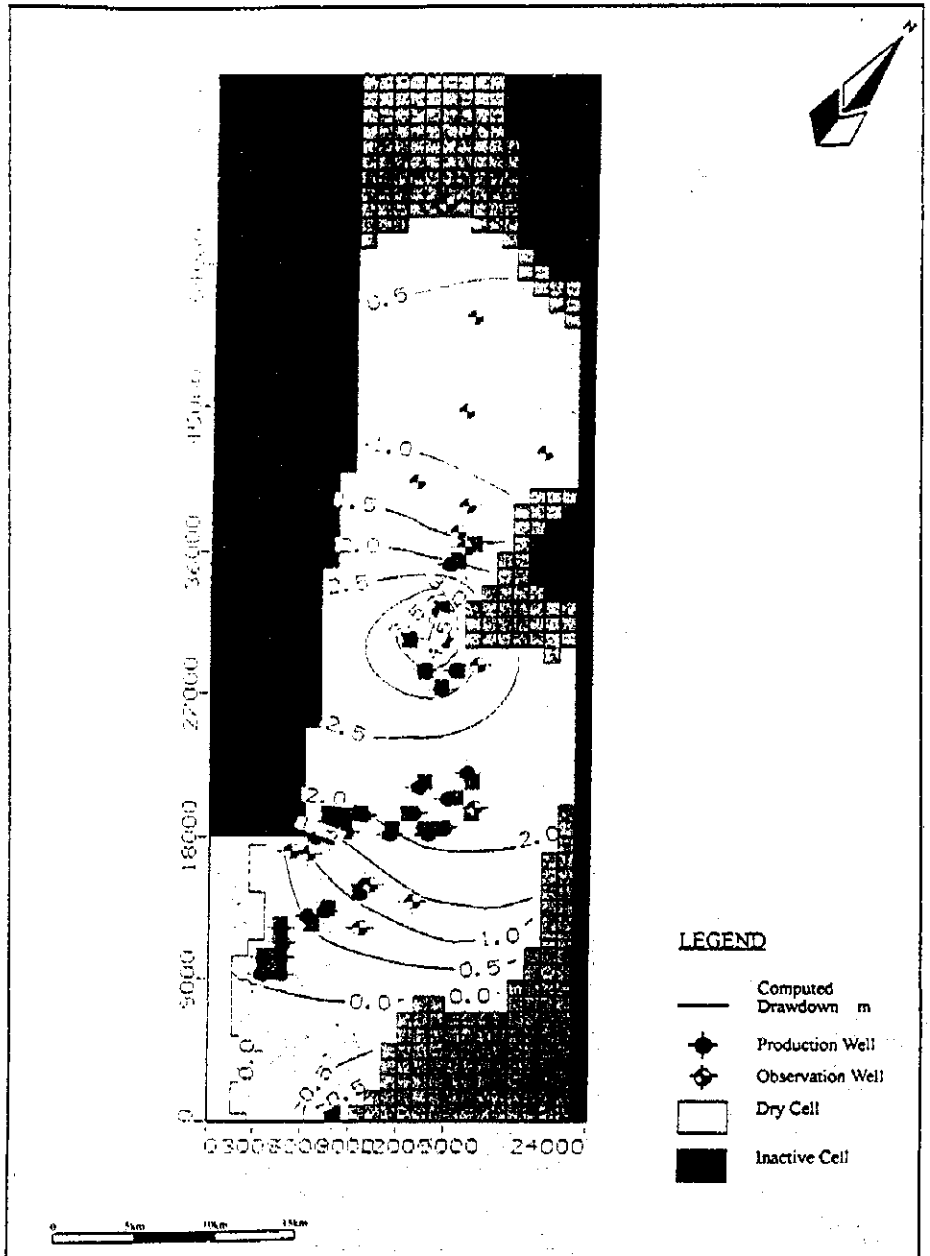

Fig. 12.4.3-14 Calculated Drawdown of Unconfined Aquifer, Case 5 (120 years after)

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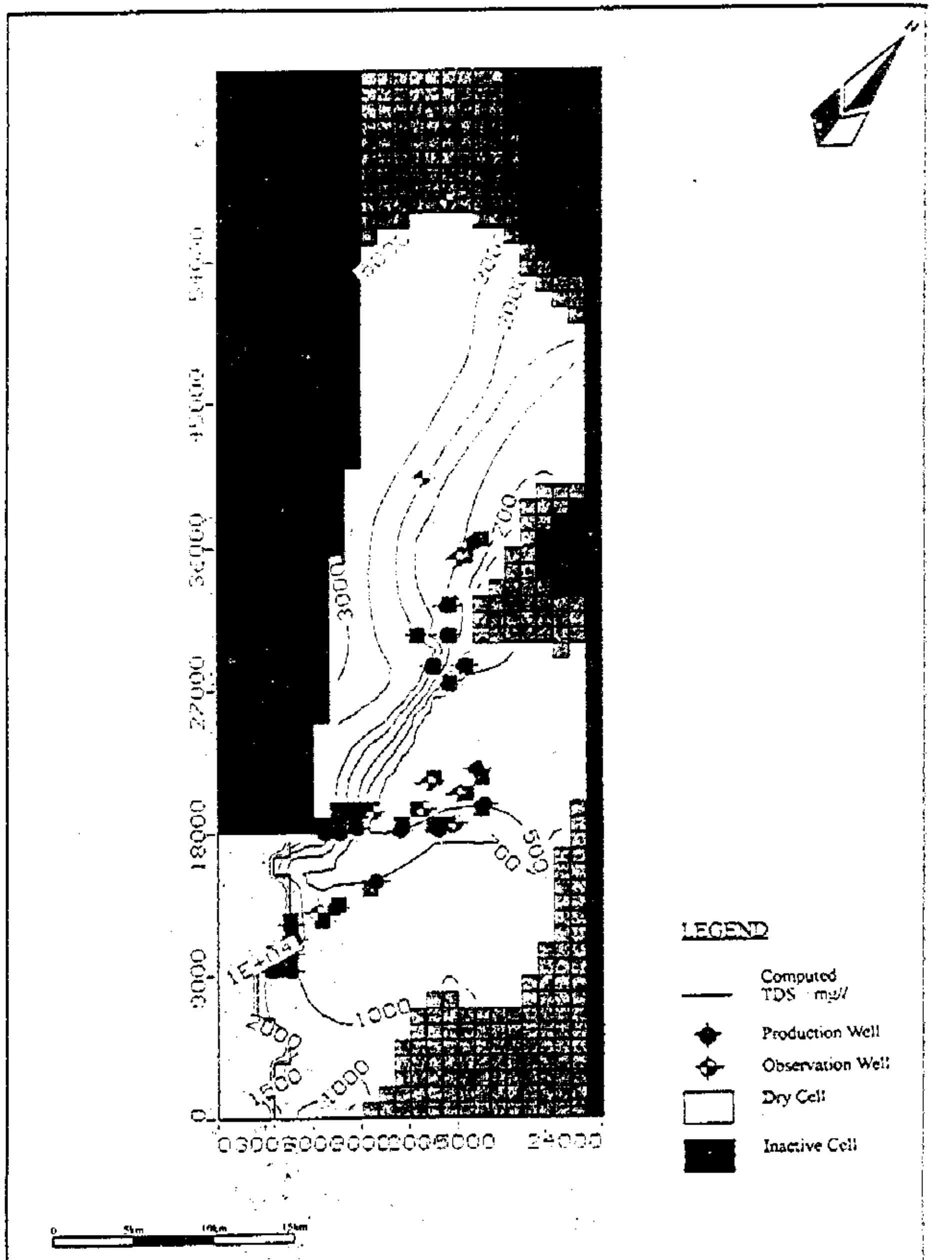

Fig. 12.4.3-15 Calculated TDS of Unconfined Aquifer, Case 5 (120 years after)

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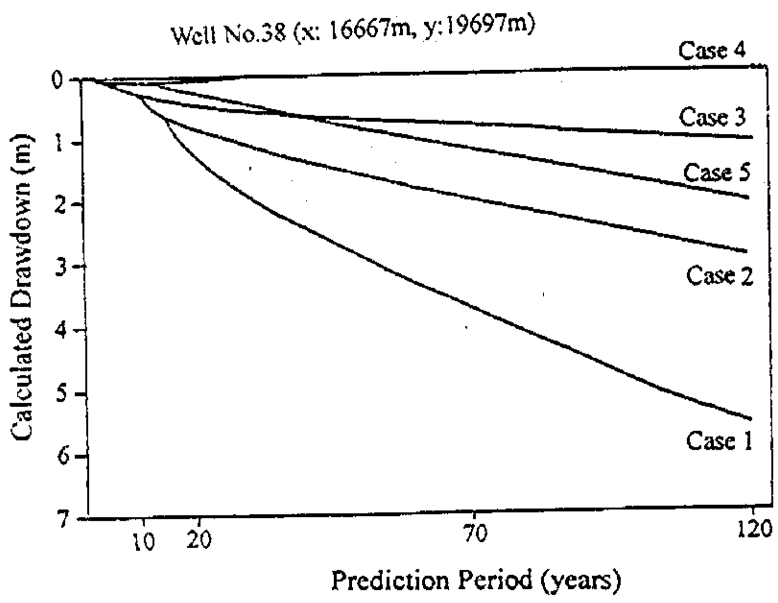

Fig. 12.4.3-16 Variation of Water Level of Unconfined Aquifer at Well No.38

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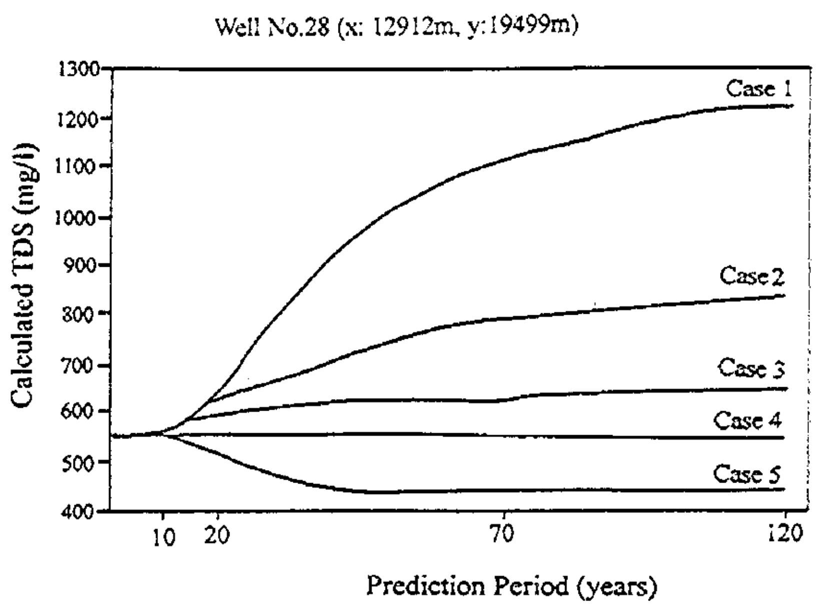

Fig. 12.4.3-17 Variation of TDS at Well No.28

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CHAPTER XIII GROUNDWATER DEVELOPMENT PLAN

13.1 Design Conditions

13.1.1 Water Sources and Development Water Capacity

1) Water Source

The water sources for potable water and irrigation use for this study is intended to the groundwater that is developed as a result of the Study. The locations of these water sources are planned in an altitude of approximately 510 to 520 m in the southern area of Nakhl city. As a part of future water demands in El Tur city, water sources will also be developed in El Qaa Plain.

2) Water Capacity to be Developed

The water capacity to be developed is determined in accordance with the future water demand forecasting as mentioned in Chapter 10. The conveyance facilities plan in this development will aim to make the best use of the developed groundwater. These facility plans will describe later.

Plan 1: Supply to West Coast (The Gulf of Suez) Side

Plan 2: Supply to East Coast (The Gulf of Aqaba) Side

Plan 3: Supply to El Tur City

Plan 4: Supply to Agriculture Use (Sudr El Heitan, Malha, Themed)

Plan 5: Supply to Bedouin Community (As a Typical Case)

The development water capacity and target year for the facility plans are shown in the following Table. The target year of Plan 3 is planned as 2007 from a point of view of the water balance in the site proposed for water source. However, other plans are designed for 2017 as target year of the Study.

Water Capacity to be Developed and Target Year

Development Plan	Target Year	Development Capacity *1 (m^3/day)	Service Area	Main Purpose of Supply
Plan 1	2017	57,500	Ras Sudr, Abu Zenima, Abu Rudeis	Residential use *2
Plan 2	2017	35,000	Nuwiba, Taba	ditto
Plan 3	2007	5,300	El Tur	ditto
Plan 4A	2017	11,700	Sudr El Heitan	Agriculture use
Plan 4B	2017	13,700	Malha	ditto
Plan 4C	2017	11,700	Themed	ditto
Plan 5	2017	5, 25, 50	Bedouin Community	Common use

Note
* 1 : Development Capacity = (water demand in target year) - (existing water demand in 1997)
* 2 : Residential use is including the future water demand for the tourism and industrial use.

13.1.2 Design Criteria

1) Design Unit Water Demand

The daily average of unit water demands of each categories are mentioned below, the detailed number of them are shown in Table 9.1.3-1.

Residential Use (urban area)	:	240 liter/capita/day
Residential Use (rural area)	:	120 liter/capita/day
Tourism Use	:	400 liter/capita/day
Industrial Use	:	107 m^3/ha/day
Agriculture Use	:	16.3 m^3/ha/day

2) Potable Water Quality Standard

Potable water quality standard for Egyptian and WHO are shown in Table 9.1.3-2. From the table, the upper limit of TDS is 1,500 mg/l and 1,000 mg/l respectively. Therefore, maximum TDS of groundwater to be developed as potable water in the Project is assumed as 1,500 mg/l. However, water quality in the plan 5 that is developed for rural area, may exceed this level because the planned water source is the dug wells.

3) Design Criteria for Water Works Facilities

Proposed water supply and auxiliary facilities for potable and agriculture use will be designed using appropriate technology, in accordance with practices proven to be suitable in Egypt. The following design standards (a,b,c) and other items will be adopted in the design.

a. The Egyptian Code of Practice for the Design and Construction of Pipelines for Water Supply and Sanitary Drainage Nets, MODANC 1990.

b. If the above code is not sufficient to do the designing, the criteria of Japan Water Works Association will be used as the reference materials.

c. WHO Guideline.

Other criteria for water works facilities are as follows.

(i) Selection of pipe materials (for collection and conveyance pipeline)

The pipe materials of collection and conveyance pipeline shall be selected with consideration to the purpose, soil condition and so on, choosing from steel pipe, polyvinyl chloride pipe (u-PVC) and ductile cast iron pipe.

(ii) Connection methods

The major connection method under the ground of pipe laying works is as follows.

ductile cast iron pipe, u-PVC	: socket type
steel pipe	: welding and flange type

(iii) Maximum inner pressure : (p)

p = less than 120 m

(iv) Minimum Soil cover depth : (h)

h = more than 120 cm

(v) Interval of valve installation : (L), for maintenance works of pipeline.

L = less than 2 km

(vi) Capacity of the distribution water reservoir : (V).

V_1 = more than 12 hr (for the urban area)
V_2 = more than 24 hr (for the rural area)

13.1.3 Major Consideration Matters of the Water Works

1) Water Source (as Groundwater)

For groundwater intake facilities, the following shall be observed:

a. Intake facilities should be of such construction as ensures long and stabilize intake of the planned intake amount. The pumping facilities should be of such type as is free from clogging of the strainer throughout the year, or for a longer period.

b. Pump lift of the water must be decided on complete surveys and investigations. Proper pumping amount for the well should be determined after sufficient investigations such as various kinds of pumping tests.

c. Intake facilities should desirably include metering instruments for water levels and quantities, intended to secure the planned intake and/or control it.

d. Intake facilities should be protected from dangers and pollution by provision of fencing, and other means. Groundwater is a quality sanctified resource and so deserves our attention to preventing pollution from outside. Particularly, for intake facilities for spring water, roofing is required.

2) Conveyance System

In principle, planned conveyance shall be based on the planned intake. However, when there is a prospect for an intake rise in the case of future development or when the conveyance facilities are to be built in such places as listed below, rooms for expansion should beneficially be provided.

a. Place where the construction is considered difficult like (like tunnels, embankment, crossing, etc.,)
b. Simultaneous in construction is considered cost saving compared to separate work for expansion in the future.
c. Spots where obtaining of site for future expansion seems difficult.

Conveyance types are classified as listed below according to the difference of the systems. Generally, gravity system is safer and more reliable than pumping system from maintenance and management standpoint. In case the water level at the starting point is lower than the end point, pumping system must be employed.

a. Gravity type or boosting pump system from difference in water level between the starting and end point.
b. Open conduit and pipeline systems from hydraulic point of view.
c. Surface style (open conduit) and underground style from the connection with the surface of ground (pipeline, closed conduit and tunnel).

3) Purification System

With regards to decision of purification method, the following must be observed:

a. Purification facilities shall have sufficient capability of supplying required quantities of purified water consistently meeting the specified quality standards.

b. In the method selection for water purification, the most suitable one shall be chosen with provision of disinfection equipment, from among the one's listed below with consideration to the quality of raw water, amount of filtrate, procurement of site, construction and maintenance costs, convenience of management, level of management, etc.

-1 Method solely dependent upon chlorination
-2 Slow sand filtration
-3 Rapid sand filtration
-4 Method accompanied by special treatment

The water sources for supply water is groundwater that has been developed in the Study. Therefore, purification facilities for groundwater to be extracted from the deep wells will not be necessary, expect disinfection facility.

13.2 Planning of Water Development Facilities

13.2.1 Plan 1 : Water Supply to West Coast Side

1) Outline

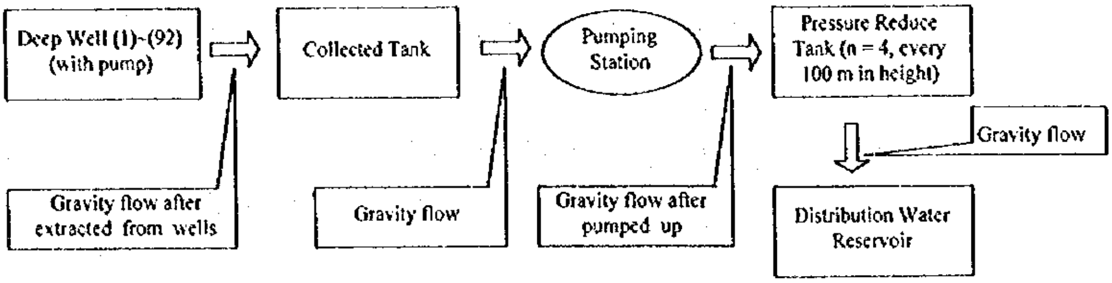
The intake points for the water sources are located in the southern area of Nakhl city, and extracted water will be conveyed to Ras Sudr city. The development water capacity is planned on the assumption with future water demand in 2017 in Ras Sudr, Abu Zenima and Abu Rudeis cities. The major development condition for the Plan 1 is listed below. The scope of the facilities plan is conveyance facilities from water intake to the distribution water reservoir located in the suburb of Ras Sudr city, through the one (1) pumping station and four (4) pressure reduce tanks.

The detailed design conditions and specifications are described in Data Book.

-1	Purpose of Supply	:	For the potable water in Ras Sudr, Abu Zenima and Abu Rudeis cities.
-2	Development Capacity	:	57,500 m^3/day
-3	Target Year	:	2017

2) Water Flow Diagram and Project Location

The water flow diagram from wells to distribution water reservoir is shown below.



Water flow diagram and project layout are shown in Figure 13.2-1(1) and 13.2-2(1) respectively.

3) Water Levels of Facilities and Longitudinal Profile of Conveyance Pipeline

Water levels of all facilities which consists of the intake wells, pumping station, pressure reduce tanks and distribution water reservoir, are shown in Figure 13.2-3(1).

The longitudinal profile of conveyance pipeline between pumping station and distribution water reservoir is shown in Figure 13.2-4(1)

4) Development Facilities

The water facilities to be developed in Plan 1 are classified into intake facility whose water source is groundwater from deep wells and conveyance facility to the distribution water reservoir. The details of each facility are described below.

(1) Intake Facility

i) Required Number of Intake Wells

The safe yield of the single well at the proposed well field is estimated as 720 m^3/day from results of the pump test and actual results in surrounding areas. The design capacity of the water source is 57,500 m^3/day. Therefore, the total number of wells required is ninety two (92) including twelve (12) spare wells for maintenance and emergency.

ii) Influence Radius of Well

The influence radius of a well is calculated employing the Modified Cooper - Jacob equation (20 years of pumping duration, 0.8 m of drawdown). The influence radius is calculated to be 380 m. The wells are allocated in flat field. Thus, the water level of each well is influenced by pumping wells located on each side. The sum of the drawdown influenced by the adjacent wells comes to 1.6 m. The drawdown of this range in the long-term pumping will have no influence on the pumping of the planned intake volume (720 m^3/day per well) because the drawdown is small. Thus, it is considered within the permissible range and the influence radius is designed to be 380 m or more. In this case, the distance between the production wells should be 760 m or more.

The distance of wells interval that is in accordance with radius of influence, refer to Data Book.

iii) Design of Intake Well

The average depth of wells is set at 1,000 m considering the depth of aquifer. The diameter of wells will be larger than the test wells. The casing diameter at pump chamber is set at 300 mm, and its length is set at 800 m. The diameter of casing and screen pipes of the other parts is proposed to be 200 mm. The total length will be 1,000 m including sand trap.

The opening of the screen is determined considering the inflow velocity of groundwater. The inflow velocity has to be set below 15 mm/sec according to the Japanese standard for water supply facility. To set the inflow velocity below the specified value in the standard, the screen pipes of Johnson type with the opening ratio of 20 - 25 % should be applied. The production yield of the proposed well is 720 m^3/day.

- Well depth : 1,000 m
- Casing size at pump : ϕ300 mm, 800 m long
- Screen size : ϕ200 mm, 150 m long
- Screen type : V slot Johnson type
- Production yield : 720 m^3/day

iv) Total Head and Installed Depth of Well Pump

The dynamic and static well water levels considering the drawdown are 300 m and 280 m below ground surface respectively. Therefore, the total pump head is calculated to be 315 m adding this variation and friction loss of the pipeline to the collected tank. The submersible pumps will be installed at a depth of approximately 320 m. These facilities consists of well, well pump, flow meter and control panel and supplementary facilities are installed in the intake pump house.

v) Collection Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. However, the required total head of submersible pumps becomes higher because of increasing friction losses in the pipe resulting in higher operation and maintenance costs of the facility.

The economical velocity in the collection pipeline is generally not more than 1.5 m/sec although the conditions at the location should be considered. In this case, the economical diameter of the collection pipeline is determined to be 125 to 900 mm.

The total length of collection pipeline will be up to approximately 110 km.

Pipe materials of the collection pipeline from wells to collected tank will be polyvinyl chloride pipe (hereinafter referred to as PVC pipe) and/or steel pipe with consideration to maximum inner pressure, required diameter, the span of life and other conditions at the location.

The drawings of the well structure, layout of well field and layout of intake pump house are shown in Figure 13.2-5(1), 13.2-6(1) and 13.2-7 respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of well pumps and collection and the list of pipe materials are shown in Data Book.

(2) Conveyance Facility

i) Pumping Station

The pumping station consists of collected tank, booster pumps and other equipment such as control panels, electric equipment, crane, generator and administration building. Required land area is approximately 2,500 m^2.

Required number of pumping station in the Plan 1 is just one (1) place for conveyance.

ii) Collected Tank

The collected tank stabilizes the water level that is arrived through the collection pipeline for booster pumps in the pumping station. It is divided into two (2) tanks considering the maintenance such as cleaning. Retention time is 0.5 hours with the whole planned development water capacity to assure safe operation of intake pumps and booster pumps. Total capacity of the tank has 1,200 m^3.

iii) Booster Pumps

Total pump number is four (4) including one stand-by. The type of pump selected is the double suction of horizontal volute type based on the following points of view.

a. Condition of the water level between pumps and collected tank
b. Required pump capacity and total head
c. Convenience for checking and maintenance
d. Economical factor (generally, the price is lower than the other type)

iv) Other Equipment in Pumping Station

A generator is installed as an emergency device. The capacity is 600 metric horsepower (PS) which will supply electric power for three (3) booster pumps. A crane, control panel and electric facility will be equipped as the incidental equipment in the pumping station.

v) Surge Tank

When things occur, such as electricity of pumps is cut off, quick operation of the valves installed at outlet of pumps, and from the geographical condition of a location of pipeline, water hammer will happen in the pipeline. In this plan, the capacity and total head of pump is comparatively large and the distance of conveyance pipeline is approximately 70 km. Considering these matters, the prevention of water hammer shall be considered in the design stage. The mitigation method in Plan 1 is installation of the surge tank to prevent the pressure drop.

Therefore, surge tank is required in the plan. The required number and capacity are one (1) and 25 m^3. The surge tank will be located about 6.5 km from the pumping station.

vi) Pressure Reduce Tanks

The pressure reduce tanks is installed to keep within the allowable pressures of the pipe materials. The interval of installation should be less than an altitude of 100 meters considering allowable strength of the pipe material, chosen to be steel pipe which is suited to these conditions. The number and capacity are four (4) tanks and 400 m^3.

vii) Conveyance Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. It is similar to the collection pipeline mentioned above.

The economical velocity in the conveyance pipeline is generally not more than 2.5 to 3.0 m/sec although the conditions at the location and other things should be considered. Therefore, the economical diameter of the conveyance pipeline is determined to be 600 to 900 mm.

The total length of conveyance pipeline will be up to approximately 64 km.

Pipe materials of the conveyance pipeline from pumping station to distribution water reservoir will be steel pipe considering from maximum inner pressure, required diameter, the life span and other conditions at the location.

viii) Distribution Water Reservoir

The distribution water reservoir is located in the suburbs of Ras Sudr city situated on the required altitude for connection to the existing main pipeline or pump sump of the pumping station from Suez city to Abu Rudeis city.

The capacity equivalent to 50 % of the daily average demand is applied. Therefore, the retention time of distribution water reservoir is twelve (12) hours of the water demands.

The total capacity is calculated to be 29,000 m^3, and it is divided into four (4) tanks each with a capacity of 7,250 m^3.

Water level is controlled within regular level by the level controller and/or operation of the pumping station.

The drawings of the pumping station, collected tank, surge tanks, break pressure tanks and distribution water reservoir of the Plan 1 are shown in Figure 13.2-8(1), 13.2-9(1), 13.2-10, 13.2-11 and 13.2-12(1) respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of conveyance facilities such as the booster pumps, the conveyance pipeline, the water hammer analysis of booster pumps and the list of pipe materials are shown in Data Book.

13.2.2 Plan 2: Water Supply to East Coast Side

1) Outline

The intake points of the water sources are located in the southern area of Themed city, and extracted water will be conveyed to Nuwiba and Taba acities. The development water capacity is planned on the assumption with future water demand in 2017 in Nuweiba and Taba cities. The major development conditions for the Plan 2 are described below.

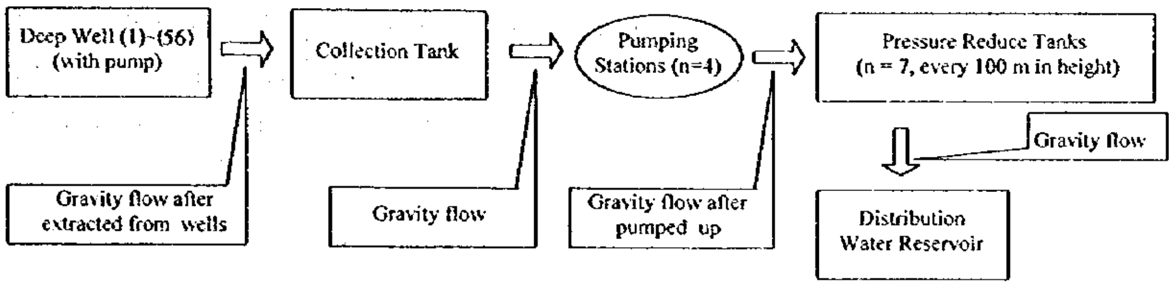
The scope of the facilities plan is conveyance facilities from water intake to the distribution water reservoir located in a suburb of Nuweiba city, through the four (4) pumping stations and seven (7) pressure reduce tanks.

The detailed design conditions and specifications are described in Data Book.

-1	Purpose of Supply	: For the potable water in Nuweiba and Taba cities.
-2	Development Capacity	: 35,000 m^3/day
-3	Target Year	: 2017

2) Water Flow Diagram and Project Location

The water flow diagram from wells to distribution water reservoir is shown below.



Water flow diagram and project layout are shown in 13.2-1(2) and 13.2-2(1) respectively.

3) Water Levels of Facilities and Longitudinal Profile of Conveyance Pipeline

Water levels of all facilities which consists of the intake wells, pumping stations, pressure reduce tanks and distribution water reservoir, are shown in Figure 13.2-3(2).

The longitudinal profile of the conveyance pipeline between No1 pumping station and distribution water reservoir is shown in Figure 13.2-4(2).

4) Development Facilities

The water facilities to be developed in the Plan 2 are classified into intake facility who's water source is groundwater from deep wells and conveyance facility to the distribution water reservoir. The details of each facility are described below.

(1) Intake Facility

i) Required Number of Intake Wells

The safe yield of the single well at the proposed well field is estimated as 720 m^3/day from results of the pump test and actual results in surrounding areas. The design capacity of the water source is 35,000 m^3/day. Therefore, the total number of wells required is fifty-six (56) including eight (8) spare wells for maintenance and emergency.

ii) Influence Radius of Well

"Refer to the same item of Plan 1"

iii) Design of Intake Well

The average depth of wells is set at 1,000 m considering the depths of the aquifer. The diameter of wells will be larger than the test wells. The casing diameter at pump chamber is set at 300 mm, and its length is set at 800 m. The diameter of casing and screen pipes of the other parts is proposed to be 200 mm. The total length will be 1,000 m including sand trap. Above description is same as for the wells planned in Plan 1.

An outline of well construction works is given below although they are same as in Plan 1.

- Well depth : 1,000 m
- Casing size at pump : ϕ300 mm, 800 m long
- Screen size : ϕ200 mm, 150 m long
- Screen type : V slot Johnson type
- Production yield : 720 m^3/day

iv) Total Head and Installed Depth of Well Pump

"Refer to the same item of Plan 1"

v) Collection Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. However, the required total head of submersible pumps becomes higher because of increasing friction losses in the pipe resulting in higher operation and maintenance costs of the facility.

The economical velocity in the collection pipeline is generally not more than 1.5 m/sec although the conditions at the location should be considered. In this case, the economical diameter of the collection pipeline is determined to be 125 to 600 mm.

The total length of collection pipeline will be up to approximately 110 km.

Pipe materials of the collection pipeline from wells to collected tank will be PVC pipe and/or steel pipe with consideration to maximum inner pressure, required diameter, the span of life and other conditions at the location.

The drawings of the well structure, layout of well field and layout of intake pump house are shown in Figure 13.2-5(1), 13.2-6(1) and 13.2-7 respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of well pumps, collection pipeline and the list of pipe materials are shown in Data Book.

(2) Conveyance Facility

i) Pumping Stations

The pumping station consists of collected tank or pump sump, booster pumps and other equipment such as control panels, electric equipment, crane, generator and administration building. Required land area of a pumping station is approximately 2,500 m^2

Required number of pumping stations for boost up the supply water in the Plan 2 is four (4) places.

ii) Collected Tank/ Pump Sump

The collected tank stabilizes the water level which is arrived through the collection pipeline for booster pumps in the No1 pumping station. It is divided into two (2) tanks considering the maintenance such as cleaning. Retention time is 0.5 hours with the whole planned development water capacity to assure a safe operation of intake pumps and booster pumps. Total capacity of the tank has 800 m^3.

Pump sumps is set up at No 2 to No4 pumping stations to assure a safe operation of booster pumps and stabilize the water level. The structure, retention time and capacity of pump sumps are same as for the collected tank.

iii) Booster Pumps

Total pump number to be installed in a pumping station is four (4) including the one stand-by. The type of pump selected is the double suction of horizontal volute type based on the following points of view.

a. Condition of the water level between pumps and collected tank

b. Required pump capacity and total head

c. Convenience for checking and maintenance

d. Economical factor (generally, the price is lower than the other type)

iv) Other Equipment in Pumping Station

A generator is installed as an emergency device. The capacity is 1,500 metric horsepower (PS) which will supply electric power for three (3) booster pumps. A crane, control panel and electric facility will be equipped as the incidental equipment in the pumping station.

v) Surge Tank

When things occur, such as electricity of pumps is cut off, quick operation the valves installed at outlet of pumps, and from the geographical condition of a location of pipeline, water hammer will happen in the pipeline. In this plan, the capacity and total head of pump is comparatively large and the distance of conveyance pipeline is approximately 190 km. Considering these matters, the prevention of water hammer shall be considered in the design stage. The mitigation method in Plan 2 is installation of two (2) surge tanks between No2 and No3 pumping station, and between No.4 pumping station and distribution water reservoir to prevent the pressure drop.

The capacity to be required of a surge tank is 25 m^3, i.e. same as in Plan 1. The surge tanks between No2 and No3 pumping station will be located about 1.5 and 5.2 km from No2 pumping station. Also the other surge tanks between No4 pumping station and distribution water reservoir will be located about 2.5 and 15 km from No4 pumping station.

vi) Pressure Reduce Tanks

The pressure reduce tanks is installed to keep within the allowable pressures of the pipe materials. The interval of installation should be less than an altitude of 100 meters considering allowable strength of the pipe material, chosen to be steel pipe which is suited to these conditions. The required number and capacity are seven (7) tanks and 400 m^3 per tank.

vii) Conveyance Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs, similar to the collection pipeline mentioned above.

The economical velocity in the conveyance pipeline is generally not more than 2.5 to 3.0 m/sec, although the conditions at the location and other things should be considered. Therefore, the economical diameter of the conveyance pipeline is determined to be 200 to 1000 mm.

The total length of conveyance pipeline will be up to approximately 180 km.

Pipe materials of the conveyance pipeline from pumping station to distribution water reservoir will be steel pipe considering maximum inner pressure, required diameter, the life span and other conditions at the location.

viii) Distribution Water Reservoirs

The distribution water reservoirs are located in the suburbs of Nuweiba and Taba cities situated on the required altitude for connection to the distribution pipeline.

The capacity equivalent to 50 % of the daily average demand is applied. Therefore, the retention time of distribution water reservoir is twelve (12) hours of the water demands.

The total capacity for Nuweiba city is calculated to be 15,800 m^3, and it is divided into four (2) tanks each with a capacity of 7,900 m^3. For Taba city, total capacity is 1,500m^3. It is divided into two tanks also.

Water level is controlled within regular level by the level controller and/or operation of the pumping station.

The drawings of the pumping stations, collected tank, surge tanks, break pressure tanks and distribution water reservoir of the Plan 2 are shown in Figure 13.2-8(1), 13.2-9(2), 13.2-10, 13.2-11 and 13.2-12 respectively. And the typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of conveyance facilities such as the booster pumps, the conveyance pipeline and the water hammer analysis of booster pumps and the list of pipe materials are shown in Data Book.

13.2.3 Plan 3: Water Supply to El Tur City

1) Outline

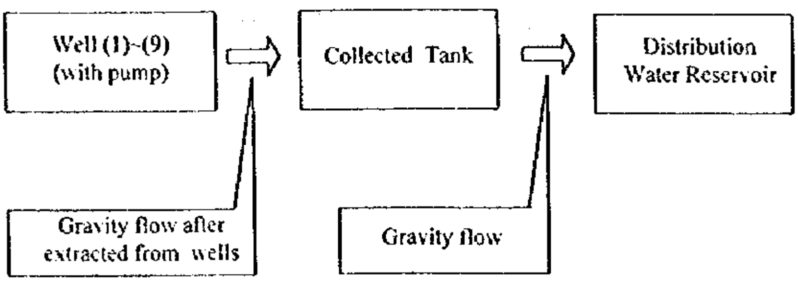
The intake points of the water sources are located in the northern area of El Tur city, and extracted water from the shallow wells will be conveyed to El Tur city. The development water capacity will be planned on the assumption with future water demand in 2007 from the point of view of water balance in the El Qaa Plain.

The major development condition for the Plan 3 is described below. The scope of the facilities plan is the conveyance facilities from water intake to the distribution water reservoir through the conveyance pipeline. To be applicable to the future water demand, after 2007 consider another water source such as River Nile Water with the exception of El Qaa Plain, has to be considered.

-1	Purpose of Supply	:	For the potable water in El Tur city.
-2	Development Capacity	:	5,300 m^3/day
-3	Target Year	:	2007

2) Water Flow Diagram and Project Location

The water flow diagram from wells to receiving water tank is shown below.



Water flow diagram and project layout are shown in Figure 13.2-1(3) and 13.2-2(2) respectively.

3) Water Levels of Facilities and Longitudinal Profile of Conveyance Pipeline

Water levels of all facilities which consists of the intake wells, collected tank and distribution water reservoir, are shown in Figure 13.2-3(3).

The longitudinal profile of conveyance pipeline between collected tank and distribution water reservoir is shown in Figure 13.2-4(3).

4) Development Facilities

The water facilities to be developed in Plan 3 are classified into intake facility who's water source is groundwater to be extracted from shallow wells in El-Qaa Plain and conveyance facility to the distribution water reservoir. The details each facility are described below.

(1) Intake Facility

i) Required Number of Intake Wells

The safe yield of the single well at the proposed well field is estimated as 720 m^3/day from actual results in surrounding areas. The design capacity of the water source is 5,300 m^3/day. Therefore, the total number of wells required is nine (9) including one (1) spare wells for maintenance and emergency.

ii) Design of Intake Well

The average depth of a well is set at 155 m considering the depths of the aquifer. The diameter of well is same as existing wells. The casing diameter at pump chamber is set at 300 mm, and its length is set at 120 m. The diameter of casing and screen pipes of the other parts is proposed to be 300 mm, same as the pump chamber. The total length will be 155 m including sand trap.

An outline of well construction works is mentioned below.

- Well depth : 155 m
- Casing size at pump : ϕ300 mm, 120 m long
- Screen size : ϕ300 mm, 30 m long
- Screen type : V slot Johnson type
- Production yield : 720 m^3/day

iii) Total Head and Installed Depth of Well Pump

The dynamic and static well water levels considering the drawdown are 120 m and 100 m respectively. Therefore, the maximum of total pump head is calculated to be 124 m adding this variation and friction loss of the pipeline to the collected tank. The submersible pumps will be installed at a depth of approximately 130 m. These facilities consist of well, well pump, flow meter and control panel and supplementary facilities installed in the intake pump house.

iv) Collection Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. However, the required total head of submersible pumps becomes higher because of increasing friction losses in the pipe resulting in higher operation and maintenance costs of the facility.

The economical velocity in the collection pipeline is generally not more than 1.5 m/sec although the conditions at the location should be considered. In this case, the economical diameter of the collection pipeline is determined to be 125 to 300 mm.

The total length of collection pipeline will be up to approximately 20 km.

Pipe materials of the collection pipeline from wells to collected tank will be PVC pipe and/or steel pipe with consideration to maximum inner pressure, required diameter, the span of life and other conditions at the location.

The drawings of the well structure, layout of well field and layout of intake pump house are shown in Figure 13.2-5(2), 13.2-6(3) and 13.2-7 respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of well pumps, collection pipeline and the list of pipe materials are shown in Data Book.

(2) Conveyance Facility

i) Collected Tank

The collected tank stabilizes the water level of groundwater that is conveyed through the collection pipeline from intake wells.

It is divided into two (2) tanks considering the maintenance such as cleaning. Retention time is 0.5 hours for the whole planned development water capacity to assure a safe operation of intake pumps. Total capacity of the tank is 120 m^3.

ii) Conveyance Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. it is similar to the collection pipeline which mentioned above.

The economical velocity in the conveyance pipeline is generally for not more than 2.5 to 3.0 m/sec although the conditions at the location and other things should be considered. Therefore, the economical diameter of the conveyance pipeline is determined to be 450 mm.

The total length of conveyance pipeline will be up to approximately 9 km.

Pipe materials of the conveyance pipeline from collected tank to distribution water reservoir will PVC pipe considering maximum inner pressure, required diameter, the life span and other conditions at the location.

iii) Distribution Water Reservoir

The distribution water reservoir is located in the suburbs of El Tur city situated on the required altitude for connection to the distribution pipeline.

The capacity equivalent to 50 % of the daily average demand is applied. Therefore, the retention time of distribution water reservoir is twelve (12) hours of the water demands.

The total capacity is calculated to be 2,800 m^3, and it is divided into two (2) tanks each with a capacity of 1,400 m^3.

Water level is controlled within regular level by the level controller and/or operation of the well pumps.

The drawings of the collected tank and distribution water reservoir of the Plan 3 are shown in Figure 13.2-8(2) and 13.2-12 respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of conveyance pipeline and the list of pipe materials are shown in Data Book.

13.2.4 Plan 4: Water Supply to Agriculture Use

1) Outline

The intake points for the water sources are generally located near the agricultural development area and it is supplied to agriculture land use. The supply water quality in this area should keep the potable water quality standards because the supply water will be used for residents use in rural areas as well as for agriculture use.

The development water capacity is planned in accordance with the future water demand by 2017. The major development conditions for the Plan 4A, 4B and 4C are listed below. In Plan 4, there is a well as the water source and a distribution water reservoir in each agricultural land to be developed.

-1 Purpose of Supply : For the agriculture use and residents use

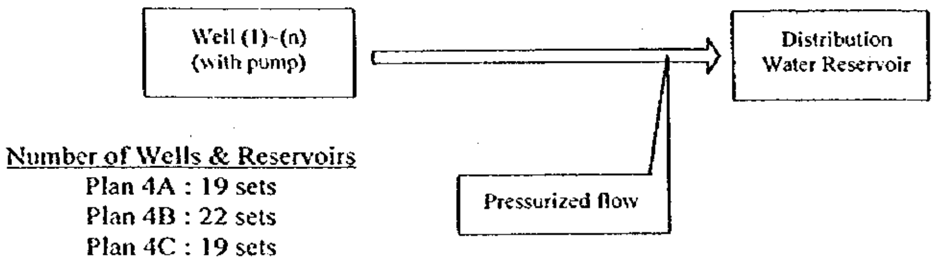
-2 Development Water Capacity of the Plans

Plan 4A	:	Sudr EL Heitan	:	11,700	m^3/day
Plan 4B	:	Malha	:	13,700	m^3/day
Plan 4C	:	Themed	:	11,700	m^3/day

-3 Target Year : 2017

2) Water Flow Diagram and Project Location

The water flow diagram from wells to distribution water reservoir is shown below.



Water flow diagram and project layout are shown in Figure 13.2-1(4) and 13.2-2(1) respectively.

3) Water Levels of Facilities

Water levels of the facility is shown in Figure 13.2-3(4).

4) Development Facilities

The water facilities to be developed in Plan 4 consists of intake facility who's water source is groundwater from deep wells and the distribution water reservoir. Details each facility are described below.

(1) Intake Facility

i) Required Number of Intake Wells

The safe yield of the single well at the proposed well field is estimated as 720 m^3/day from results of the pump test and actual results in surrounding areas. The design water capacity and required number of intake wells are shown in the table blow.

Plan	Number of Intake Wells	
	Total (including standby)	Standby
Plan 4A	19	2
Plan 4B	22	3
Plan 4C	19	2

ii) Influence Radius of Well

"Refer to the same item of Plan 1"

iii) Design of Intake Well

The average depth of wells is set at 1,000 m considering the depth of the aquifer. The diameter of well will be larger than the test wells. The casing diameter at pump chamber is set at 300 mm, and its length is set at 800 m. The diameter of casing and screen pipes of the other parts is proposed to be 200 mm. The total length will be 1,000 m including sand trap. The description is same as for the wells planned for Plan 1.

An outline of well construction works is mentioned below although they are same as in Plan 1.

- Well depth : 1,000 m
- Casing size at pump : ϕ300 mm, 800 m long
- Screen size : ϕ200 mm, 150 m long
- Screen type : V slot Johnson type
- Production yield : 720 m^3/day

iv) Total Head and Installed Depth of Well Pump

"Refer to the same item of Plan 1"

v) Collection Pipeline

The diameter of pipeline should be set as small as possible to facilitate the ease of construction and the economical costs. However, the required total head of submersible pumps becomes higher because of increasing friction losses in the pipe resulting in higher operation and maintenance costs of the facility.

The economical velocity in the collection pipeline is generally not more than 1.5 m/sec although the conditions at the location should be considered. In this case, the economical diameter of the collection pipeline is determined to be 100 to 125 mm.

The total length of collection pipeline of Plan 4A, 4B and 4C will be up to approximately 7 km respectively.

Pipe materials of the collection pipeline from wells to collected tank will be PVC pipe and/or steel pipe with consideration to maximum inner pressure, required diameter, the life span and other conditions at the location.

The drawings of the well structure, layout of well field and layout of intake pump house are shown in Figure 13.2-5(1) and 13.2-7 respectively. The typical drawings of pipe installation works are shown in Figure 13.2-13.

The hydraulic calculation of well pumps, collection pipeline and the list of pipe materials are shown in Data Book.

(2) Conveyance Facility

i) Distribution Water Reservoir

The distribution water reservoir is located under the shadow of intake well.

The capacity equivalent to 50 % of the daily average demand is applied. Therefore, the retention time of distribution water reservoir is twelve (12) hours of the water demands.

The capacity of a reservoir is calculated to be 360 m^3 and the total numbers of them are same as the well numbers in each Plan 4A, 4B and 4C.

The drawings of the distribution water reservoir of the Plan 4 is shown in Figure 13.2-12.

The typical drawings of pipe installation works are shown in Figure 13.2-13.

13.2.5 Plan 5: Water Supply to Bedouin Community

1) Outline

The water source is extraction from dug wells located in the rural area.

The supply water quality of the Plan 5 should keep the potable water quality standards because the supply water will be used to residents use in rural area as well as agriculture use.

The development water capacity is planned on the assumption of 5, 25 and 50 m^3/day.

The major development condition for the Plan 5 is as follows.

-1 Purpose of Supply : For the residents use and agriculture use in rural area.

-2 Development Water Capacity of the Plans

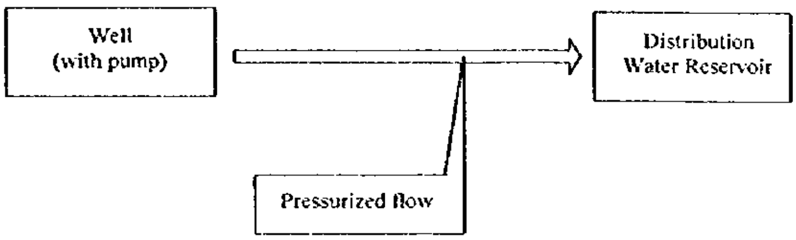
Plan 5A : 5 m^3/day

Plan 5B : 25 m^3/day

Plan 5C : 50 m^3/day

2) Water Flow Diagram

The water flow diagram from well to distribution water reservoir is shown below.



Water flow diagram is shown in Figure 13.2-1(5).

3) Water Levels of Facilities

Water levels of the facility are shown in Figure 13.2-3(5).

5) Development Facilities

The water facilities to be developed in the Plan 5 consist of intake facility who's water source is groundwater from the dug wells, and the distribution water reservoir. Details of each facility are described below.

(1) Intake Facility

i) Required Number of Intake Wells

The safe yield of the single well at the proposed well field is estimated as 5 m^3/day for Plan 5A and 25 m^3/day for Plan 5B and 5C from actual results in surrounding areas. The design water capacity and required number of intake wells are shown in table blow.

Plan	Design Water Capacity (m^3/day)	Number of Intake Wells (Nos)
Plan 5A	5	1
Plan 5B	25	1
Plan 5C	50	2

ii) Design of Intake Well

The average depth of the well is set at 20 m considering the depth of the aquifer. The diameter and depth of well is set at ϕ 2.5 m and 20 m which is same as existing wells.

An outline of well construction works is mentioned.

- Well type : Dug Well
- Well depth : 20 m
- Diameter : 2.5 mϕ
- Production yield : 5 and 25 m^3/day·well

iii) Total Head and Installed Depth of Well Pump

The maximum well depth is 20 m and the top level of distribution water reservoir is 2.5 m from ground level. Therefore, the total head of well pump is calculated to be 26 m adding the friction loss of the pipeline to the distribution water reservoir.

The submersible well pump will be installed at a depth of 20 m. These facilities consists of well, well pump, flow meter, control panel and supplementary facilities installed in the intake pump house.

iv) Collection Pipeline

The economical velocity in the collection pipeline is generally not more than 1.5 m/sec although the conditions at the location should be considered. In this case, the economical diameter of the collection pipeline is determined to be 40 mm for Plan 5A and 80 mm for Plan 5B and 5C. The total length of collection pipeline is about 50 m in each Plan.

Pipe materials of the collection pipeline from wells to distribution water reservoir will be PVC pipe considering maximum inner pressure, required diameter, the life span and other conditions at the location.

The drawings of the well structure is shown in Figure 13.2-5(3).

The hydraulic calculation of well pumps and collection pipeline and the list of pipe materials are shown in Data Book.

(2) Conveyance Facility

i) Distribution Water Reservoir

The distribution water reservoir is located under the shadow of intake well.

The capacity equivalent to 100 % of the daily average demand is applied. Therefore, the retention time of distribution water reservoir is twenty four (24) hours of the water demands.

The capacity of a reservoir is calculated to be 5 m^3 and 25 m^3 for Plan 5A, and 5B, 5C. In Plan 5C, the numbers of it is two (2) reservoirs.

13.3 Management, Operation and Maintenance Plan

13.3.1 Management Plan

The requirement of consumers on the water supply is that, it is satisfactory with regard to the water quantity, quality, and constant supply. In other way, the supplier has to supply consumers with stable and safe water supply. As water is one of the basic human needs which affects preservation and evolution of the society, it is not too much to say that the water supply project is very important for the social welfare. Moreover, the development and/or expansion of water sources which is in accord with the water supply planning for the future water demand, is an essential element for the administrative bodies. The establishment of an impartial water tariff system should be considered to reach early achievements from above points of view.

From above things, administrative bodies for water works shall take into account that it is required of each service area to conduct operation and maintenance of the water works. Administration of these matters must consider the fairness for consumers and unification of the policy.

Important matters for the indices of water control are mentioned below, and establishment and execution of them are expressly matters of weight for South Sinai with insufficient water sources.

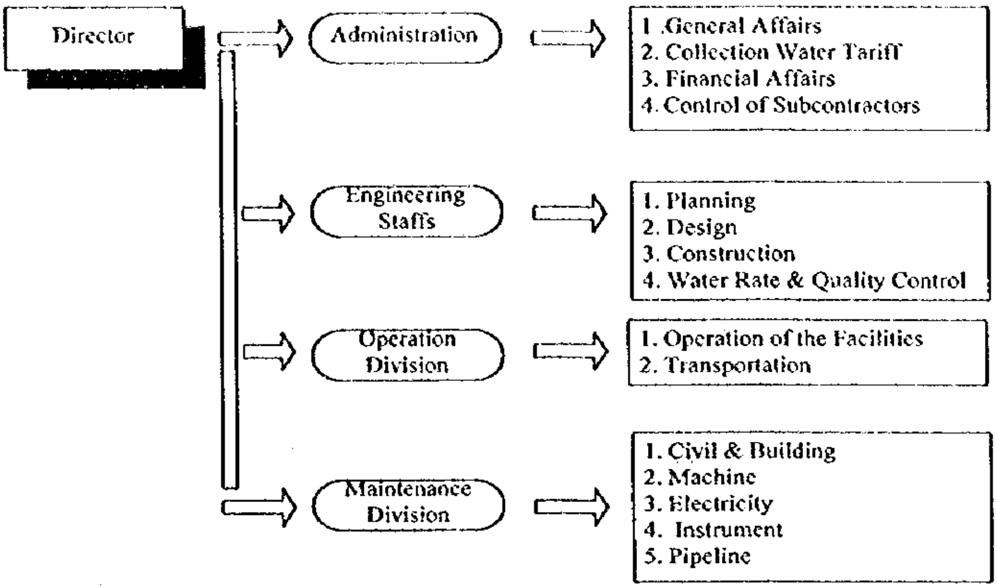
1. Establishment of management organization for municipalities.
2. Establishment of an impartial water tariff system
3. Thorough review of the water quantity and quality control
4. Diffusion of economization concept to increase water consumption awareness and improvement of the water implements
5. Prevention of water sources pollution

13.3.2 Operation and Maintenance Plan

Proper operation and maintenance of water works might be difficult due to lack of water engineers, related facilities and budgets in respective municipalities. However, it is desirable in the future that it is carried out by a unit organization with proper knowledge and experience of water works. Of course central administration such as Governorate and States, should actively participate. The required matters and the outline of an operation and maintenance are given below.

1) Organization and Personnel Plan

As an example, the organization for water control department is shown below. However, personnel required will depend on the scale and technical outline of the facilities. The personnel plan will also be related to the employment situation in the jurisdiction area.



2) Operation and Maintenance of Facilities

The operation of facilities is conducted for the whole water works from intake to distribution facilities. Facilities are usually operated twenty four (24) hours a day. Therefore, the working schedule of operation personnel for the water facilities will be considered in three or four shifts a day in addition to the workers in the daytime.

On the other hand, management of the maintenance system should be composed with personnel from each special field mentioned above. The working system of maintenance personnel is suitable for works in the daytime except the special case such as an emergency measures.

The important matters of daily works which are indispensable for the maintenance works and for which even greater efforts should be made are;

- detectors for water meter and automatic control system.
- disinfection facility for direct influence to human bodies