3.4 Groundwater

3.4.1 Hydrological condition

(1) Climatological characteristic at Doha International Airport

A climatological observation station is located at Doha International Airport and its observation items are as follows:

- Temperature
- Rainfall
- Relative humidity
- Wind speed
- Radiation
- Evaporation
- Others

There are several other rainfall observation stations but their observation data have not sufficient reliability as mentioned in the previous studies. One of the main reasons is the difficulty of maintenance of apparatus caused by the falling of suspended solids in air carried by strong wind.

Among above mentioned items, rainfall and evaporation are most important in the hydrological cycle. Evaporation is closely related to such factors as temperature, wind speed, relative humidity and radiation.

(i) Rainfall

Mean annual rainfall amount during the period of 1962 to 1985 is 75 mm. Mean monthly rainfall amount is more than 10 mm from December to March during the rainy season.

There is almost no rainfall from June to October.

The value of maximum rainfall in 24 hours is very random in its occurrence. Records showing more than 50 mm in 24 hours are as follows:

- May 1963 64.0 mm
- December 1964 80.1 mm
- January 1969 58.0 mm
- March 1982 66.5 mm

Mean monthly rainfall amount (Rtot) and maximum rainfall in 24 hours (Rmax 24) are as shown in Fig. 3.4.1.

1986 rainfall information at Doha International Airport is as follows:

- Jan. 30 to Jan. 31 4.7 mm
- Feb. 6 to Feb. 7 7.4 mm
- Apr. 9 to Apr. 10 37.0 mm

According to the opinion of the Department of Meteorology, rainfall in Doha has particular characteristics, mentioned as follows:

a) Rainfall pattern can be recognized as thunderstorm, having high intensity of rainfall.

b) Rainfall does not occur uniformly. In particular case, there is rainfall in some places while there is no rainfall in the nearby vicinity.

c) As rainfall cloud is usually moved by the NW-SE wind, it is expected that a place located on the same NW-SE line has strong correlationship in rainfall amount.

(ii) Evaporation

Evaporation amount is measured by the following two methods:

a) Open Pond : 1976 to Present
b) Piche Pan : 1977 to Present

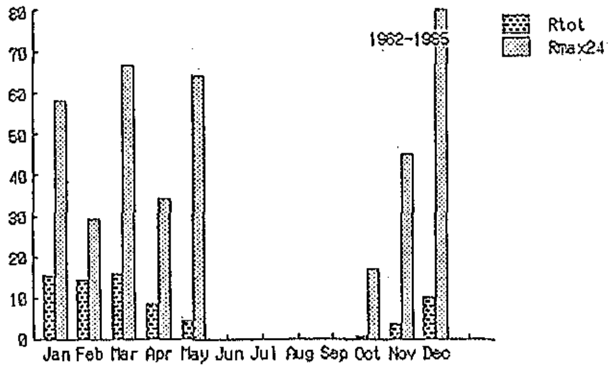
Mean monthly evaporation amount is shown in Fig. 3.4.2. As open pond has much larger surface contacting with atmosphere than Piche pan, evaporation amount observed by open pond is some 60 to 70 percent of Piche pan values. It seems that evaporation amount from the free water table such as standing water is very similar to that of open pond values.

The factors related to evaporation such as temperature, solar radiation, wind speed and humidity are respectively shown in Fig. 3.4.3 to Fig. 3.4.6.

The following can be denoted as evaporation phenomena:

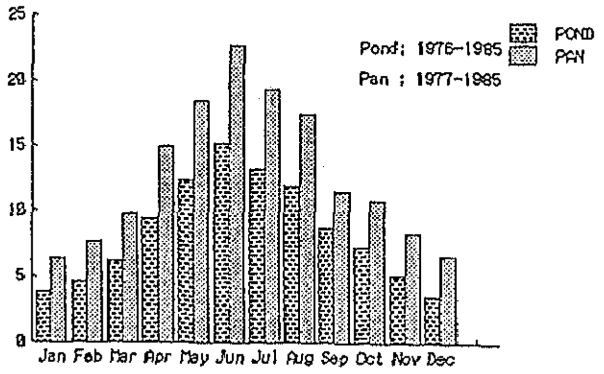
- The most important factor controlling evaporation is solar radiation. Saturated vapor pressure is well known as a function of temperature. When radiation is high, temperature rises and, consequently, evaporation becomes higher. In contrast, when radiation is low, temperatures decrease and evaporation becomes lower.

- Humidity and wind are associated factors governing the contact condition of atmosphere and water. In dry season between May to October, June has most intensive amount of evaporation. This phenomena can be explained not only from highest value of radiation but from lowest value of relative humidity and high frequency of windy days.



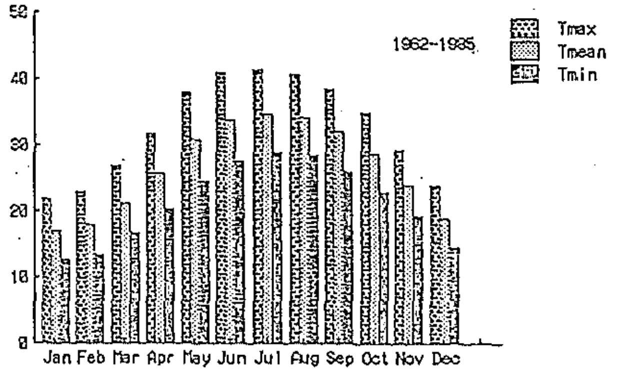
Doha Airport : Unit (mm)

Fig. 3.4.1 Total Rainfall and Rmax24



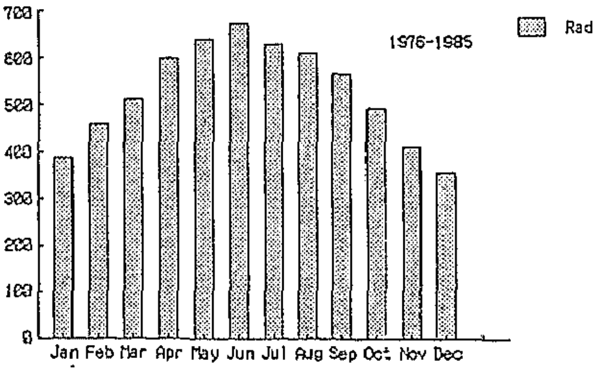
Doha Airport : Unit (mm/day)

Fig. 3.4.2 Monthly Evaporation Amount



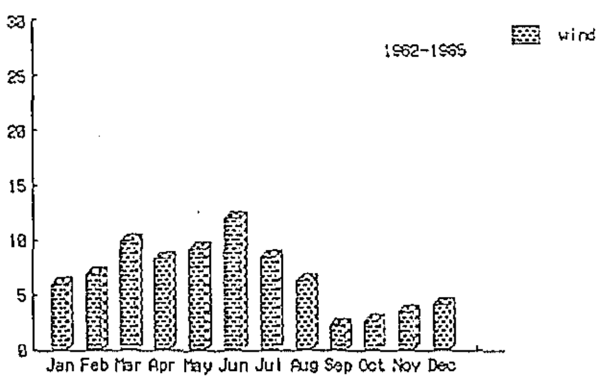
Doha Airport : Unit (°C)

Fig. 3.4.3 Monthly Temperature



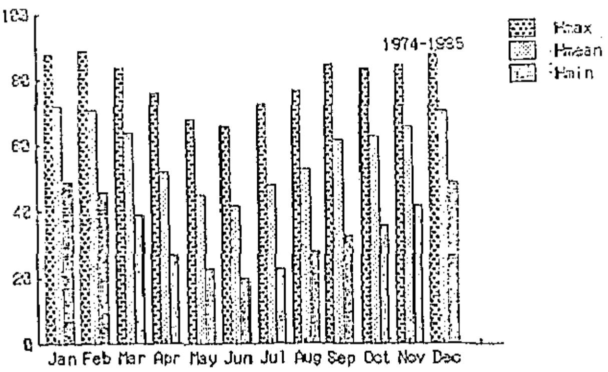
Doha Airport : Unit (mWh/cm2)

Fig. 3.4.4 Mean Daily Global Radiation



Doha Airprt : Unit (day)

Fig. 3.4.5 Wind Speed 20 Knots or More



Doha Airport : Unit (%)

Fig. 3.4.6 Relative Humidity

(2) Meteological Observation at Rayyan

Meteological observation equipment were installed beside the Rayyan Test Work trench to understand the regional characteristics at Rayyan.

The following four equipments were installed for collecting local data during Test Work period at Rayyan site.

- Automatic rain gauge
- Automatic temperature, humidity and barometric pressure recorder
- Automatic wind direction and velocity recorder
- Evaporation pan

The meteorological observation for the following items were carried out from the 1st June to the end of October.

- Temperature
- Humidity
- Barometric pressure
- Rainfall
- Surface wind direction and velocity
- Evaporation

The observation results are summarized by the representative values of each month from June to October 1986 as shown in Table 3.4.1.

With regard to the evaporation which is the most important factor during the Test Work Period, the observation results obtained at Rayyan show nearly the same configuration of variation characteristics as that of Doha Airport.

Table 3.4.1 Monthly Meteorological Characteristics

Month	JUNE	JULY	AUG	SEPT	OCT
MEAN TEMP. (°C)	34.3	34.3	36.2	34.9	31.5
MEAN HUMIDITY (%)	42	51	59	58	56
MEAN WIND SPEED (kts.)	7.7	4.7	3.9	3.9	3.6
TOTAL EVAPORATION (mm/month)	490.1	398.8	312.6	263.5	207.0

3.4.2 Potable Water Supply

Data analysis on the potable water supply amount is indispensable both to interpret the water balance calculation results in the past studies and to well understand the fluctuation characteristics of groundwater level.

Basic data and information on the potable water supply were obtained with the collaboration of Telemetric Center (MEW).

Data analysis procedure is as follows.

1) Arrangement of daily data
2) Calculation of monthly data and correlation
3) Graphic representation

The variation characteristics of the potable water supply amount, as shown in Fig. 3.4.7 can be summarised as follows,

1 Airport Reservoir had an important increase of supply amount from May 1983 to Feb. 1984, corresponding to the start of new supply area. From March 1984, the required additional amount was covered by the three other reservoirs; Old Salwa, New Salwa and Garrafa.

2 Supply system was reinforced and modified from time to time to respond to the water demand. In the recent three years, supply system and operation have become more regular than before.

The variation characteristics of supply amount from these reservoirs between 1984 and 1986 are as follows.

- o Supply amounts from Old Salwa, Airport and West Bay Reservoirs have been almost stable and have had no significant seasonal variation.

- o New Salwa and Garrafa have shown an increasing tendency and slight seasonal variation.

3 Correlation between the monthly inflow amounts of different reservoirs, given in Fig. 3.4.8, started to show positive relationship after the introduciton of the new supply system in March, 1984.

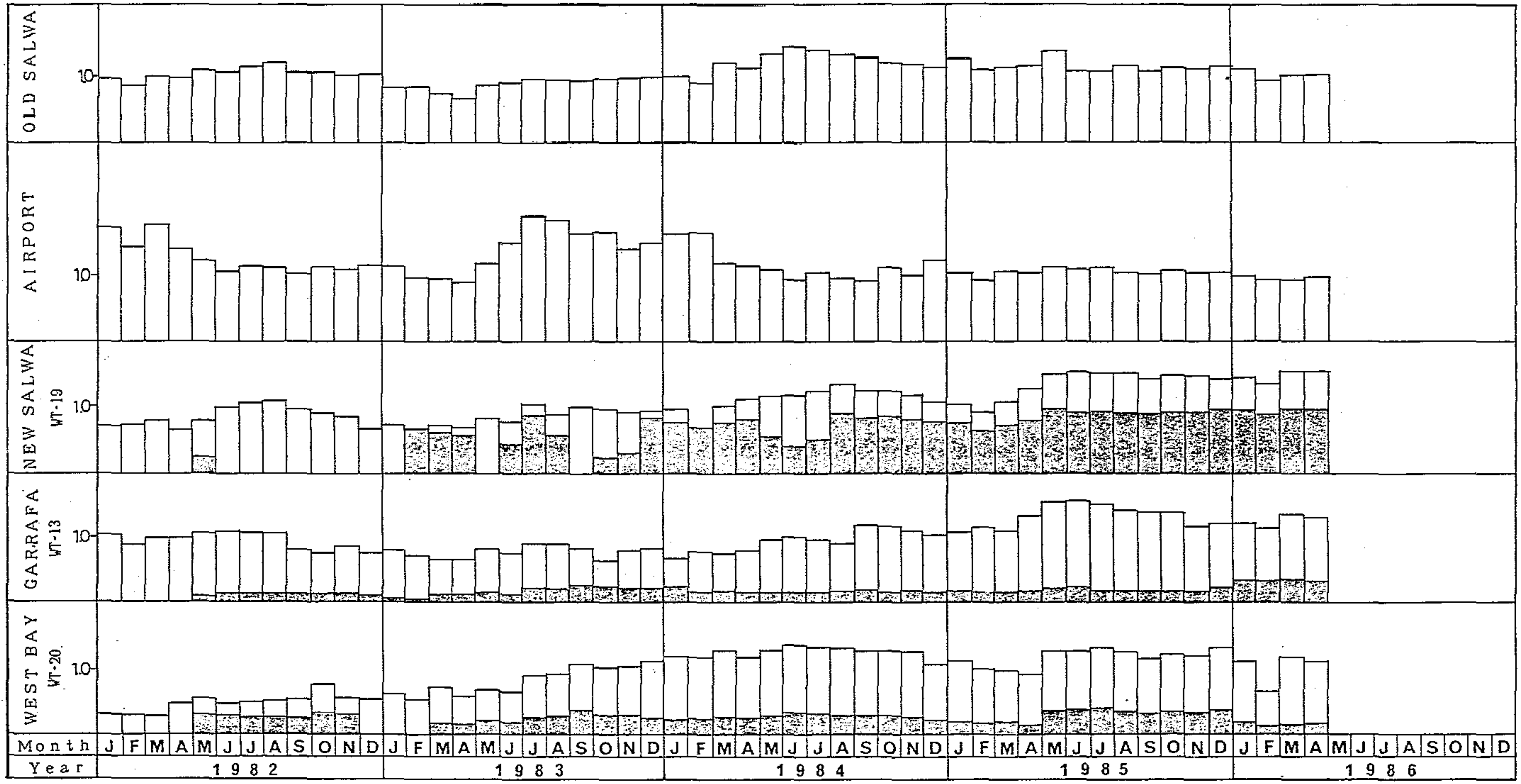


Fig. 3.4.7 Monthly Potable Water Supply Amount

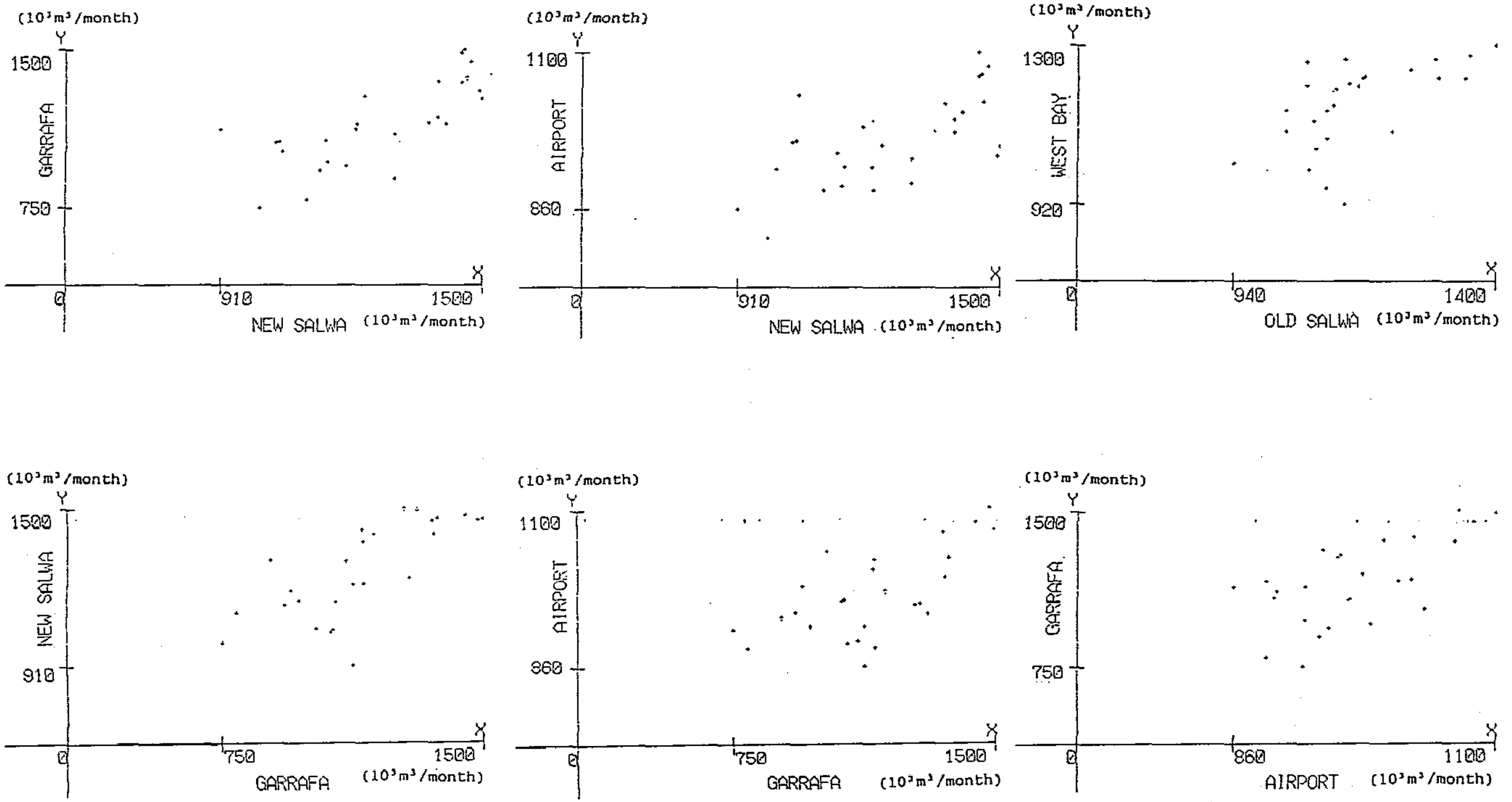


Fig. 3.4.8 Correlationship of Reservoir Inflow Amounts from March 1984 to April 1986

3.4.3 Groundwater Level

(1) Groundwater Level Measurement

A groundwater level measurement plan was established after an inspection of MEW's monitoring wells and the policy of measurement is as follows:

- To understand groundwater level fluctuation behaviors in monitoring wells with cooperation of MEW.

- To understand groundwater rising rate and area based on the ASCO study in 1983.

Groundwater level counter map shown in Fig. 3.4.9 indicates that two groundwater mounds exist as interpretated in the previous study (ASCO), one beneath Khalifah and other Khayl.

Depths in the groundwater map are obtained from the topographic analysis. Area of shallow groundwater depth of less than 2 m below surface can be grouped as shown in Fig. 3.4.10.

In comparison with the previous study (ASCO 1983), the Group "A" area has extended seriously and the largest standing water now exists at Rayyan. The Groups "B", "C" and "D" have nearly the same configuration as they had during the previous study and the Group "E" cannot be compared in detail as it is newly appeared.

(2) Rate of Rising Groundwater Level

The amount of rise of groundwater level during the last three years, from Feb. 1983 to Feb. 1986 is shown in Fig. 3.4.11.

The amount of rise of groundwater level in Rayyan occurs in the range of 1.0 to 1.5 meters during the last three years, and the yearly rate is estimated to be 0.33 to 0.50 m.

In Wadi Musherib and its vicinity, the western and southern areas between the 'C' and 'D' Ring Roads have demonstrated an important rise of 1.0 to 1.5 meter, similar to the Rayyan area.

(3) Groundwater Level Fluctuation

Groundwater level observation results done by the Wellfield Section of MEW were used to grasp the characteristics of groundwater level fluctuation at each well.

Principal graphical representation is shown in Fig. 3.4.12 and the fluctuation characteristics can be summarised as follows.

- o Groundwater level at most wells is rising very gently and seasonal variation is only noticable in a few wells.

- o Rainfall impact to the groundwater table is thought to be local and particular and not to be the principal cause of the recent rising groundwater level.

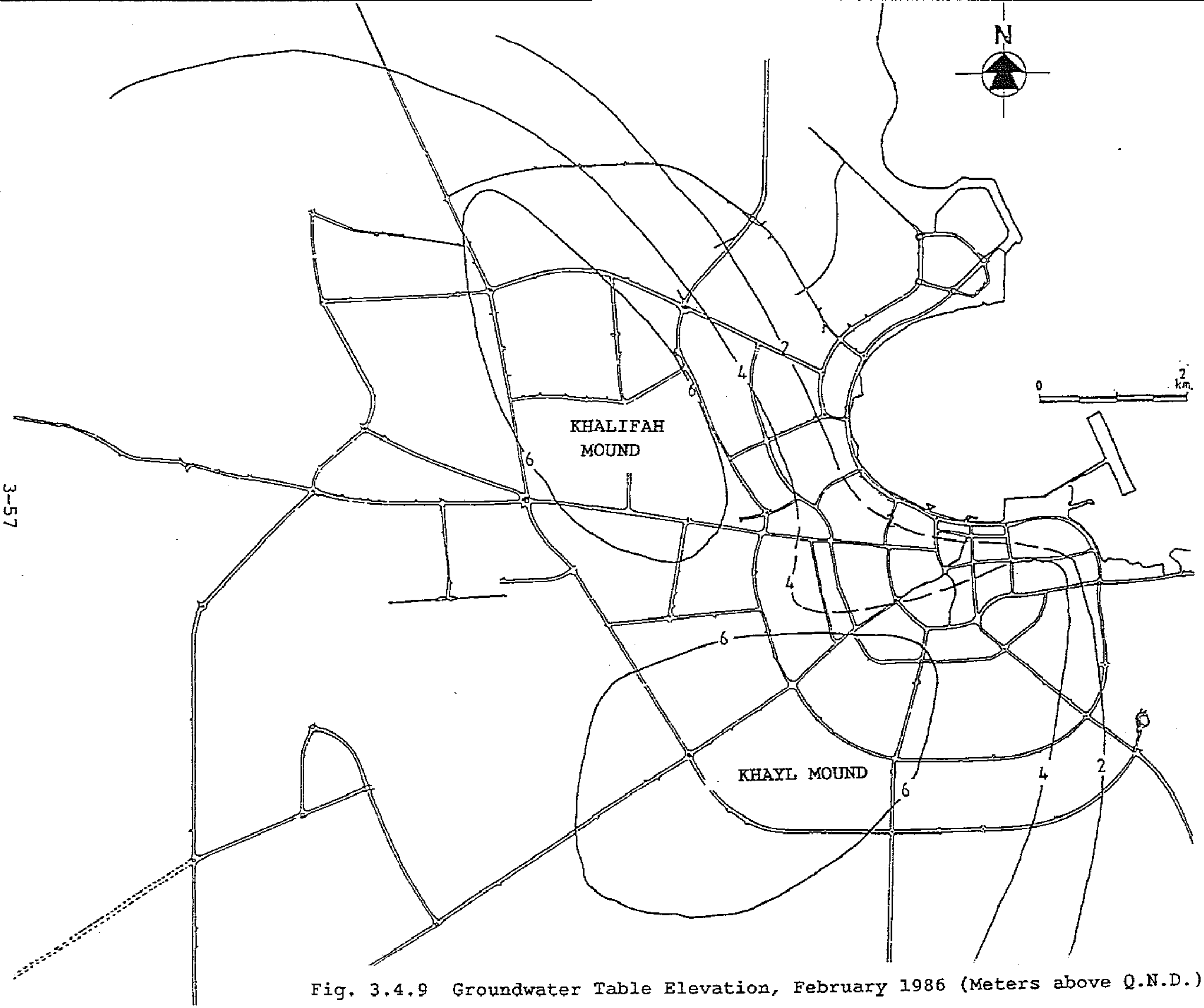


Fig. 3.4.9 Groundwater Table Elevation, February 1986 (Meters above Q.N.D.)

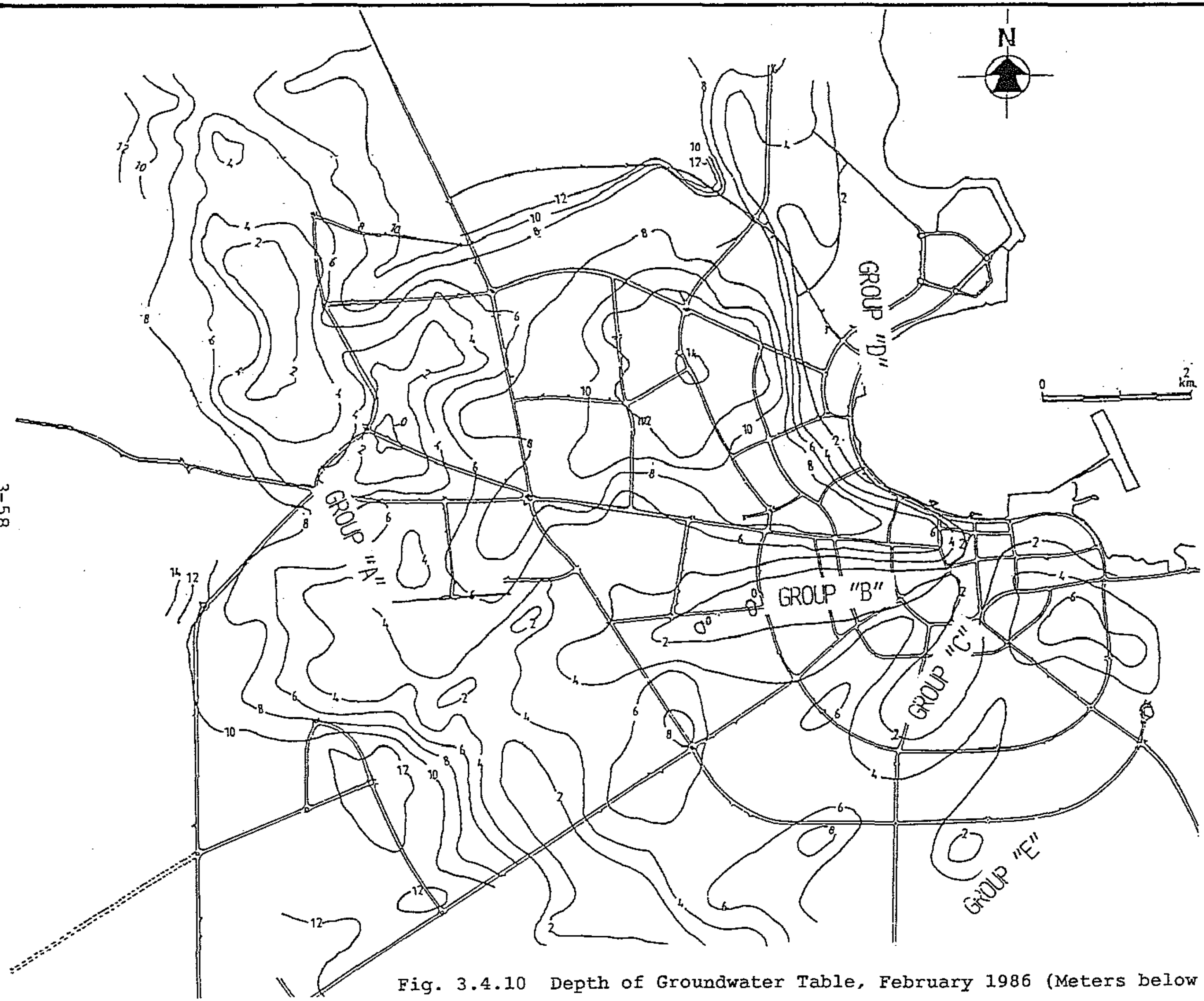


Fig. 3.4.10 Depth of Groundwater Table, February 1986 (Meters below G.L.)

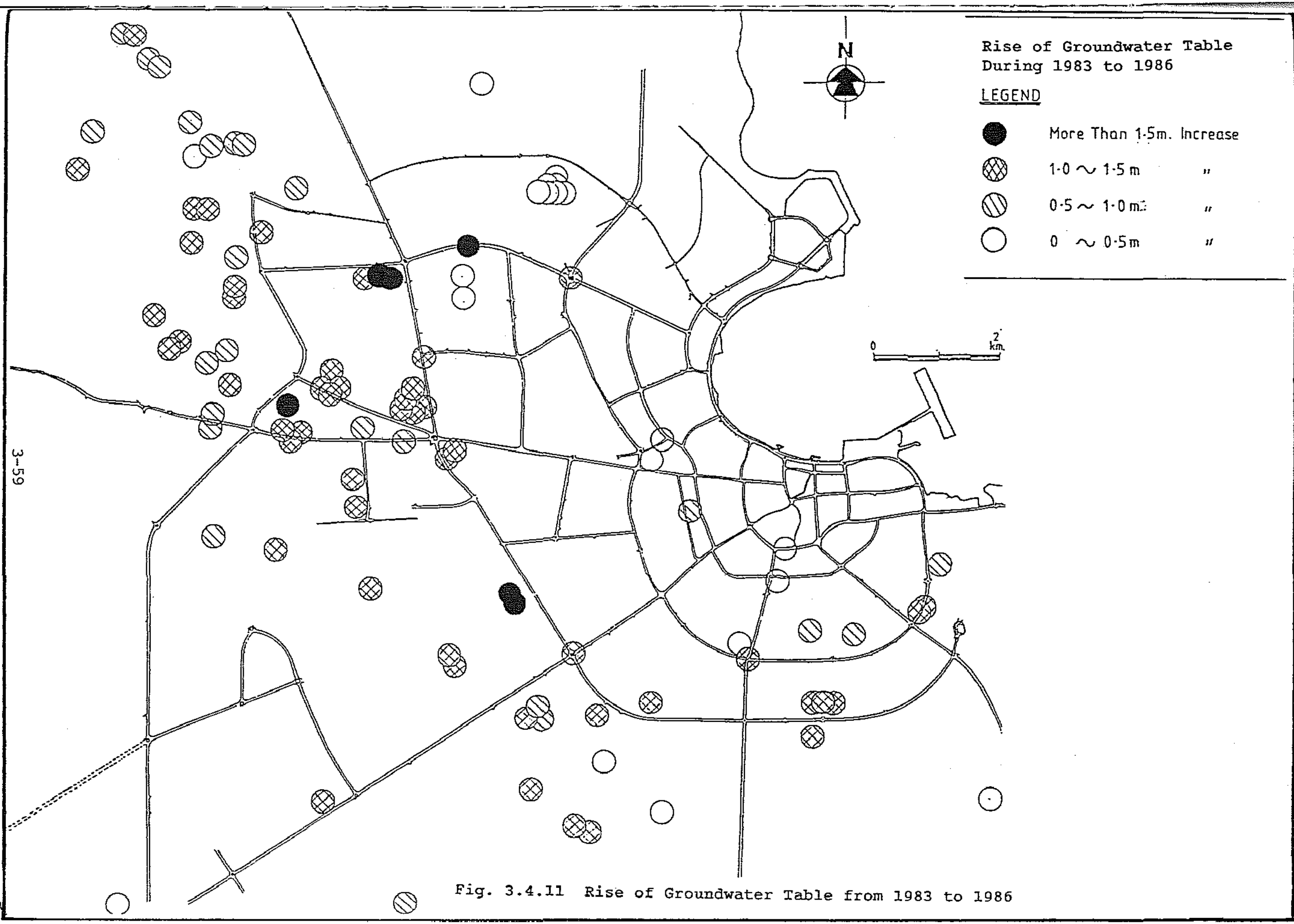
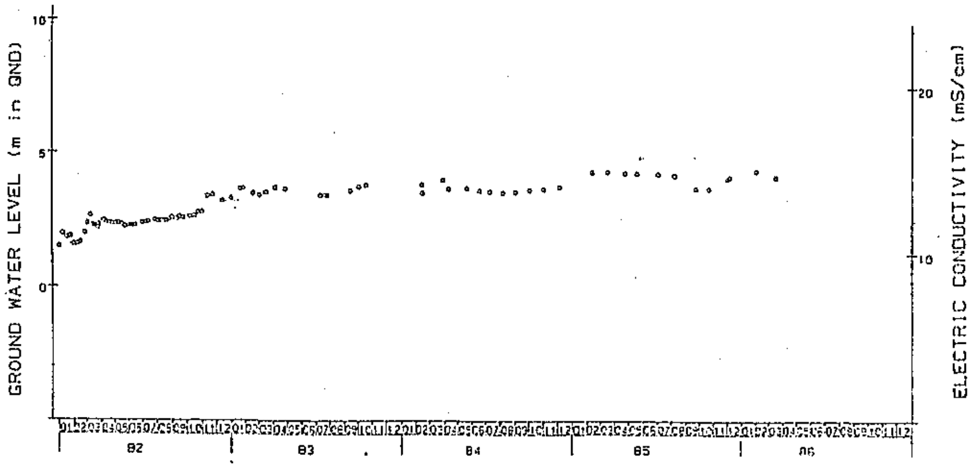
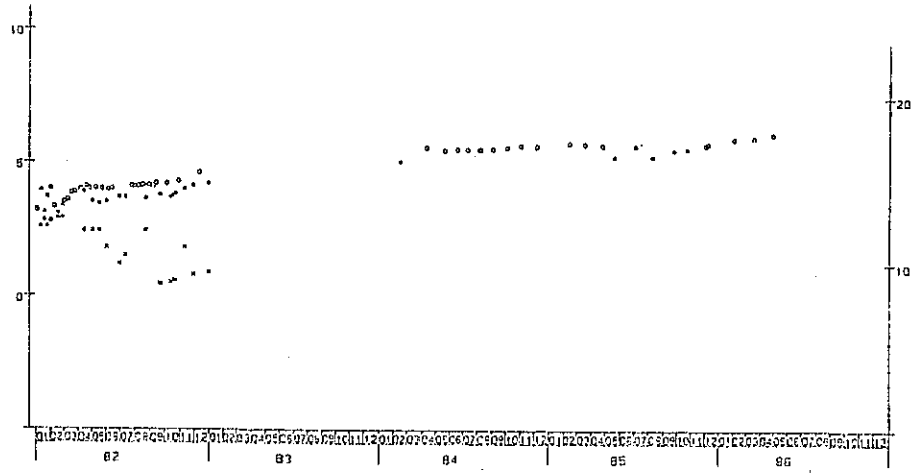


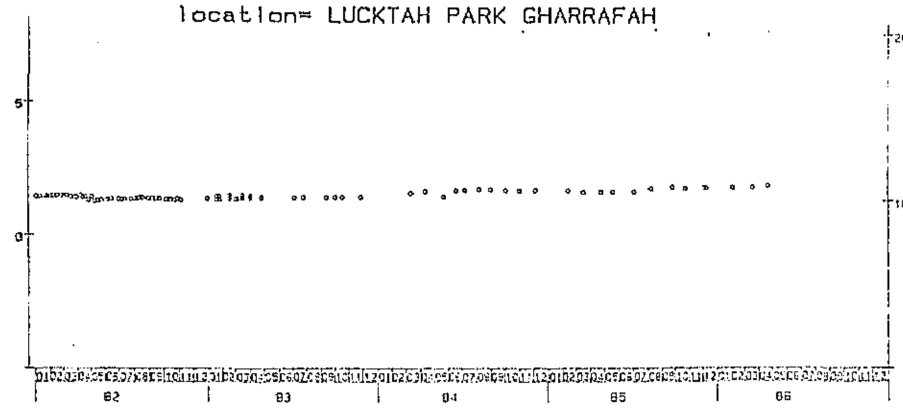
Fig. 3.4.11 Rise of Groundwater Table from 1983 to 1986



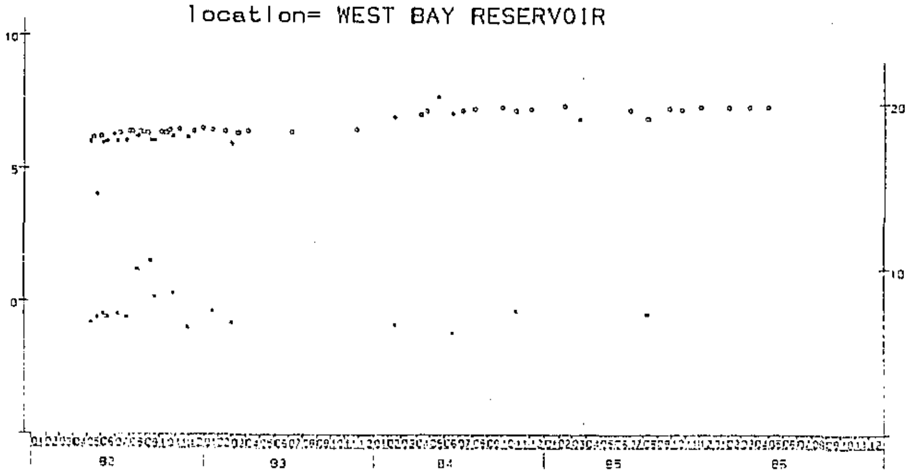
grid ref= 23709590 well number= 2239-GRF
location= GHARRAFAH RESERVOIR



grid ref= 25219610 well number= 2239-44
location= LUCKTAH PARK GHARRAFAH

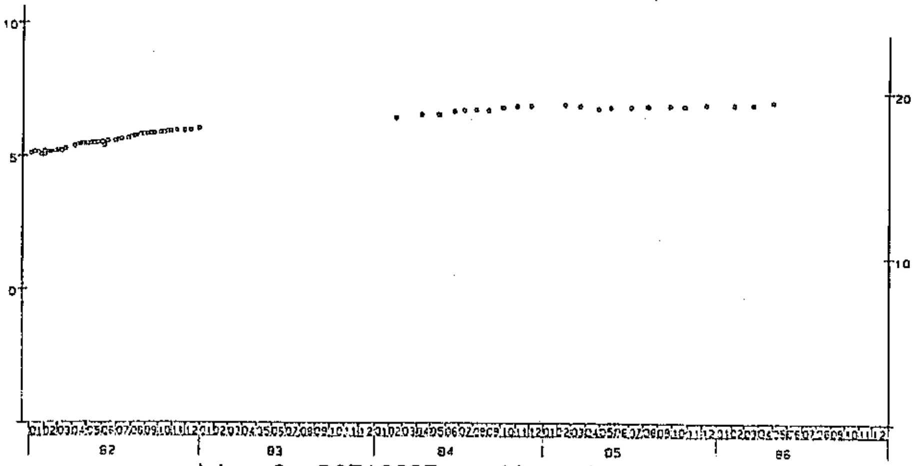


grid ref= 26819946 well number= 2239-121
location= WEST BAY RESERVOIR

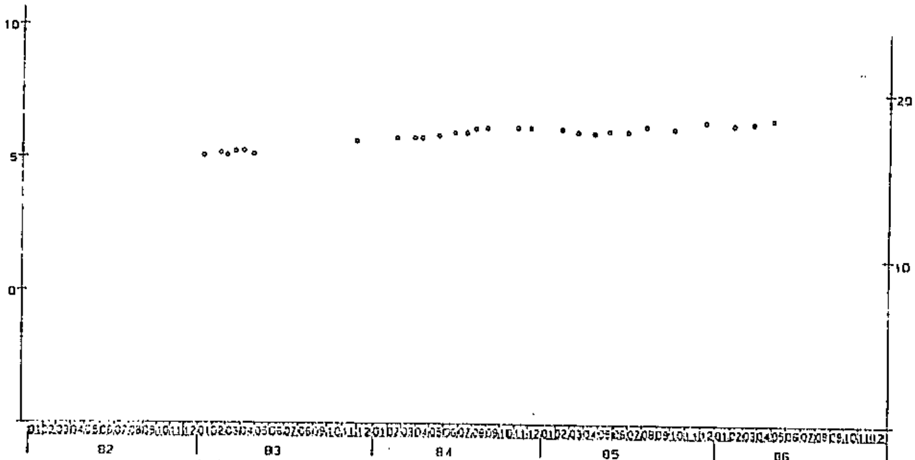


grid ref= 28309013 well number= 2239-41
location= FOOD CENTER ROUNDABOUT

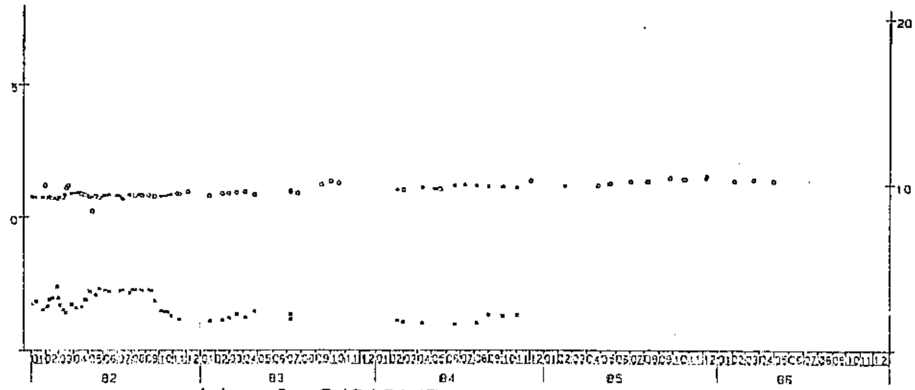
Fig. 3.4.12 Graphical Representation of Groundwater Level Fluctuation



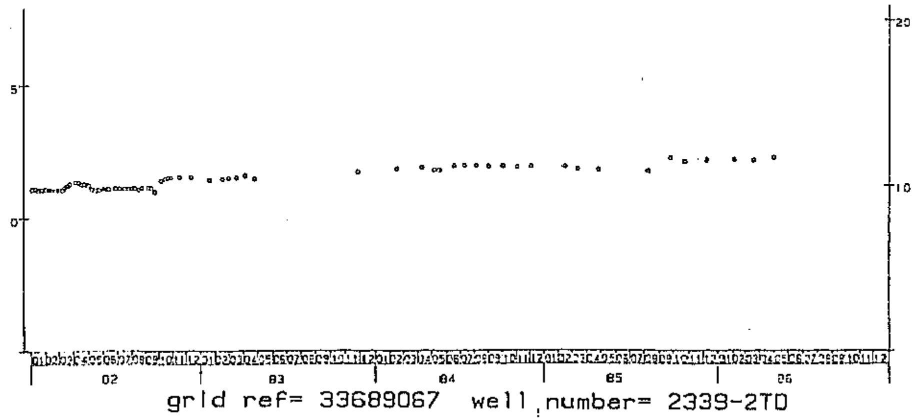
grid ref= 29718935 well number= 2238-41
location= D RING ROAD FARM



grid ref= 31069021 well number= 2239-63
location= MONTAZA PARK OLD B/H



grid ref= 34219147 well number= 2339-26
location= ALR PORT RESERVOIR



grid ref= 33689067 well number= 2339-2TD
location= ALMANA ROUNDABOUT

3.4.4 Numerical Analysis for Groundwater Level Simulation

(1) Objectives

The objectives of numerical analysis are first to well understand the rising groundwater behavior and second to obtain the basic information on drainage amount necessary to lower the groundwater level in the subject areas.

(2) Selection of Model

The hydrogeological studies developed upto the present are related to the characteristics of the shallow aquifer which is the direct cause of damage. In comparison, the information concerning deeper aquifers is very limited.

The applicability of the different simulation models is shown in Table 3.4.2. Quasi-three dimension model is the most adequate model to analyse the recent rising groundwater level and the other models are not suitable in this regard due to the insufficient basic hydraulic and hydrogeological information.

Table 3.4.2 Comparison of Simulation Models

	Quasi-Three Dimension Model	Multi-Aquifer Model	Three Dimension Model
Advantage	Multi-layers' horizontal flow can be represented and application is easy.	Leakage through aquifers is well represented.	Most detailed representation of groundwater phenomena is possible.
Disadvantage	Vertical flow element cannot be represented.	Accurate data for deep aquifers is necessary.	Node points are too many and accurate data supply is very difficult. Computing cost is expensive.

(3) Mathematical Basis

i) Governing Equation

The governing equation of quasi-three dimensional plane model is derived form the continuity equation and dynamic equation according to the Dupuit-Forchheimer's assumption in which the vertical flow element is negligible.

The *governing equation can be expressed as follows;*

$$\nabla\cdot(T\nabla h) = S \frac{\partial h}{\partial t} + q \quad .... \quad (3.4.1)$$

where,

- T: Transmissivity
- h: Hydraulic Head
- S: Storage Coefficient
- t: Time
- q: Source Term

As the groundwater level varies in time, transmissivity for corresponding groundwater level is calculated by the sum of the product of permeability coefficient and saturate thickness of each aquifer.

$$T = \sum_{i} Ki \times Di$$

where,

- K: Permeability Coefficient
- D: Saturate Thickness
- i: Subscript of the i th Aquifer

ii) Concept of Source Term

The following parameters effect the source term evaluation.

1 Groundwater evaporation varying with groundwater level
2 Groundwater extraction
3 Groundwater recharge
4 Local leakage through fractures and/or, into water vein system such as vugs, sewerage etc.

The schematic representation of each parameter is shown in Fig. 3.4.13 and the source term can be expressed by the following equation.

$$q = Ev + Ex + Lt - \varepsilon \quad .... \quad (3.4.3)$$

where;

- Ev : Groundwater Evaporation Amount
- Ex : Groundwater Extraction Amount
- Lt : Total Leakage Amount
- ε : Groundwater Recharge Amount

Ev is calculated by the function of groundwater depth from surface. Ex and ε are estimated from the land use conditions and the previous study result of water balance. Lt almost corresponds to the inferred recharge.

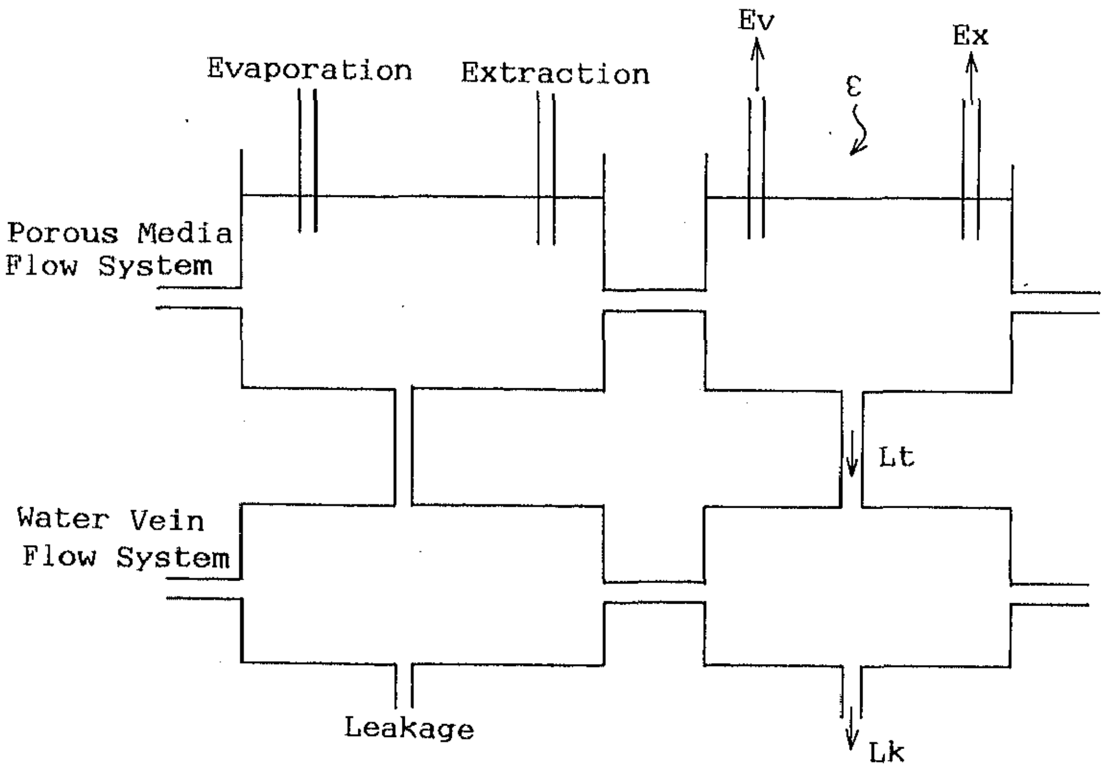


Fig. 3.4.13 Schematic Representation of Source Term

iii) Initial and Boundary Conditions

Initial condition is given by the groundwater level contour map of 1983. Boundary condition is given by the linear interpolation of groundwater level of 1983 to 1986 level.

- Initial Condition
  $h(xi,o) = h(xi)$ .... (3.4.4)
- Boundary Condition
  $h(xi,t) = h_{\Gamma}(xi,t)$ .... (3.4.5)

where, i : Subscript which expresses the coordinates
Γ : Subscript which expresses the boundary of calculation domain.

iv) Calculation Method

The calculation method used is the finite element method, introducing the following trial function;

$$h(xi,t) = Nn(xi)\ hn(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4.6)$$
$$(n = 1, 2, \ldots\ldots\ldots)$$

where, Nn(xi) : Function Determined by the coordinates
hn(t) : Unknown Parameter in Function of Time

The residual (R) expressed by Eq 3.4.7 which is obtained by the substitution of Eq. 3.4.6 into Eq. 3.4.1 is minimised by the trial calculation in each time step to satisfy the convergency condition.

$$\nabla \cdot T\nabla(Nn(xi)hn(t)) - S\frac{\partial}{\partial t}(Nn(xi)hn(t)) - qn(t)Nn(xi) = R \quad \ldots\ldots \quad (3.4.7)$$

v) Nodes and Elements

The calculation domains for Wadi Musherib and Rayyan were determined to sufficiently cover the respective influence areas which were affected by the drawdown of the lateral drainage. Their grid systems are shown in Fig. 3.4.14 and 3.4.15.

(4) Data Preparation

The following materials were used to prepare the input data:

1 Source Term
- Land use classification map
- Groundwater recharge amount by QATS (ASCO)
- Evaporation amount

2 Aquifer Thickness

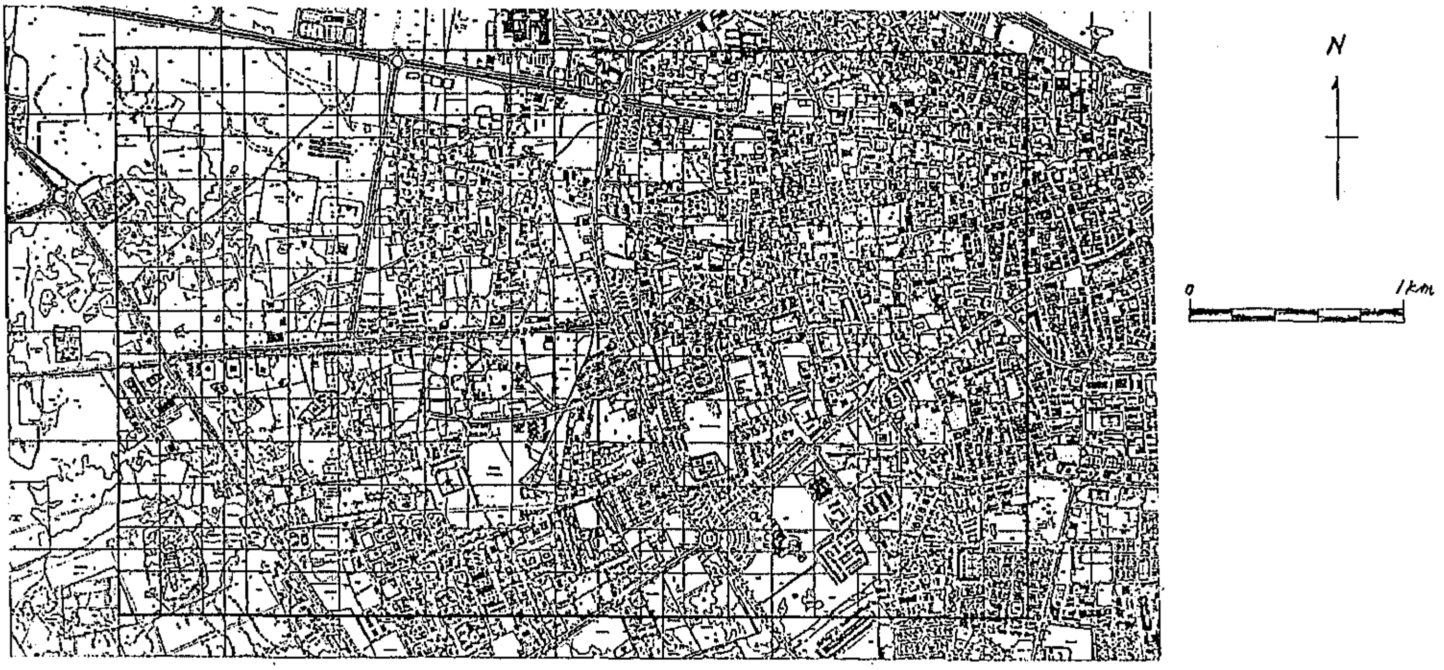
- Topographical map (1:5000)
- Contour map of weak weathered layer top
- Contour map of lower Dammam formation top
- Contour map of Rus Formation top

3 Hydraulic Constants

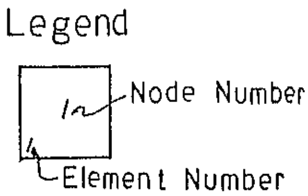
- Calculation results of steady model (ASCO)
- Lugeon test
- Pumping test results at Test Works

4 Groundwater Level

- 1983 Groundwater level contour map (ASCO)
- 1986 Groundwater level contour map



Key Map



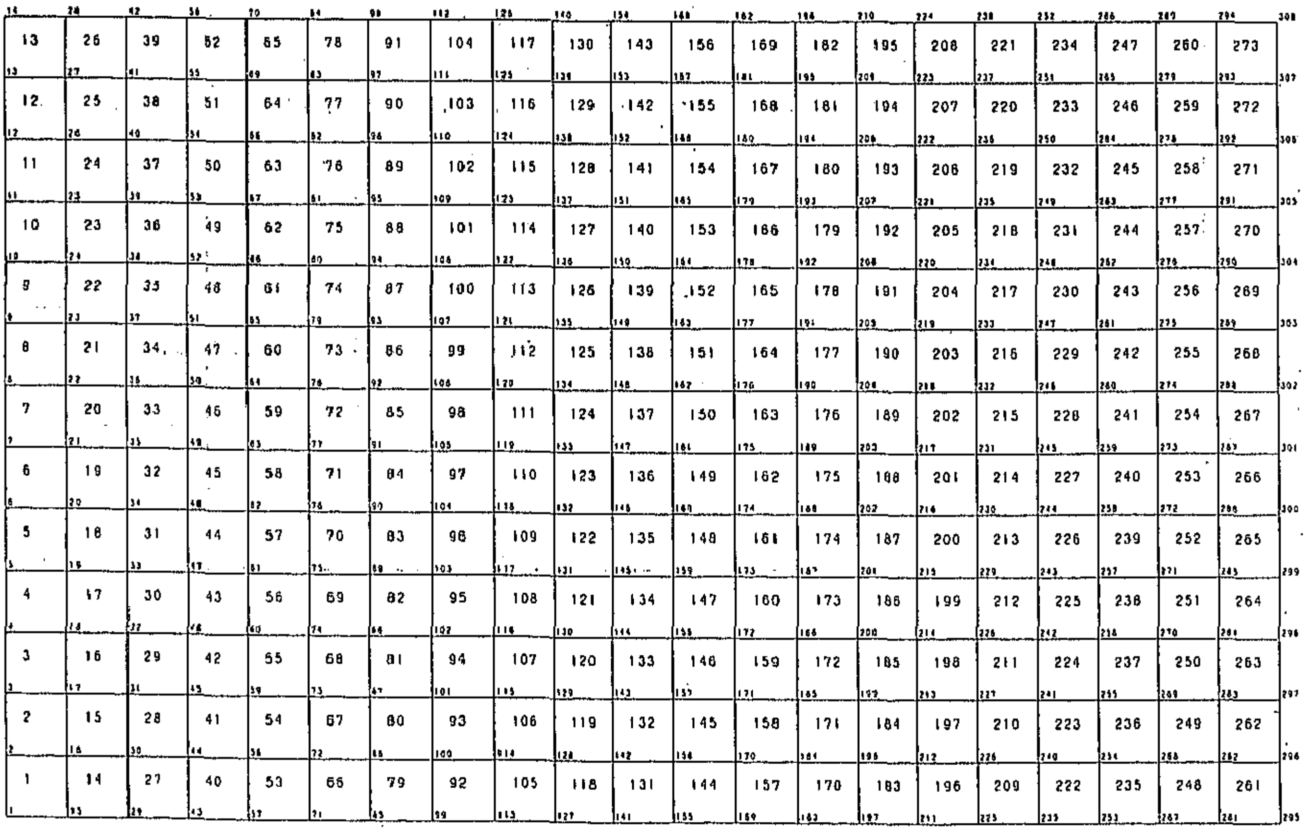


Fig. 3.4.14 Grid System of Quasi Three Dimension Analysis for Wadi Musherib

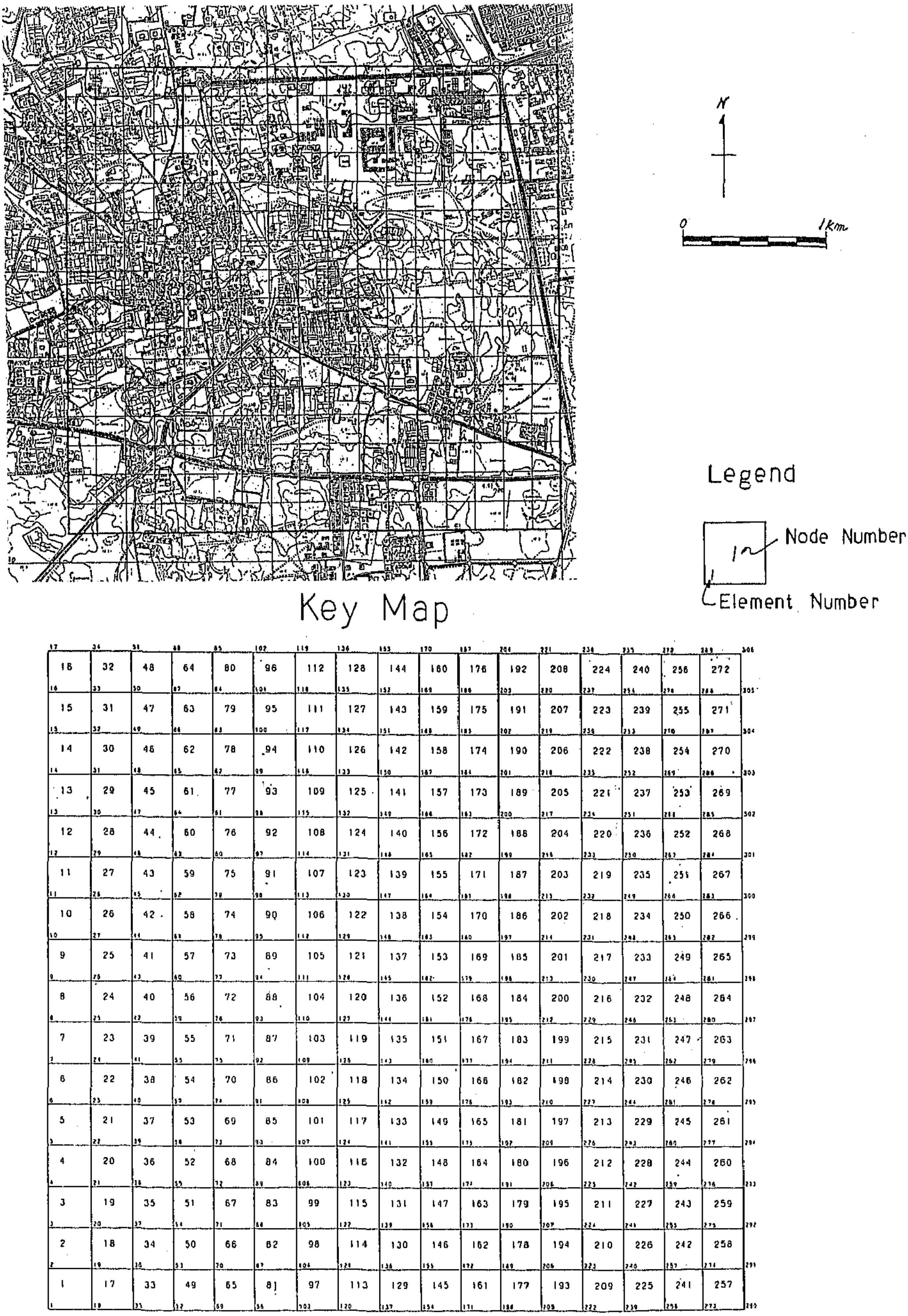


Fig. 3.4.15 Grid System of Quasi Three Dimension Analysis for Rayyan

(5) Parameter Identification Process

The following items were considered in the parameter identification process.

1 Simulation of rising groundwater level behavior
2 Simulation of expected discharge amounts near the Test work sites

Collection and modification were mainly made on the source term by the trial and error method.

(6) Calculation Results

Simulation of groundwater level from 1983 to 1986 for Wadi Musherib is shown in Fig.3.4.16. That of Rayyan is shown in Fig. 3.4.17.

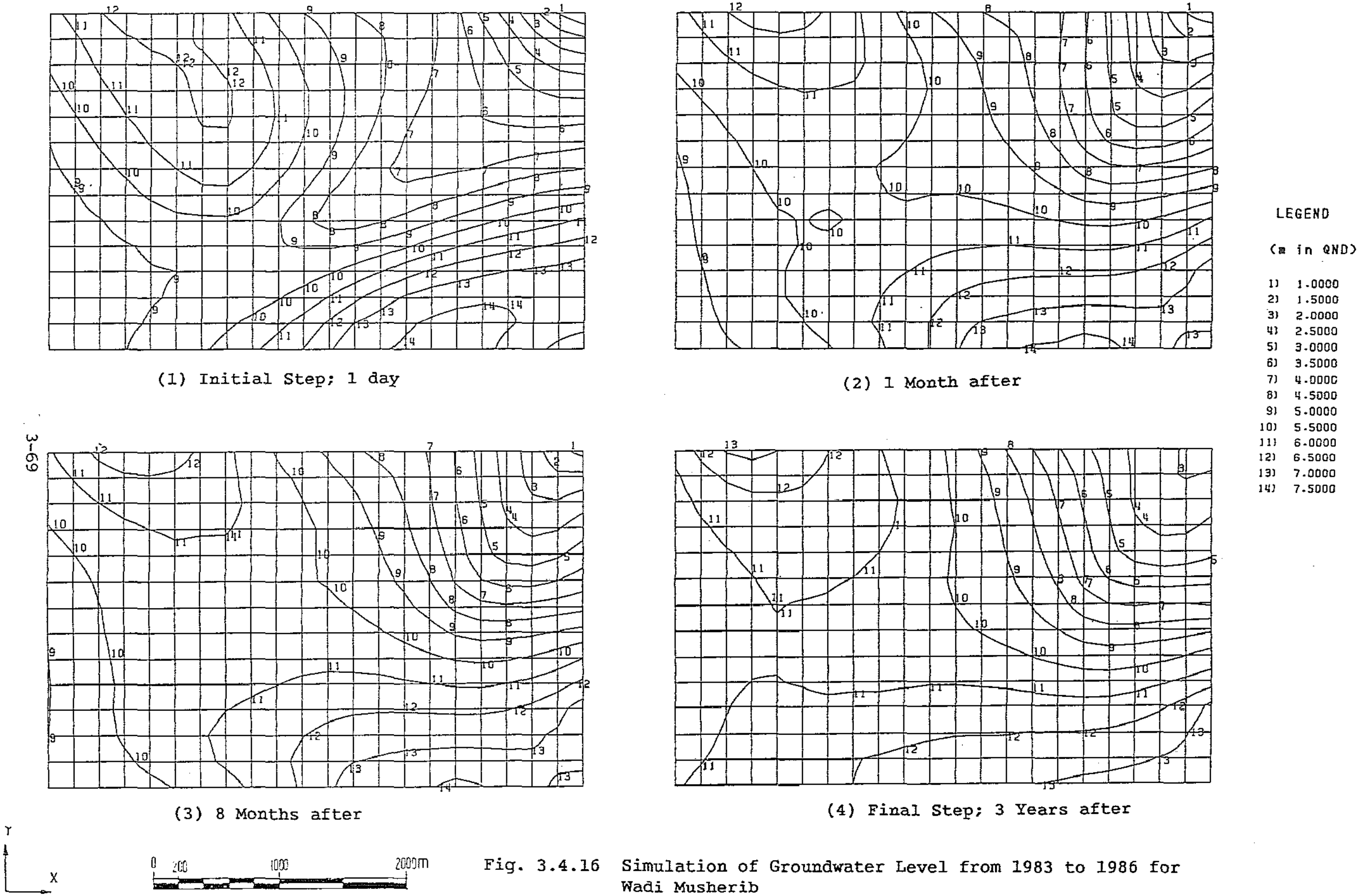
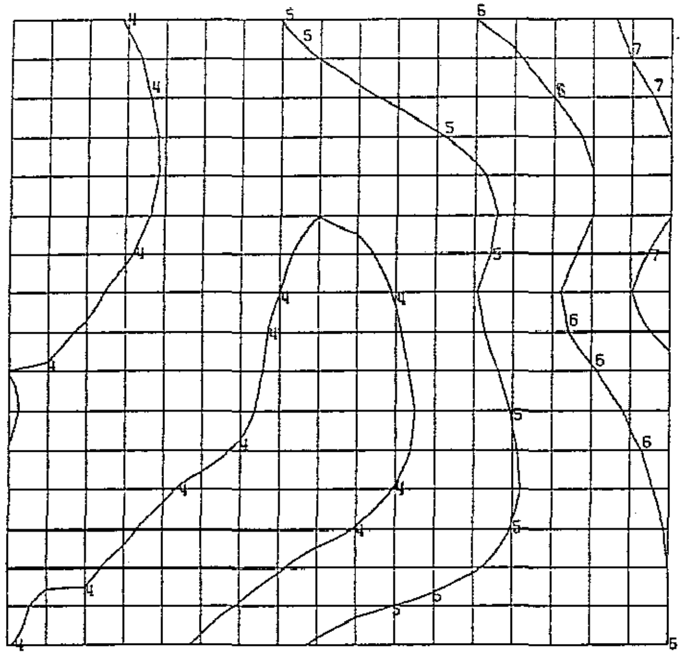
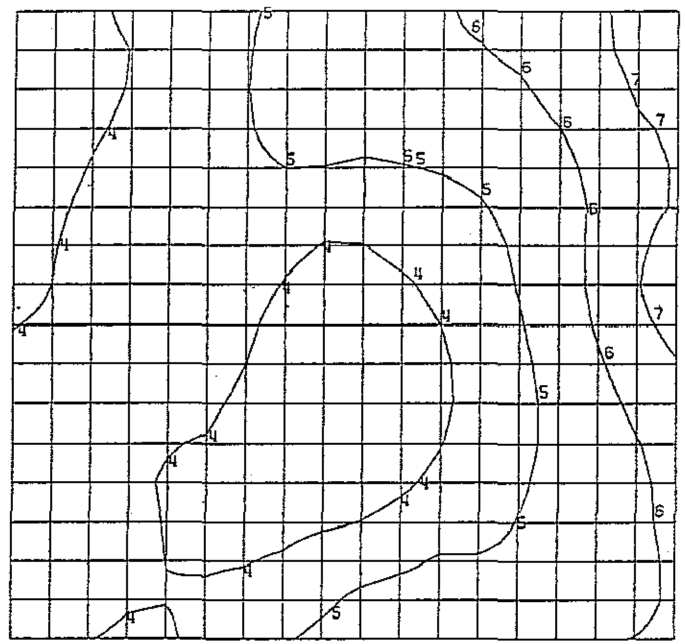


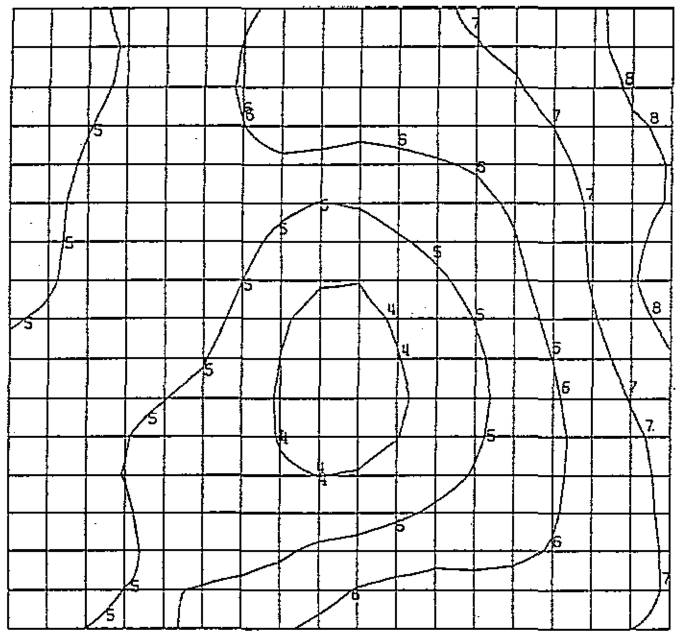
Fig. 3.4.16 Simulation of Groundwater Level from 1983 to 1986 for Wadi Musherib



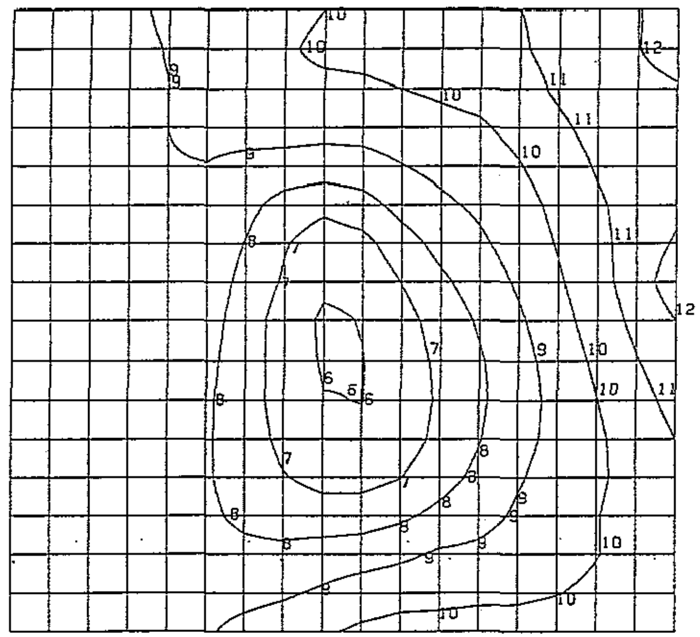
(1) Initial Step; 1 day



(2) 1 Month after



(3) 8 Months after



(4) Final Step; 3 Years after

LEGEND

(m in QND)

1) 3.5000
2) 3.7500
3) 4.0000
4) 4.2500
5) 4.5000
6) 4.7500
7) 5.0000
8) 5.2500
9) 5.5000
10) 5.7500
11) 6.0000
12) 6.2500
13) 6.5000

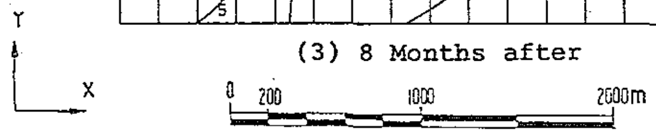


Fig. 3.4.17 Simulation of Groundwater Level from 1983 to 1986 for Rayyan

3.5 Water Quality

In order to develop solutions against the problems caused by the rise of groundwater level, the water quality survey was carried out with the following specific objectives.

1) To grasp the general characteristics of groundwater quality in the three study areas, namely Doha city, Rayyan and New District areas.

2) To know the change of groundwater quality by depth in relation to the geological formation in the areas.

3) To collect basic data on the reuse and treatment of groundwater. (refer to Section 6.6)

4) To grasp the present conditions of pollution of the groundwater and seawater in order to examine the possibility of disposal of the abstracted groundwater into the sea.

5) To know the quality fluctuation in groundwater during the continuous pumping-up of the groundwater from the trenches excavated at Wadi Musherib and Rayyan test work sites.

3.5.1 Method of Water Analysis

The water quality tests performed directly by the JICA Study Team were carried out using simple equipment on samples of limited quantity. In order to minimize the error of measurement, samples were also analysed at the laboratory of Doha South Sewage Treatment Works. The analyses in the laboratory were made based on the Standard Methods for Water Quality Analysis by Am. Water Works Association.

(1) Equipment for analyses

The following equipment were used for the water quality tests by the Study Team.

a) Field analysis kit attached with portable photoelectric spectrophotometer (Model, DREL/5)
b) Potable COD Meter (Model, HC-307)
c) Potable pH Meter
d) Electric Conductivity Meter (Model, 30ET)

(2) Analyses items

As the general items of the water quality analysis, the values of Electric Conductivity (EC), pH, cations of major metals (Ca, Mg, Na and K) and anions (CO_3, SO_4, Cl, NO_3, PO_4 and B,F,I) were measured. In addition to the above, the values of COD, BOD, PO_4 and Total Sulphide in the sampled water were also measured to know the degree of pollution. Items selected for water quality analyses are as shown in Table 3.5.1.

Table 3.5.1 Selected Items for Water Quality Analyses

Test Items	Unit	ASCO	JICA Study Team	JICA Doha South Lab.
EC	us/cm	o	o	o
pH		o	o	o
Major Ions: Cations				
Ca	mg/l	o (as Ca)	o CaH (as $CaCO_3$)	o (as $CaCO_3$)
Hg	"	o (as Mg)	o MgH (as $CaCO_3$) o Total H (as $CaCO_3$)	o (")
Na	"	o		o
K	"	o		o
Anions				
HCO_3	"	o (as HCO^3)	oT-Alk (as $CaCO_3$)	
SO_4	"	o		o
Cl	"	o (as Cl)	o	o
NO_3	"	o (as NO_3)	o NO_3-N (as N)	
PO_4		o		
B,F,I		o		
Items Suggesting Pollution				
COD ($KMnO_3$)	mg/l		o (as O)	o
BOD	"			o
PO_4	"		o (as O-PO_4)	
Total-Sulphide	"			o (as S)

(3) Sampling sites

Water for the quality test was sampled from the ASCO project wells, wells of irrigation, excavated site and others. The total number of sampling locations is 82.

a) ASCO project wells: 16 sites

2239 - 1PD, 3TD, 5PD, 7PD, 9PD, 11PD
2339 - 2TR, 3PD, 4PD, 6PD, 7PR, 9PR, 10PD, 11TR, 12PR, 13PD

b) Doha city area: 36 sites

Water was sampled mainly from the inner side of the 'C' Ring Road.

c) Rayyan area: 26 sites

Sampling was mainly carried out from wells for irrigation in the farms scattered in the area.

d) New District: 4 sites

In this area, there are only a few wells where sampling can be performed; therefore, the number of samples was restricted.

3.5.2 Results of the Analyses

From the above locations a total of 136 samples were analysed, 93 samples by the JICA Study Team and the remaining 43 by the laboratory of Doha South Sewage Treatment Works. The result of analysis are summarized in Section C of the Supporting Report. Major findings of the survey are summarized below.

3.5.3 General Characteristics of the Groundwater Quality

(1) pH value

The pH values of the groundwater show the normal level of pH 7 in all three study areas.

(2) Electric conductivity (EC)

The results of EC test are divided into the following four classes.

L Class		EC<=3,000 micro mhos/cm
LM Class	3,000 micro mhos/cm	<EC<=6,000 "
M Class	6,000 "	<EC<=9,000 "
H Class	9,000 "	<EC

The distribution of EC values is illustrated in Fig. 3.5.1. From the figure, the following are made clear.

a) EC value of the groundwater in Doha city area

In Doha city area, the EC values increase towards the outer side of the 'B' Ring road. Especially, the EC values are significantly high along the 'C' Ring Road. The distribution of EC values of 33 samples collected from the Doha city area is shown below.

Class			No. of samples
L		EC<=3,000 micro mhos/cm	5(15%)
LM	3,000 micro mhos/cm	<EC<=6,000 "	16(49%)
M	6,000 "	<EC<=9,000 "	7(21%)
H	9,000 "	<EC	5(15%)

It is clear from the above table that EC values of 21 samples or about 65% of the total number of samples show relatively low figures of less than 6,000 micro mhos/cm. It seems that the salinity contained in the groundwater is considerably weakened due to the urban water, in the area inside of the 'B' Ring Road.

High EC values are recorded along the 'C' Ring Road and Wadi Musherib. The EC values in these areas are as high as 10,000 to 20,000 micro mhos/cm.

b) EC value of the groundwater in Rayyan area

The distribution of EC values of 24 samples taken in Rayyan area is shown below. EC values in Rayyan area are evidently higher than those in Doha city area.

Class				No. of samples
		EC<= 3,000 micro mhos/cm		1(4%)
5,000 micro mhos/cm		<EC<= 8,000	"	4(17%)
8,000	"	<EC<=10,000	"	2(6%)
10,000	"	<EC<=20,000	"	13(54%)
20,000	"	<EC		4(17%)

The EC value of 17 samples or about 70% of the total number of samples show high figures of more than 10,000 micro mhos/cm. Such high EC values are mainly measured in the wells for irrigation in the existing farm land located at the northwestern part of the test work sites in the Rayyan area.

c) EC value of the groundwater in New District area

In New District area, where the eastern side is formed by reclamation, only four samples were taken because only few wells where water sampling can be performed could be found. It would be difficult to judge the general characteristics of groundwater quality in the area with such limited number of samples. All the EC values of sampled water record very high figures. Some data show as high as 56,000 micro mhos/cm which is near to the EC values of seawater.

(3) COD values

Sampling of water for the COD test was carried out at 63 sites in total, namely 39 sites in Doha city area and 24 sites in Rayyan area. The COD values obtained are illustrated in Fig. 3.5.2.

Except for certain bad-smelling areas, COD values of the groundwater in the study areas are less than 5 ppm. It is generally judged that the present groundwater is not seriously polluted by the sewage water.

COD values in Rayyan area are a little bit higher than those in Doha city area.

a) COD value of the groundwater in Doha city area

The distribution of COD values of 39 samples collected in Doha city area is shown below.

Class	No. of samples
COD <= 3 ppm	17(44%)
3 ppm < COD <= 5 ppm	10(26%)
5 ppm < COD <=10 ppm	6(15%)
10 ppm < COD	6(15%)

About 70% of the samples show that COD is less than 5 ppm.

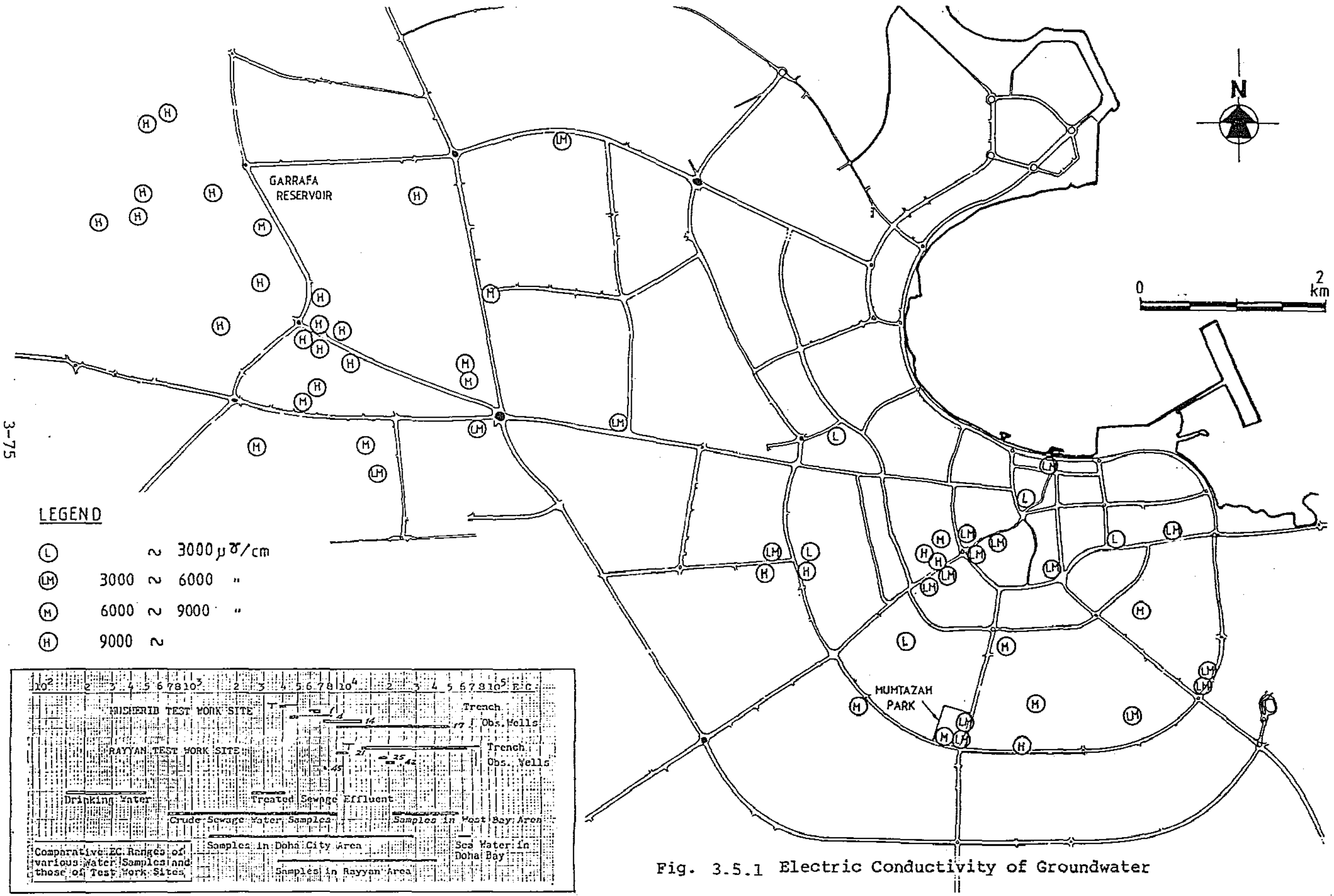


Fig. 3.5.1 Electric Conductivity of Groundwater

The COD values of samples collected from the 16 ASCO project wells are shown as follows.

Class	No. of samples
COD <= 3 ppm	9(56%)
3 ppm < COD <= 5 ppm	2(13%)
5 ppm < COD <=10 ppm	4(25%)
10 ppm < COD	1(6%)

Among the 16 test wells, high values of COD can be seen in the following three wells.

2339-3PD	8.8 ppm
2339-9PR	11.6 "
2239-11TR	7.2 "

b) COD value of the groundwater in Rayyan area

The distribution of COD values of 24 samples collected in Rayyan area is shown below.

Class	No. of samples
COD <= 3 ppm	6(25%)
3 ppm < COD <= 5 ppm	7(29%)
5 ppm < COD <=10 ppm	8(33%)
10 ppm < COD	3(13%)

The samples in Rayyan area were mainly collected from ponds and reservoir in the existing farm lands. There was rubbish and aquatic weeds floating on the water surface. The COD values of these samples would not be the same as those of the natural groundwater. Along with the groundwater sampling, faecal bacteria tests were also made by using field test paper.

Class	Total No. of samples	No. of samples with positive faecal bacteria
COD <= 3 ppm	2	2
3 ppm < COD <= 5 ppm	4	3
5 ppm < COD <= 10 ppm	5	5
10 ppm < COD	3	3

The above samples include 8 samples of pumped water. Out of these 8 samples, five showed positive faecal bacteria.

c) COD value of the seawater

In order to know the polluted conditions of the seawater, COD test was carried out for samples obtained along the coastal line of the Doha Gulf and from within Doha Bay.

The latter samples from Doha Bay have 3.9 to 4.5 mg/l in COD, which are less than those values for samples along the coastal line.

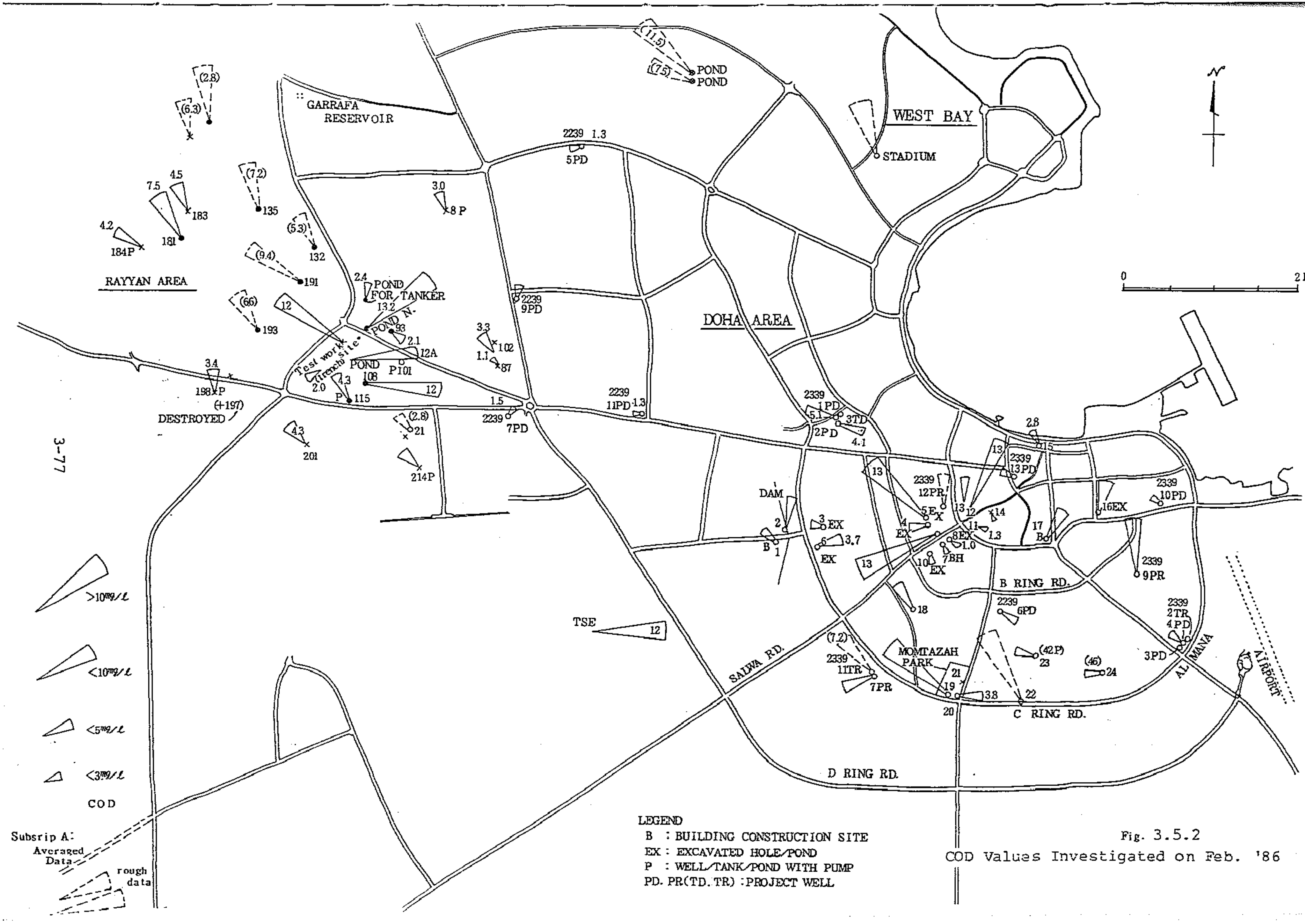


Fig. 3.5.2
COD Values Investigated on Feb. '86

(4) BOD value

By reason of the ease of analysis, the COD values are adopted for judging the degree of pollution of the groundwater. However, in order to know the relationship between the COD value and the BOD value, BOD values were also observed for the samples taken from the test work trenches in Musherib and Rayyan. The BOD values of the samples are shown below together with the COD values.

Item	Musherib	Rayyan
BOD value	0.2 ppm	0.2 ppm
COD value	1.7 ppm	11.9 ppm

(5) Harmful heavy metals

The existence of harmful heavy metals in the groundwater for the proposed area was neglected by the ASCO Study. However, in order to confirm such existence, samples of the groundwater pumped up from the test work trenches in Musherib and Rayyan were analyzed by the Food Pollution Monitor Bureau of District Centre of Ministry of Public Health of Qatar and these analyses results are summarized as Table 3.5.2. It can be judged that the content of harmful metals is less than the figures stipulated in the Japanese standard; therefore, there are no problems for all the samples.

Table 3.5.2 Analysis results for heavy metals of groundwater

Item	Unit	Musherib trench groundwater	Rayyan trench groundwater	Japanese Water Quality Standard for Water Supply
Fe	(mg/l)	<0.0010	<0.0010	<0.30
Mn	"	<0.0010	<0.0010	<0.30
Cu	"	0.0230	0.0450	<1.00
Zn	"	0.0230	0.0450	<1.00
Pb	"	0.0290	0.0166	<0.01
Cd	"	0.0006	0.0033	<0.01
Hexad Cr	"	0.0027	0.0091	<0.05

3.5.4 Change of Groundwater Quality

ASCO project wells were classified by the following two symbols, namely, "PD" (Dammam Formation) and "PR" (Rus Formation).

(1) Change in 5 years

Comparison of EC values measured in ASCO study and JICA study are respectively listed in Table 3.5.3. With the exception of a few figures, there is a general tendency that EC values of stored groundwater are gradually decreasing along the progress of time.

Table 3.5.3 Historical Variation in EC Values at ASCO Project Wells

(1) Lithological Classification

	Project Wells		EC Values (micro mohs/cm)			Ⓑ / Ⓐ
	No.	Locations	Ⓐ ASCO Records		Ⓑ JICA	
			1982 Sep.-Dec.	1983 Jan.-Mar.	1986 Mar.	(%)
DAMMAM	2239- 1PD	Rumaillah Hospital	2,800	-	2,080	74
	- 3TD	"	2,100	1860-1950	1,500	71
	- 5PD	Regional Trading Center	6,100	-	3,140	51
	- 7PD	Nearby the Grand Mosque	12,400	-	6,970	56
	- 9PD	Mamoor corner	3,500	-	6,080	174
	2239- 3PD	Al-Mana Roundabout	8,800	-	5,220	59
	- 5PD	Water Dept. Store	12,500	-		96
	- 6PD	"	-	-	12,000	
	-10PD	Wimpey's corner	-	4,700	3,440	73
	-13PD	Nearby the Court house	-	4,400	2,450	56
RUS	2339- 2TR	Al-Mana Roundabout	14,500	-	4,580	32
	- 7PR	Toy's Club	-	7,100-8,700	6,810	96-78
	- 9PR	Umm Guwalina Clinic	-	11,100	6,630	60
	-11TR	Toy's Club	-	9,500	7,200	76
	-12PR	NEW Head quarter	-	32,000	32,700	102

(2) Regional Classification

Region	Project Well No.	EC Value (micro mohs/cm) Ⓐ ASCO Records 1982 Sep.-Dec.		Ⓑ JICA	Ⓑ / Ⓐ (%)
		1982 Sep.-Dec.	1983 Jan.-Mar.	1986 Mar.	
Inside 'A' Ring Road	2339-13PD	-	4,400	2,450	56
	-10PD	-	4,700	3,440	73
Between 'A' and 'B' Ring Roads	2339-12PR		32,000	32,700	102
Between 'A' and 'B' and 'C' Ring Roads	2239- 1PD	2,800	-	2,080	74
	- 3TD	2,100	1860-1950	1,500	71
	2339- 5PD	12,500	-	-	96
	- 6PD	-	-	12,000	
	- 2TR	14,500	-	4,580	32
	- 3PD	8,800	-	5,220	59
	- 9PR	-	11,100	6,630	60
Outside of 'C' Ring Road	2239- 5PD	6,100	-	3,140	51
	- 9PD	3,500	-	6,080	174
	- 7PD	12,400	-	5,200	42
	2339- 7PR	-	7,100-8,700	6,810	96-78
	11TR	-	9,500	7,200	76

(2) Change in depth

In order to know the vertical change in groundwater quality, water was sampled from various water depths in 7 existing test wells and the EC values were measured. Specifically, 6 wells (2239-7PD, 2239-3PD, 5PD, 10PD and 13PD) were selected from the Dammam Formation and 1 well (2239-12PR) from the Rus Formation.

The water sampled from the deep point of the well 2339-3PD showed very high EC value. It was the same as the EC value of seawater. However, the vertical changes in EC value were negligible in other wells in the Dammam Formation. It is estimated that no significant vertical change takes place in the wells in the Dammam Formation. On the other hand, the EC value on the sample of the Rus Formation was high, although the number of samples is limited to only one from 2339-12PR.

Table 3.5.4 EC values in various depths in wells

Unit: (micro mohs/cm)

Item No.	Dammam Formation										Rus Formation	
	2239-7PD		2339-3PD		2239-6PD		2239-10PD		2239-13PD		2239-12PR	
WD	10 m	17.5 m	7.5 m	20 m	2 m	13 m	6 m	10.5 m	5 m	13 m	6.5 m	40 m
EC	-	-	5,220	49,700	12,200	12,000	-	-	2,400	2,450	28,600	32,700
	(5,470)	(6,970)	(5,220)	(5,400)	-	-	(3,560)	(3,450)	-	-	(29,200)	(35,520)

Note: 1) WD mean depth of water.
2) The value in the parentheses was measured by Laboratory of Doha South Sewage Treatment Plant.

3.5.5 Possibility of Disposal of the Pumped Groundwater into the Sea

Based on the ASCO Report and the results of water quality survey by the JICA Study Team, it is judged that the seawater in the Doha Gulf would not be polluted even though the groundwater is disposed from the project area into the sea. However, certain considerations should be made in order to conserve the environmental conditions of the Gulf at present and in the future. Concerning this, the following are proposed.

As to the discharge into the sea area, both flow rate and water quality have an impact on environment; however, there is no problem for the former in comparison with brine amount drained from the desalination projects being carried out in the Gulf countries.

(1) Water quality standard for disposal of waste water

Standard for disposal of waste water should be established in Qatar.

In Japan, the quality of sewage and waste water of industrial factories are restricted to follow certain standards, when they are disposed into a public water area (river, lake and sea areas). Allowable maximum values of pH, organic and nonorganic materials (BOD, COD), harmful materials (Cd, cyanogen compound and organic phosphate compound) and heavy metals (Pb, Hg. etc.) are stipulated in the standard. As an example, Japanese basic water quality standard for waste water disposal is shown in Table 3.5.5.

Furthermore local governments also have their own standards on the quality of disposed water. The standard are determined by taking the local conditions of industries, environmental conditions, recent water treatment technology and the purpose of water use in the public water area into account. In general, the standards by the local governments are more strictly determined than the National Standard. Some local standards stipulate that the values of COD and BOD should be less than 10 ppm.

Table 3.5.5 National Drainage Standards from Japanese Prime Minister's Office Law

Items	Unit	Allowable Limit
Ph		5.8~8.6 for fresh water area 5.0~9.0 for sea area
BOD	mg/l	160 (Average 120 daily)
COD	"	160 (Average 120 daily)
SS	"	200 (Average 150 daily)
Normal hexane (mineral oils)	"	5
Normal hexane (oils and fats from animals or plants)	"	30
Phenol	"	5
Copper (Cu)	"	3
Zinc (Zn)	"	5
Dissolved iron (Fe)	"	10
Dissolved manganese (Mn)	"	10
Chromium (Cr)	"	2
Fluorine (F or Fl)	"	15
Colon Bacilli	No./cm^3	Average 3,000 daily
Cadmium & other compounds	mg/l	0.1
Cyanogen compound	"	1
Organic phosphorus (Parathion, methyl parathion, methyl and EPN)	"	1
Plumbum & its compound	"	1
Hexad chromium	"	0.5
Arsenic & its compound	"	0.5
Mercury, alkyl mercury & other mercury compound	"	0.005
Alkyl mercury compound	"	None
PCB	"	0.003

(2) Environmental standard of water quality

In Japan, the environmental standard for water quality, which is suitable for the utilization objectives of public water area (sources of water supply, industrial use, marine industrial use, and environmental conservation etc.), is regulated so as to preserve the living environment.

As an example, the environmental standard of water quality in Japan is shown in Table 3.5.6.

Table 3.5.6 Environmental Standard of Water Quality in Japan

1) Seawater body

	pH	COD	DO	Faecal Bacteria	n-Hexane Ext.
Class A	7.8-8.6	<2 ppm	>7.5 ppm	<10 MPN/ml	No detection
Class B	7.8-8.3	<3 "	>5 "	-	No detection
Class C	7.8-8.3	<8 "	>2 "	-	-

Note: MPN means Most Probable Number
Class A to be applied to the 1st grade sea areas for marine products and bathing.
Class B to be applied to the 2nd grade sea areas for marine products and industrial use.
Class C to be applied to the conservation of natural environment.

2) River water body

	pH	BOD	SS	DO	Faecal Bacteria
Class AA	6.8-8.5	<1 ppm	<25 ppm	>7.5 ppm	0.5 MPN/ml
Class A	6.8-8.5	<2 "	<25 "	>7.5 "	10 "
Class B	6.8-8.5	<3 "	<25 "	>5 "	50 "

Note: Class AA to be applied to the source for 1st grade drinking water supply and cases for the conservation of natural environment.
Class A to be applied to the 2nd grade drinking water supply and sources for 1st grade sea areas marine products and bathing.
Class B to be applied to the 3rd grade drinking water supply and sources for the 2nd grade fresh water marine products.
Lower classes than the above are omitted from this table.

3) Lake water body

	pH	BOD	SS	DO	Faecal Bacteria
Class AA	6.5-8.5	<1 ppm	< 1 ppm	>7.5 ppm	0.5 MPN/ml
Class A	6.5-8.5	<3 "	< 5 "	>7.5 "	10 "
Class B	6.5-8.5	<5 "	<15 "	>5 "	50 -

Note: MPN means Most Probable Number.
Class A to be applied to the sources for 1st grade drinking water supply and 1st grade marine products, and conservation of natural environment.
Class A to be applied to the source for 2nd and/or 3rd grade drinking water supply, 2nd grade marine products, industrial and irrigation uses.
Class B to be applied to the sources for 3rd grade drinking water supply and 2nd grade marine products.
Lower classes than the above are omitted from this table.

3) Consideration of groundwater discharge into the sea

In the case of groundwater discharge into the sea, its COD value should be regarded as the most important factor among various factors having an impact on the environmental conditions of the sea.

The results of continuous pumping tests carried out at the test work sites show that the COD values fluctuated in a range of 0.7 to 3.6 ppm and 3.9 to 14.6 ppm at Musherib and Rayyan area respectively. The COD values in Rayyan area are relatively higher those of Musherib area and they scatter widely.

As far as the COD value of the seawater seems to be slightly less than that of the Rayyan groundwater, it seems permissible that the groundwater be discharged into the sea (Doha Bay).

However, it should be necessary to continue long-term monitoring of the quality of discharged groundwater to ensure that no negative environmental consequences occur. Furthermore, it is desirable that the groundwater abstracted from Rayyan area should be indirectly discharged by passing through the simple organism treatment basin such as plantation of mangrove.

4. BASIC APPRAISAL OF MEASURES TO PREVENT RISING GROUNDWATER LEVEL

4. BASIC APPRAISAL OF MEASURES TO PREVENT RISING GROUNDWATER LEVEL

4.1 Overall Review of Measures to Prevent Rising Groundwater Level

4.1.1 Background of Problem

(1) Origin of Problem

The problem of the rising groundwater level occurred in the agricultural sector in the 1970's and was described as "water logging". When agriculture is carried out in a semi-dry area, a large volume of water is used to wash out the accumulated salt in the soil in order to control the salt density in the soil moisture. The permeation of a large volume of water from the ground surface underground eventually destroys the water balance of the groundwater, causing a rise in the groundwater level. As the original groundwater tends to be brackish with a high salt density, the rise of this type of groundwater near to the surface inevitably affects both the growth and yield of the crops in the area. When the groundwater level further rises to the surface, the result is standing water which provides a breeding ground for mosquitoes and makes the area barren due to the solidification of the salt by the sun.

Special emphasis was always placed on irrigation in early agricultural development projects. In addition to irrigation facilities, however, the importance of drainage facilities to control the groundwater level in agricultural fields and the salt density in the soil moisture has now been duly recognised.

Doha is very poorly endowed with natural water resources, having annual rainfall of a mere 75 mm. In the past, wells were dug in lowland areas and, up until recently, agricultural activities were carried out. The most common wells were those which were dug without timbering and which were some 10 m wide, 10 m long and 5 m deep. In recent years, however, the practice has been to dig deeper using a drilling machine and for a pump to be installed, suggesting that the groundwater level dropped in the 1970's when agriculture was still active due to the excessive pumping up of the water.

Following the increase of the oil revenue in 1974, drastic changes occurred in Doha's water supply and agricultural activities. With the completion of the distillation plant, a large volume of good quality water could be obtained from distilled seawater and this was distributed to the residents free of charge, thus reducing the value of the groundwater with a high salt density. The construction of the urban infrastructure became extremely active, attracting the farmers to construction development projects and agriculture, which offered a low income vis-a-vis such development projects, was subsequently abandoned. A number of former agricultural fields can be seen today outside the 'D' Ring Road.

The above-described changes in the water utilization had the following effects on the water balance of the groundwater.

1 The volume of water recharged underground increased due to the leakage of distilled water from potable water pipes, cesspits and septic tanks.

2 The volume of pumped up groundwater decreased with the decline in agricultural activities.

The rise of the groundwater level was, therefore, caused by the increased volume of water recharged underground and the decreased pumped up volume. The recharge of distilled water was mainly responsible for the increase of the groundwater volume.

As described above, the problem of the rising groundwater level in Doha originates from the production of a large volume of distilled water and its permeation underground. This problem can be described as an urban version of water logging. In addition, the problem is common to all cities of gulf countries where distilled water is currently in use.

(2) Mechanism of Rising Groundwater Level

Any change in the groundwater level is caused by an unbalanced water inflow and outflow. The rise of the groundwater level may be caused by either an increased inflow or a decreased outflow. According to the study results obtained upto the present, the rise of the groundwater level in Doha is caused by an increased inflow. To be more precise, the study results conclude that the leakage of water from potable water pipes, the permeation of sewage underground and over-irrigation following the rapidly increasing water demand in recent years have increased the volume of water recharged underground.

In general, rainwater also affects the groundwater recharge. The following observations are made based on the analysis of the secular changes in the groundwater level at observation wells.

1 Only a limited number of wells show a rise in the groundwater level due to rainwater and the increased level returns to normal after a few months.

2 A large majority of those wells where a rise in the groundwater level has been observed in the last 3 years show a gentle rising curve with little seasonal fluctuation.

Therefore, from this analysis of the secular changes in the groundwater level, it is obvious that the rise of the groundwater level is not caused by rainwater but by a rapidly expanding water demand.

With this rapidly expanding water demand, the recharge of the water underground can take 3 different forms, as illustrated in Fig. 4.1.1.

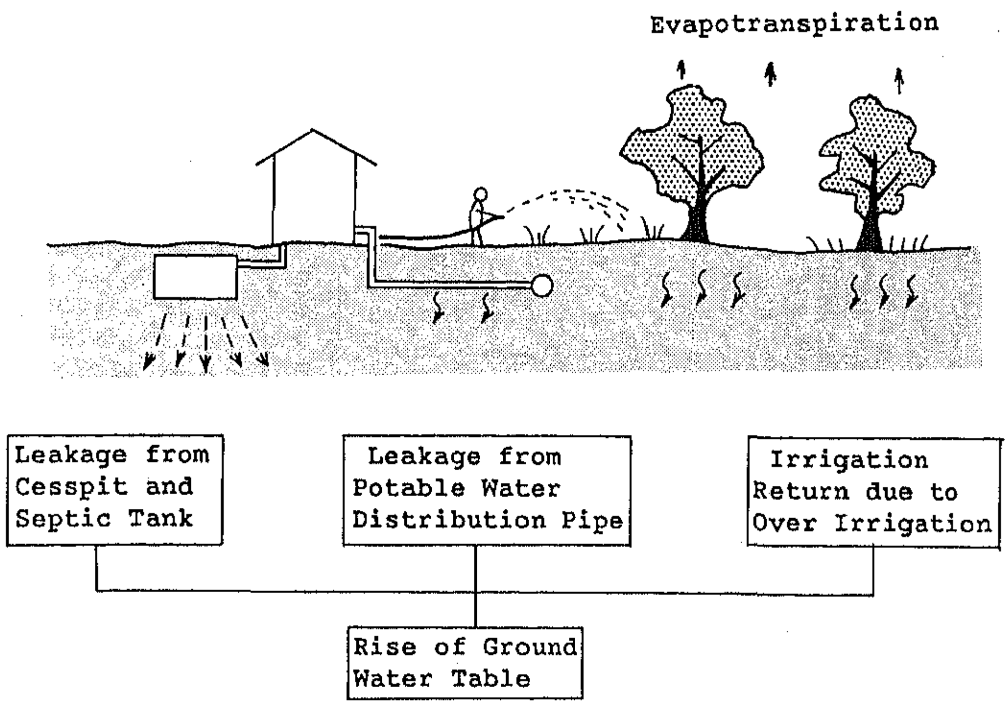


Fig. 4.1.1 Schematic Representation of Groundwater Recharge Increase Patterns

(3) Water Demand Trend

The production of the distillation plant has been steadily increasing in the last decade, as shown in Fig. 4.1.2, and Doha is supplied with 95% of the water produced.

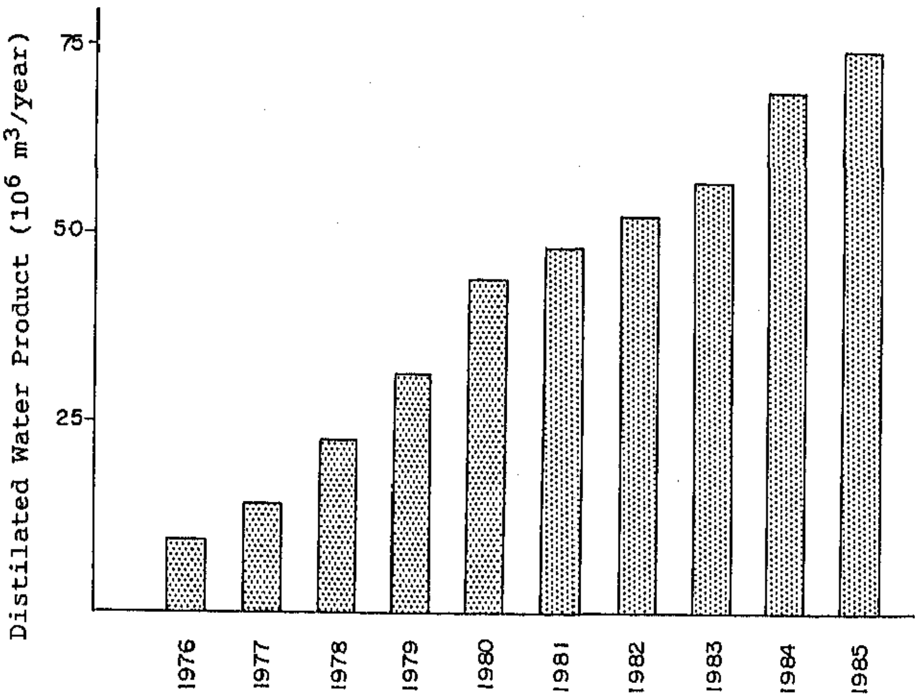


Fig. 4.1.2 Distilled Water Production in the Last Decade

The water demand is classified into municipal water, industrial water and irrigation water and the future prospects of the economy and industry provide important indices for an estimate of the future water demand.

Here, it is assumed that municipal water accounts for most of the water consumption in Doha. In this case, the water consumption has close relationship with the size of the population and this relationship can be shown by the following equation.

$$W_C = C_P \times A_P \times P_P + C_T \times A_T \times P_T$$

Here,

W_C : Water Consumption in Doha
C_P : Per Capita Water Consumption in Potable Water Service Area
A_P : Total Area where Potable Water Service is Provided
P_P : Population per Unit Area of Potable Water Service Area
C_T : Per Capita Water Consumption in Tanker Water Supply Area
A_T : Total Area where Tanker Water Supply is Provided
P_T : Population per Unit Area of Tanker Water Supply Area

The following reasons can be given for the increased water demand.

1. Population growth
2. Increased per capita water consumption
3. Expansion of the service area due to the inclusion of new areas

The results of the population survey conducted in March, 1986 should provide important information for predicting the future trend of the water demand. The final results, however, have not yet been released. Based on the interviews, those opinions suggesting that the population is unchanged are the mainstay.

Assuming that the population is unchanged, the increased water demand in recent years can be partly explained by the increased per capita water consumption in high-class residential areas, reflecting the construction boom of superior detached houses in the suburbs, and partly by the additional new potable water service areas.

With the decline in the oil price, the rapid urbanisation that has been seen upto the present may slow down in the future. However, with the continued construction of superior detached houses in the suburbs and the development of the New District, the water demand will continue to rise unless the current mode of water consumption is altered.

(4) Types of Groundwater Recharge

(i) Groundwater Recharge from Potable Water

Fig. 4.1.3 shows the process of groundwater recharge from potable water. The main types of recharge are as follows.

1. Leakage from potable water pipes
2. Leakage from water storage tanks
3. Leakage from tanker filling stations

As any large leakage from potable water trunk lines and water storage tanks can be quickly detected by the Telemetric Centre which is responsible for controlling the daily flow of these facilities, appropriate measures can be immediately taken. In comparison, house connections, which can be damaged by heavy vehicles running over them, corroded by groundwater with a high salt density or the surface coating cracked by underground temperature fluctuations, have an enormous total length and, therefore, small holes or cracks may not be detected for a long time. In those cases where the leaked water surfaces to form standing water, however, repairs are swiftly carried out.

While leaked water may easily find its way to the surface in those places where the groundwater level is shallow, it is difficult to detect leakage in those places where the groundwater level is deep.

The Ministry of Electricity and Water sometimes receives complaints of the ground surface being flooded by potable water leakage. Upon examination, however, these are often found to be sewage leaks.

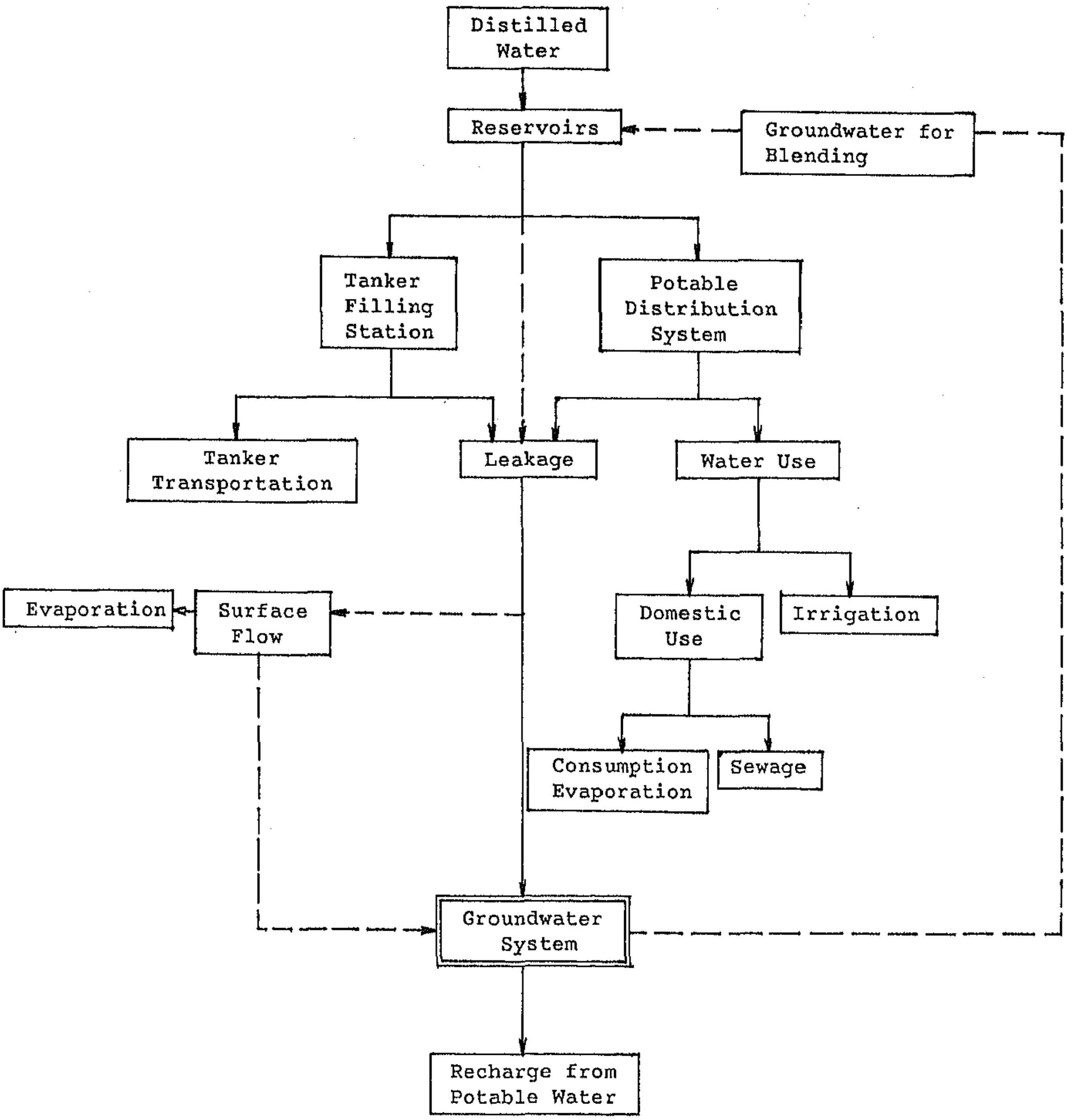


Fig. 4.1.3 Groundwater Recharge from Potable Water

(ii) Groundwater Recharge from Sewage

Fig. 4.1.4 shows the process of groundwater recharge from sewage. The main types of recharge are as follows.

1 Leakage from cesspits and septic tanks
2 Leakage or overflow from sewage pipes
3 Leakage from TSE distribution pipes

Leakage from cesspits and septic tanks will gradually decline as the number of these facilities will be reduced in accordance with the progress of the Sewerage Improvement Project. Leakage of sewage tends to occur at joints and overflows occur when the sewage pipes are partially blocked by dirt, etc. The total volume of leakage from the TSE distribution pipes is rather low in comparison with the other two types of leakage.

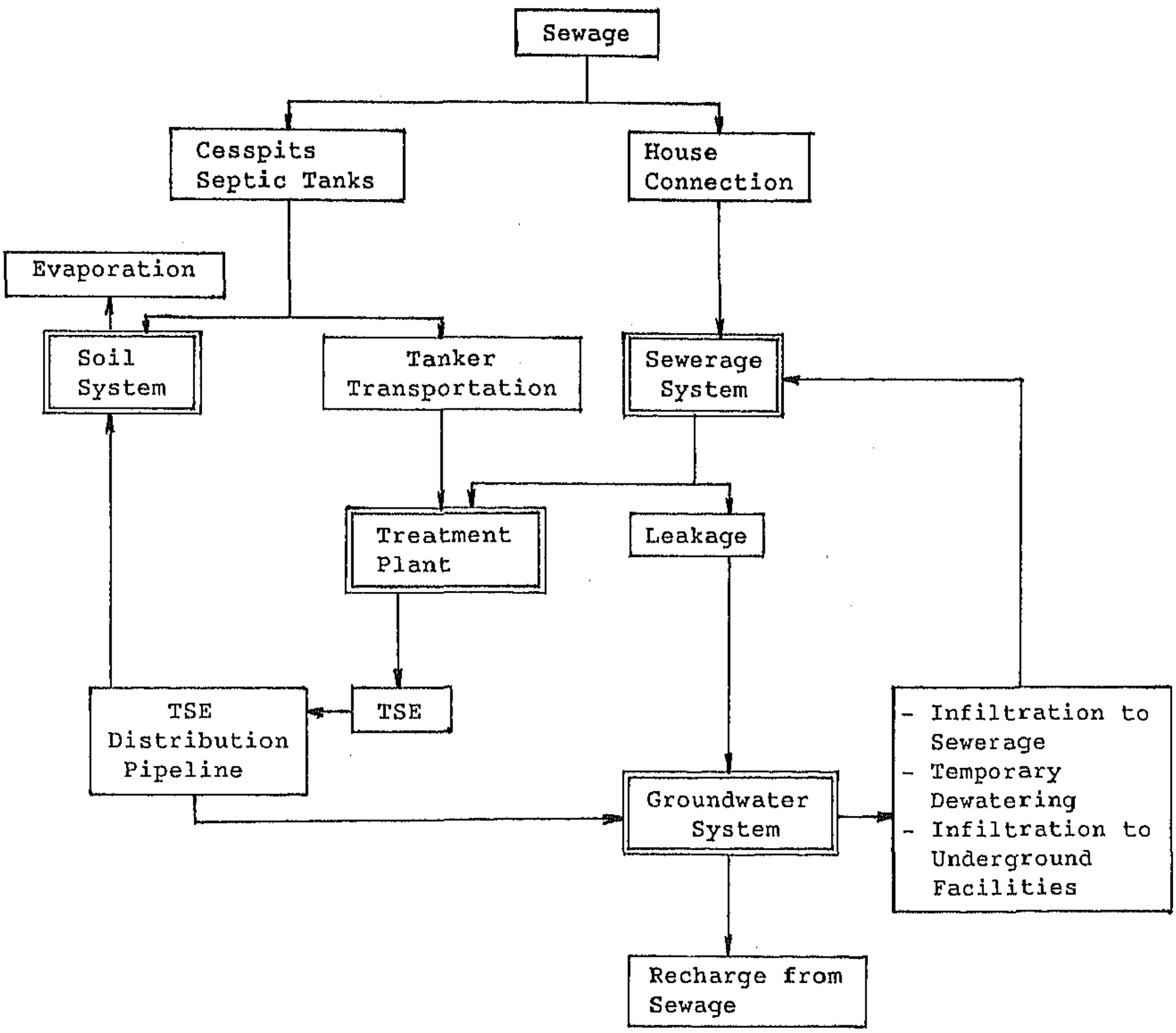


Fig. 4.1.4 Groundwater Recharge from Sewage

Those sewage pipes which are laid beneath the ground surface are infiltrated by groundwater. In fact, some 30% of treated sewage is said to consist of groundwater. The infiltration of seawater is particularly noticeable in sewage from the New District.

(iii) Groundwater Recharge from Irrigation Water

Fig. 4.1.5 shows the process of groundwater recharge from irrigation water. Some of the irrigation water enters the soil system where it is held in the pores of the soil grains. It is absorbed by the roots of plants and is eventually lost to the atmosphere through evaporation. The remaining water freely descends through the soil to merge with the groundwater.

As the volume of water held by the soil grains is limited, the volume of water infiltrating through the soil system to directly merge with the groundwater increases on over-irrigation. As some of the irrigation water was originally groundwater, however, some of the recharged groundwater is reused as irrigation water and, therefore, calculation of the groundwater recharge volume from irrigation water should take this fact into account.

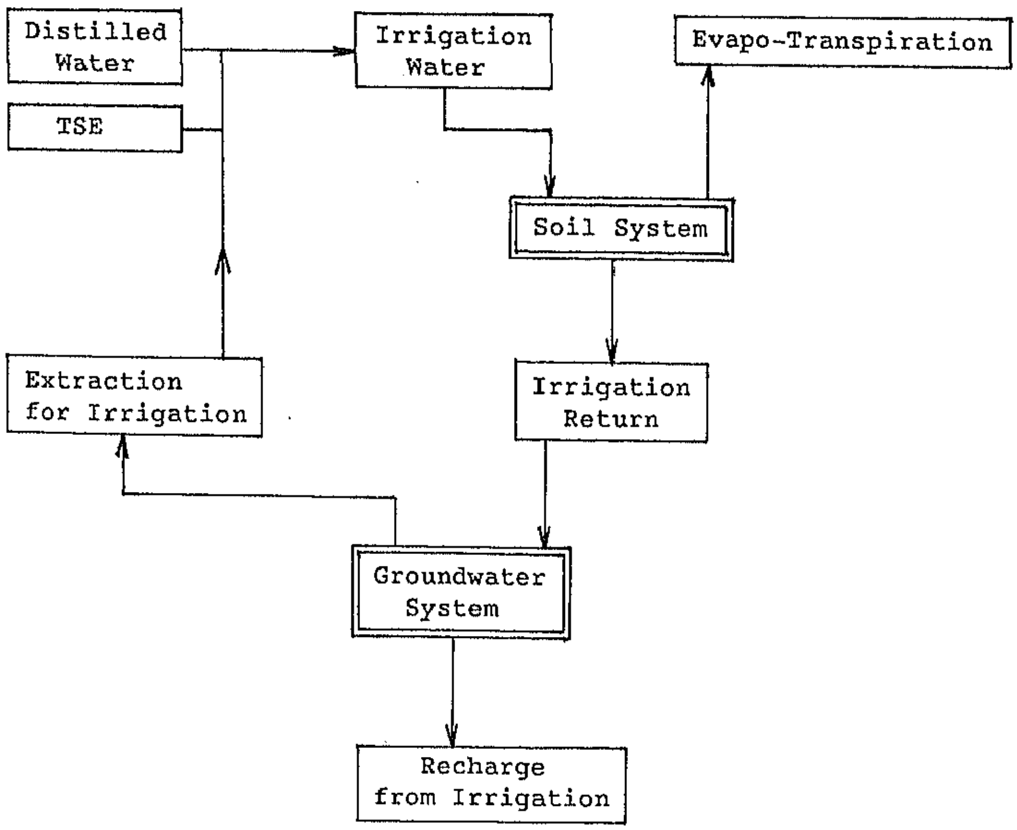


Fig. 4.1.5 Groundwater Recharge from Irrigation Water

4.1.2 Basic Strategy

(1) Measures to Deal with Damage Caused by Rising Groundwater Level

Since there are various type of standing water, ranging from the local standing water caused by leakage from potable water or sewage pipes to the extensive standing water in Rayyan, appropriate measures should be introduced depending on the types of causes.

The following objectives for each stage of the study have been set in view of introducing appropriate measures to deal with the damage caused by the rising groundwater level.

1. Understanding of the current situation of damage
2. Examination of the causes of damage
3. Judgement of possible extension of damage
4. Planning of measures
5. Verification of the feasibility of the planned measures and judgement of their effects
6. Project implementation

Fig. 4.1.6 shows the stages of the study and the corresponding measures to be implemented. Previous studies achieved various results upto stage 4 and the Sewerage Improvement Project and the Stormwater Drainage Project, etc. have been accordingly implemented.

The policy decision to introduce a project consisting of comprehensive and systematic measures for the prevention of the rising groundwater level should be generally appreciated. However, as an underground phenomenon cannot be directly observed by eye, complete understanding can hardly be obtained through a number of borings or lateral excavations. Therefore, there is the dilemma that if the measures are planned to be implemented after comprehensive understanding of the problem has been achieved, the implementation of urgent measures will be delayed.

In view of this, it is important for priority to be given to the early commencement of the Groundwater Drainage Project with stress on the necessity and urgency of introducing urgent measures and for the results obtained through the implementation of these urgent measures to be applied to those areas where the damage is expected to become extensive in the future.

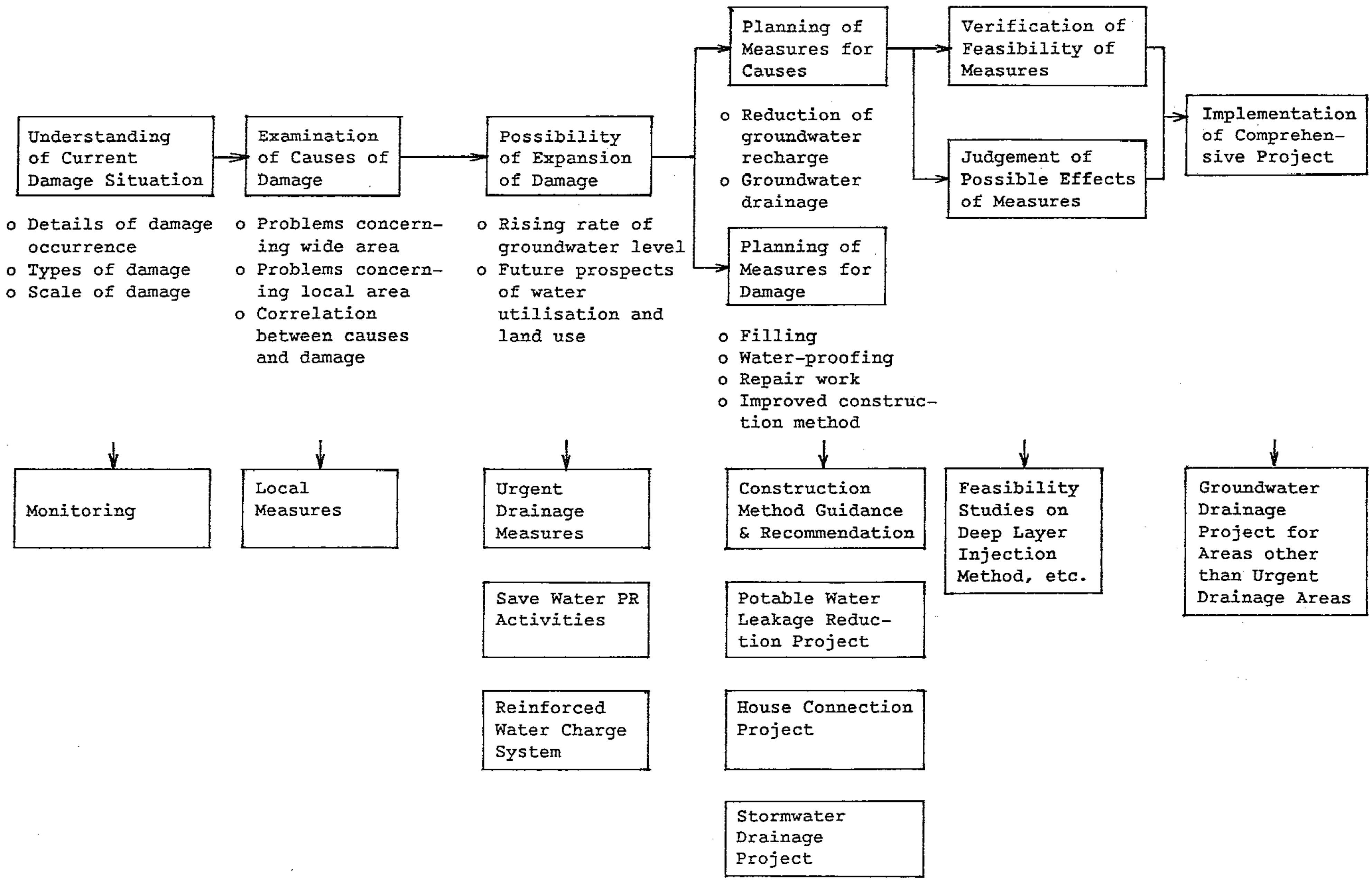


Fig. 4.1.6 Development of Measures to Deal with Damage Caused by Rising Groundwater Level

(2) Classification of Measures in Terms of Groundwater Balance

As the measures to prevent the rise of the groundwater level can be classified into two categories, i.e. measures to reduce the inflow volume and measures to increase the outflow volume, the currently proposed measures are systematised as shown in Fig. 4.1.7.

The inflow to the groundwater can be reduced by the following measures.

- o Reduction of the infiltration of rainwater into the ground by draining the standing water caused by rain.
- o Reduction of the infiltration of potable water into the ground by the proper repair and maintenance of supply pipes.
- o Reduction of the infiltration of sewage into the ground by connecting cesspits and septic tanks to the sewerage system.
- o Reduction of the sewage generation by reducing the potable water consumption.
- o Prevention of over-irrigation.

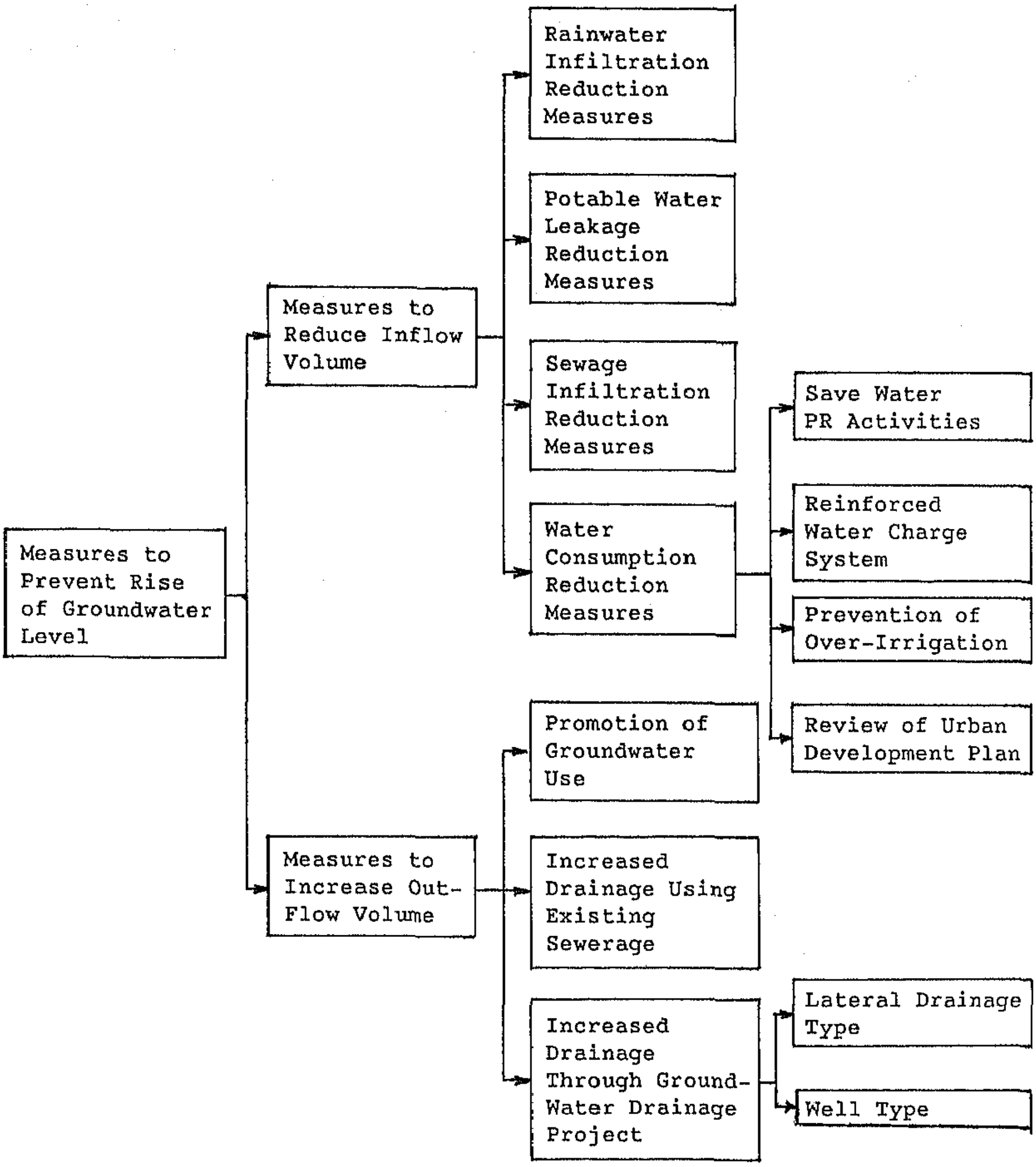


Fig. 4.1.7 Classification of Measures in Terms of Groundwater Balance

In comparison, the measure to increase the outflow volume intends the control of the groundwater level by means of pumping up or draining the groundwater.

The measures to reduce the inflow can be described as indirect measure to control the groundwater level while those to increase the outflow are more direct measures. In particular, when the groundwater level conspicuously rises, resulting in a rapid expansion of the damage, the implementation of these direct measures should be given priority to achieve immediate results. In this context, therefore, the forced drainage of the groundwater based on groundwater drainage is considered to be the best measure. Although the other measures are also important in view of their own advantages, some time is required for their intended results to be achieved. In addition, their maximum effects can only be felt when a comprehensive linkage between them is established.

(3) Development of Comprehensive Measures

It is important that comprehensive plan be established, the future trends of the rising groundwater problem in Doha studied and possible prevention measures implemented.

The prediction of the rising groundwater problem in the future is a necessary precondition for the Basic Plan and can be clarified by studying the following;

1 The industrial and urban development plans in Doha based on the national development plans

2 The estimated water demand in Doha

3 The estimated groundwater recharge

4 The estimated rise of the groundwater level

5 The forecast of future damage

As the stabilisation of the oil price is closely related to the industrial and urban development plans in Doha, the current general opinion is that the economic growth will slow down and rapid urbanisation, as well as industrialisation, will not occur.

Assuming that a rapid population growth will not occur, the water demand will still continue to steadily increase for some years until the migration of the population within the city, i.e. families leaving the central area for high-class suburban residential area, is arrested.

In terms of the groundwater recharge, the appearance of new residential areas will result in new sources for recharging the groundwater. The recharge volume in the existing areas, however, will not drastically change as few changes in the form of the water utilisation are expected.

The groundwater level can be estimated using a simulation model based on the groundwater level observation results and the geological survey results. An actual estimate can be obtained by computing the groundwater recharge volume, which depends on the surface conditions, the initial conditions of the groundwater level, the hydraulic constant of each layer and the layer thicknesses, etc.

Those areas where damage is likely to occur due to the rising groundwater level can be predicted by correlating the estimated groundwater level and the surface conditions. In addition, the possible types of damage in these areas can also be predicted by taking the land use situation into account.

One important judgement criterion for the implementation of the Project is the consideration of whether or not any measures should be implemented in view of predicted damage. The economic effects of a non-productive project, such as a project concerning the problem of the rising groundwater level, can be calculated in the following manner.

a Reduction in the cost of damage due to the implementation of the project.

b Construction cost required to return the groundwater to its original level (substitutional construction cost).

In general, the damage includes both the indirect damage and the potential damage which in many cases cannot be accurately calculated . In addition, bold assumptions may have to be made in regard to those items which appear calculable. In the case of underground facilities for example, telephone and power cables tend to suffer more damage in accordance with the rise of the groundwater level as they are inherently weak against water.

The substitutional construction cost is the cost required for a groundwater drainage project to restore the original groundwater level at which no damage is caused. This can be more easily calculated than the reduced damage amount. Therefore, the establishment of an appropriate groundwater drainage project size, taking the future prospects of the rising groundwater level problem into account, is required in order to avoid an excessive financial burden in the future.

As so far described, 2 measures for lowering the groundwater level have been proposed, i.e. by reducing the inflow and by increasing the outflow. In reality, it important that these measures be employed in a comprehensive manner to promote an economical, as well as effective project which meets the social requirements.

There is an increasing awareness of the rapid expansion of the damage caused by the rising groundwater level and the urgent establishment of measures for specific areas affected by this damage is socially required.

From an economic point of view, a groundwater drainage project would reduce the economic burden posed by the rising groundwater level. The cost of a groundwater drainage project to prevent damage by the rising groundwater level can be determined by the design groundwater level and the required drainage volume. As described in Section 3.2 of the Damage due to Groundwater, the desirable groundwater level is set at approximately 1.0 to 1.5 m below the ground surface. If the intention is to lower the groundwater level to 1.5 m below the surface in the entire subject area, however, a number of branch channels will be required, in addition to trunk channels, due to the fact that the permeability changes from place to place, necessitating the introduction of a huge network. In comparison, if those facilities which satisfactorily prevent the accumulation of salt are deemed to be adequate for the present purpose, even though the groundwater level may not lowered to the design level due to the different permeabilities, the project's objective will be development of trunk channels. The present study considers the latter to be both more economical and rational than the implementation of the project based on a complete study of the different permeabilities throughout the entire subject area. The following points can be made in regard to the effects of the project.

1 The groundwater drainage measures would be effective in directly controlling groundwater level and, therefore, should be considered as the main focus of a project dealing with rising groundwater level. Details of the drainage measures are given in 4.2.

2 Although the measures to reduce the groundwater recharge volume would not directly control the groundwater level, they would help in reducing the required drainage volume. Details of these measures are given in 4.3.

(4) Application of Concrete Measures

Since studies on the damage caused by groundwater usually commence after the actual damage has been recognised due to the simple fact that the underground situation cannot be seen in advance, the basic information required for planning measures is often insufficient. Another problem associated with underground surveys is that however thorough the original survey, there are always some discrepancies between the original survey results and the results observed at the time of project implementation. Particularly in the case of limestone, it is well-known that the permeability of a limestone layer shows heterogeneity due to the karst phenomenon on which the general groundwater flow theory will be sometimes difficult to apply.

There are two (2) methods for applying concrete measures, i.e. the implementation of urgent measures selected from established measures over a whole area and the implementation of urgent measures in a particular area while, at the same time, the monitored results of these measures are applied to other areas for environmental conservation. The latter method is applied in the present study because of the following reasons.

1 Since the urgent implementation of measures is required, it would be inappropriate at the present time to try to make a comprehensive appraisal which includes those measures whose feasibilities have not yet been confirmed.

2 The data obtained from the implementation of the urgent measures will more accurately show the state of the actual conditions than the present study data.

3 Despite the fairly large cost involved, groundwater surveys, including boring surveys, only offer a partial understanding of the actual groundwater conditions and leave a great deal to guesswork.

Of these areas which have a noticeable rising groundwater level problem, the following (3) three areas are examined in the present study as concrete examples for an urgent groundwater drainage project.

- Wadi Musherib .... Wadi Type
- Rayyan .... Inland Depression Type
- New District .... Coastal Area Type

As these 3 areas have different topographical characteristics, i.e. wadi, inland depression and coastal area, it is believed that the data obtained from these areas can be effectively used for future groundwater drainage projects in other areas.

4.1.3 Examination of ASCO's Recommended Long Term Measures

The opinions of the JICA Study Team, which largely differ from those expressed by ASCO's long term measures, are given here.

(1) Comprehensive Drainage Plan

a) Summary

ASCO suggests that the overall lowering of the groundwater level could be achieved by direct pumping from the two (2) groundwater mounds in Doha.

b) Comments

The distribution of the existing wells shows a high concentration of wells in the inland depression area due to the fact that groundwater tends to gather there and that the necessary excavation depth is shallow. The groundwater mounds, however, are located on high ground where weathering of the layers by rainwater is unlikey in view of the environment. From the hydrogeological points of view, therefore, there may be varying permeabilities and a diffusive structure where it is difficult for groundwater to collect. For high ground drainage methods, refer to Section 4.2 which examines the use of either tunnels or shafts.

(2) Reutilisation of Extracted Groundwater

a) Summary

It is suggested that the project to lower the groundwater level be accompanied by an auxiliary project whereby the extracted groundwater could be used for irrigation.

b) As a long time is required for plants, particularly trees, to grow, the restoration of the former farming fields outside the 'D' Ring Road appears difficult without a large subsidy. One reason for the original abandonment of these fields was the fact that the groundwater tends to show a higher salinity in accordance with its continued use as irrigation water, depending on the original salinity of the groundwater.

4.1.4 Vertical Injection

(1) Vertical Injection Application Problems

Water injection to the Umm Er Radumah (UER) layer could result in the pollution of the deep groundwater. It could also cause the level of the groundwater in the Upper Dammam layer to rise again as the cap rock could be destroyed by the injected water and water could infiltrate into the Upper Dammam layer, thus becoming a recharge source.

Before the implementation of this method, therefore, it is necessary that the following items, relating to the abovementioned possible results of this method, be studied in order to pinpoint possible problems and to clarify the measures required to prevent them.

1) Water Quality

Although a problem of pollution is involved, it is essentially a question of the inherent quality of the water concerned.

Therefore, a thorough water quality analysis should be conducted in regard to both the groundwater extracted from the mound for injection and the water from the aquifers of the UER layer where the mound groundwater is injected.

2) Hydrogeology

1 Groundwater Level

Although the groundwater level in the Upper Dammam layer is given in the groundwater level contour map, the actual level of the groundwater may vary within the same aquifer. The aquifers in the RUS and UER layers should, therefore, be clearly distinguished and the piezometric pressure for each aquifer should be measured over a wide area.

2 Permissible Injection Amount to UER Layer

This can only be calculated by obtaining the injectivity index (II) through injection test. In theory, the injection volume does not correlate with an effective pore ratio or permeability coefficient. In practice, however, it is believed that the II should be obtained through actual testing. There are also questions of to what extent the injected water affects an aquifer and of how the high pressure zone is formed and disappears in a limited area inside an aquifer under continuous injection.

The productivity index (PI) of the extraction layer should be determined in advance in view of determining the rate at which the water can be extracted from the Upper Dammam Layer.

3 Cap Rock

The Lower Dammam layer (Midra Shale) is believed to block the vertical infiltration of the groundwater stored in the aquifers of the Upper Dammam layer and, therefore, it should act as cap rock. With regard to cap rock in general, there is the safety problem of its reliability in terms of collapse or leakage at the time of injection. As areas of depression are particularly susceptible to this problem, the vertical distribution of the Lower Dammam layer, its thickness, permeability and characteristics of collapse should be studied.

3) Injection Wells

1 Function of Injection Wells

An injection method which can be employed without danger of collapsing a layer and its scope of application (in terms of the injection pressure and injection volume) should be determined based on the results of the study on the layer strength.

2 Well distribution

The number and distribution of the injection wells should be carefully decided in view of the possible interference between wells.

Test Wells
Injection System
→ Establishment of Injection Fields and Yards

4) Maintenance

The main concern in this regard is a reduction in the injection efficiency due to clogging caused by bacteria activities. The preliminary treatment of the injection water will be necessary to avoid this. As there are two types of bacteria, i.e. aerobic and anerobic, both types should be studied and analysed.

5) Vertical Drainage System

A total system should be developed taking into consideration the above-mentioned aspects from 1) to 4).

6) Expected Injection Rate

The results of the feasibility tests on water-soluble natural gas in Niigata, Japan suggest that the expected injection rate expressed by the relation between the productivity index (PI) and the injectivity index (II) should be given careful consideration in view of a discrepancy between the originally expected injection volume and the actual volume. It is natural to suppose that the injection plan be carried out with the belief that the injection volume is equal to the production volume in so far as the same aquifer is concerned. It is suggested, however, that the actual relation between the II and PI is unequal and that the II is approximately one-fifth of the PI.

If the injection volume is increased without considering this fact, the well bottom will be destroyed and a lot of time and expense will be required to restore it.

The largest injection volume recorded in the Niigata gas field is 4,000 kl/day/well.

(2) Vertical Injection Methods

1) Types of Vertical Injection

There are 3 vertical injection methods which can be considered (see Fig. 4.1.8) and the injection is carried out under a higher pressure than the original pressure at the head of aquifer concerned in all 3 cases. Therefore, the expression "pressurised injection" is used to describe them.

a) Direct Gravity Injection Method
b) Gravity Injection with Pretreatment
c) Pressurised Injection Method

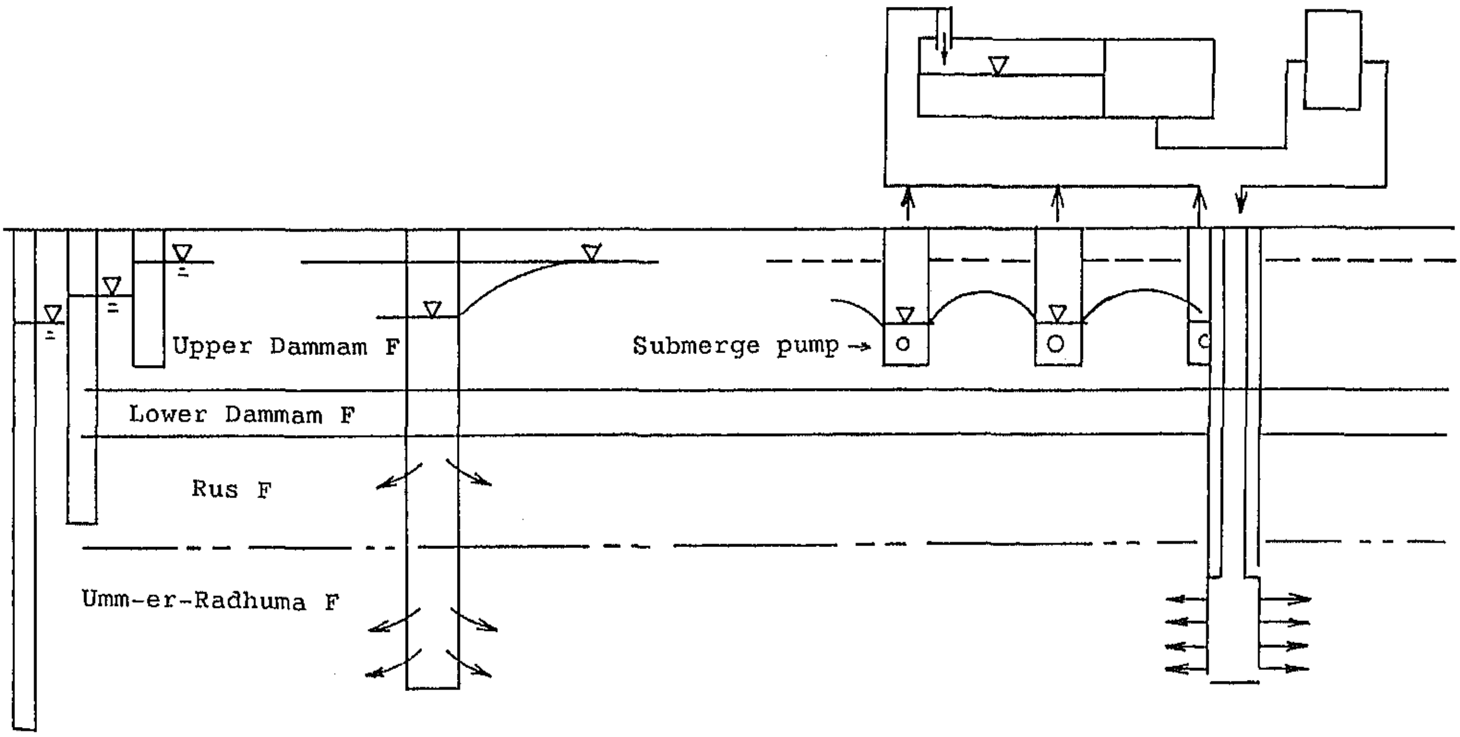


Fig. 4.1.8 Conceptual Diagram of Vertical Injection Methods

1 Direct Injection Method

While this is the most simple method, it involves the problems of unwanted sand grains being carried from the extraction layer and deterioration in the permeability of the injection layer due to foreign matter in the water or the propagation of bacteria (either aerobic or anerobic), as well as the problem of water pollution. Even though it is recognised at the beginning that there is a certain difference in the head, this method is likely to lose its efficiency in a short time and unless the conditions of the injection layer are good, clogging rapidly occurs even when a strainer is provided, preventing the original function from being performed at an early stage.

2 Injection Methods with Pretreatment

Pretreatment is, therefore, required to prevent the clogging of the injection layer. However, as pretreatment necessitates a large facility size and a high cost, a thorough preliminary study should first be conducted to avoid this method being wrongly introduced. In addition, the utmost care should be taken in the operation and maintenance of pressurised injection equipment.

The water pumped up from the extraction layer is stored and the pretreatment is conducted at a certain size pool on the ground. The main treatment is the sedimentation of the sand grains, sterilisation against bacteria and deoxidation.

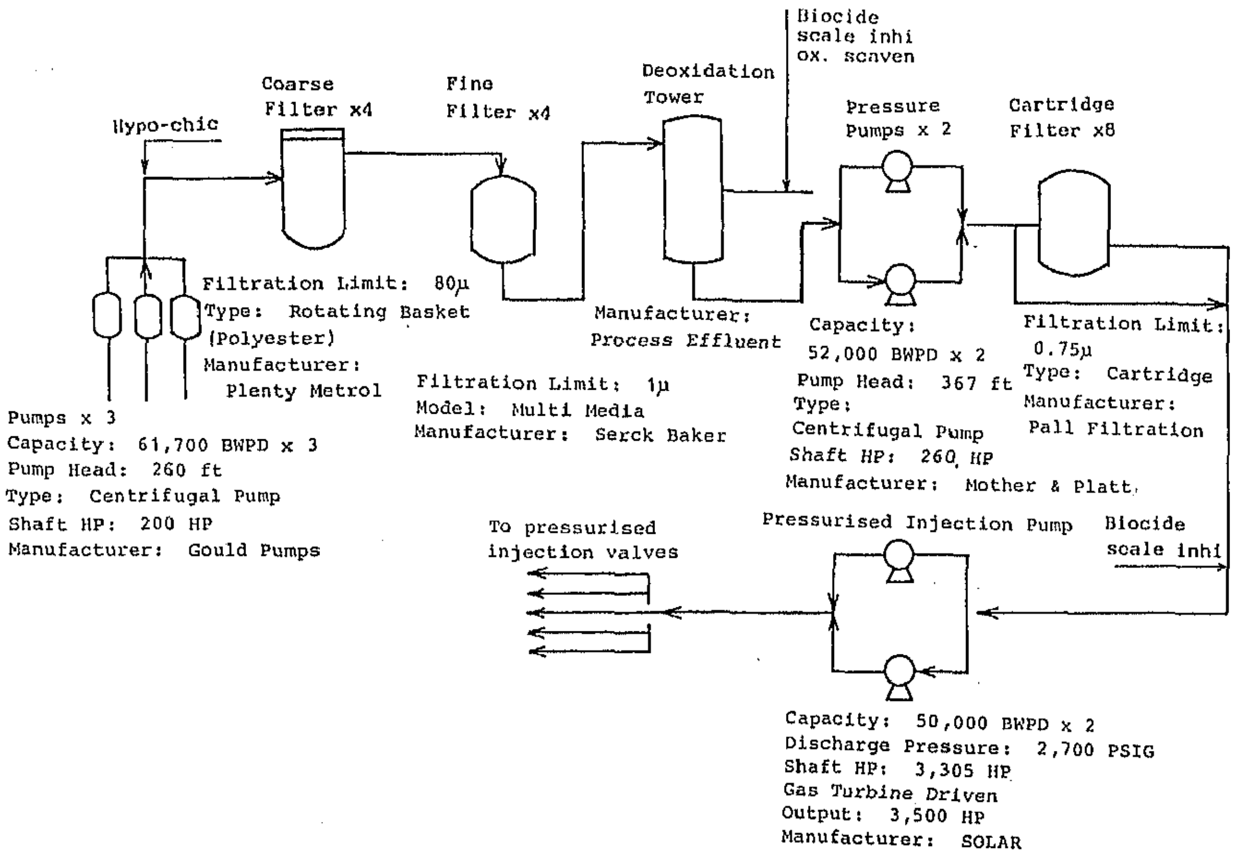


Fig. 4.1.9 Pretreatment Process for Injection

2) Points for Careful Consideration

While submerged pumps are used for pumping up the water from the extraction layer, pipes are used in various parts of the pretreatment process. Based on past experience, the following points should be carefully considered, including measures against anerobic bacteria.

- o Reinforced vinyl chloride pipes should be used instead of steel pipes which are liable to corrode. Fibre glass pipes or other pipes with similar strength should be used when extra strength is required.

- o The use of oil-less submerged pumps which are installed in the extraction layer is preferred. In most cases, the inclusion of oil cannot be avoided even when the utmost care is taken.

- o The groundwater level should be continuously monitored during the operation of the submerged pumps in order to avoid the breakdown of the motor due to idle running of the pumps which in turn is caused by the excessive lowering of the groundwater level.

- o As pretreatment is often not an effective enough sterilisation measure against bacteria, the propagation of bacteria tends to occur as soon as the constant water flow between the injection well and the injection layer stops, often resulting in clogging. Therefore, the standard provision of an auxiliary generator, etc. is required to prevent any stoppage of the water flow in the well. This requirement applies to both aerobic and anerobic bacteria. The mixing of sand grains not only causes clogging but also pump breakdowns.

3) Non-Drainage Method in Niigata Gas Field

While water-soluble natural gas is extracted from aquifers of unsolidified diluvium and alluvial deposits, there is a problem of possible land subsidence over a large area due to the extraction of a huge volume of groundwater containing gas. This land subsidence is prevented by recharging the water separated from the natural gas to the original aquifers.

Fig. 4.1.10 shows the structure of the pressurised injection wells which were used for the feasibility test in the development of the Niigata gas field to confirm the possibility of natural gas extraction while preventing possible land subsidence. (Water-Soluble Natural Gas Extraction Tests by Non-Drainage Method in Niigata, Second Interim Report, June, 1985)

(3) Injectivity Index

When water is injected into a deep aquifer through an injection well (bore hole), the pressure of the injected water (bore hole bottom pressure) must be higher than the water pressure in the injection layer. The actual pressure value required depends on the injectivity index (II), which is in turn determined by the injected water volume per unit time (day in the present example).

The volume of water extracted from a well (extraction well or production well), which is the source of injection water, depends on the productivity index (PI) of the well concerned. This index must, therefore, be thoroughly studied by test boring.

When the PI of the injection layer is known, its relation with the II of the same layer can be used as an index for injection (in the case of the Niigata gas field for example, II = 1/5 PI).

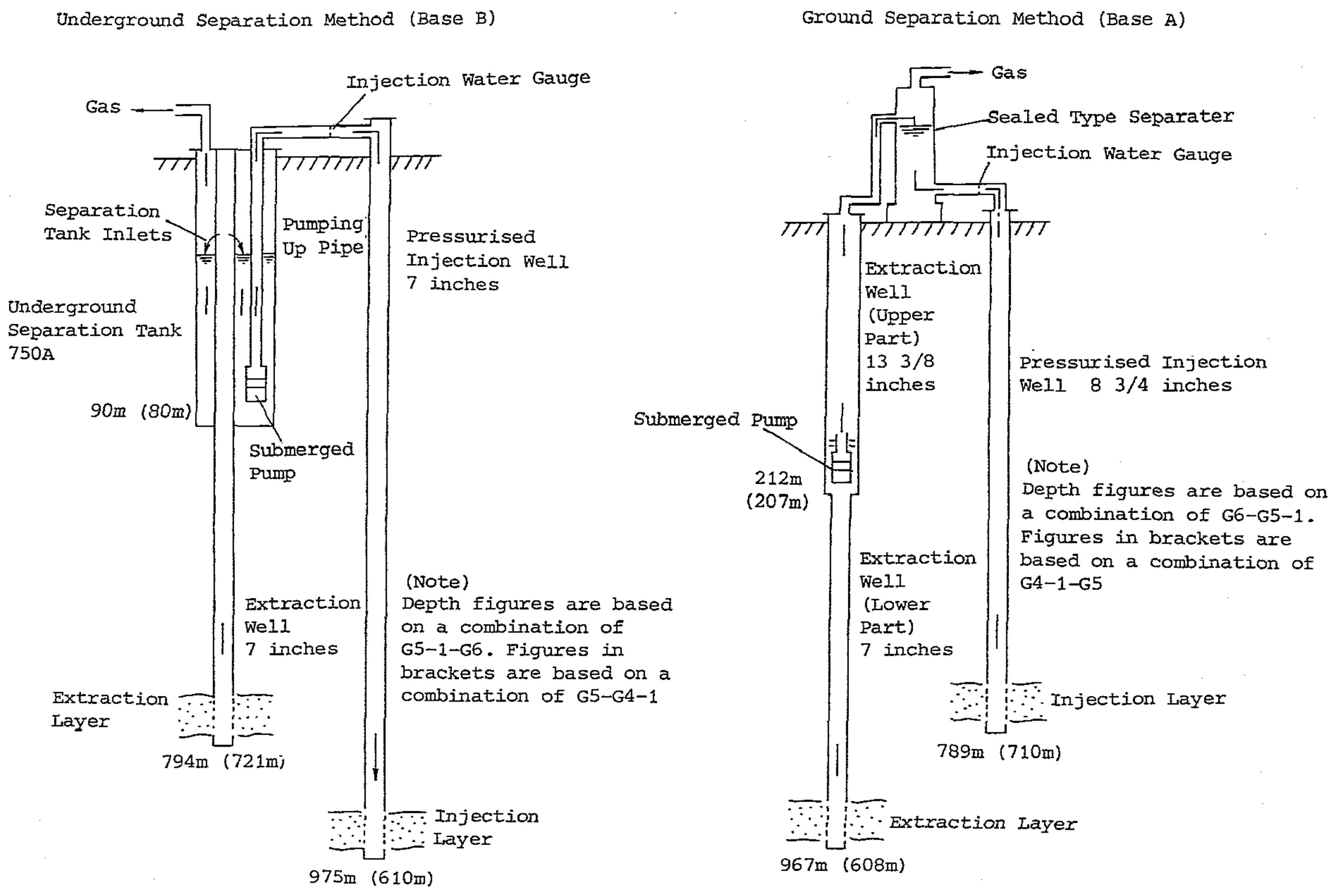


Fig. 4.1.10 Schema of Experimental Injection Wells in Niigata Gas Field

1) Injection Layer

The value of the injectivity index is determined on the basis of either theory or experience. In general, the latter is used as the theoretical value is not applicable in many cases.

1 Determination of II Based on Experience

$$II = \frac{q_{sc}}{Pw - Pe} \text{ (bbl/day/psi)}$$

q_{sc} : Injection volume (bbl/day); $q_{sc} = Pi.II$

Pw : Well Bottom Pressure (psi); Equivalent to injection pressure

Pe : Water Pressure of Injection Layer

Pi = Pw - Pe : Injection Pressure Difference

2 Theoretical Determination of II
In accordance with Darcy's Law,

$$q_{sc} = \frac{7.08Kh\ (Pe - Pw)}{\mu\, Bo\ \ln\ (\Gamma e/\Gamma w)}$$

$$II = \frac{7.08Kh}{\mu\, Bo\ \ln\ (\Gamma e/\Gamma w)}\ .$$

where,
K : Permeability (Darcy)
h : Layer Thickness (ft)
μ : Viscosity (cp)
Bo : Formation Volume Factor
Γe : Injection Radius (ft) (radius of influence)
Γw : Well Radius (ft)

3 Specific Injectivity Index (SII)

The concept of SII is often used in the comparison of wells. SII is the value of II per unit thickness of the layer.

$$SII = \frac{II}{h} = \frac{q_{sc}}{h\cdot (Pw - Pe)}$$

4 The injection water volume is, therefore, obtained by the following equation.

$$q_{sc} = (Pw - Pe)\ II$$

In practice, the water pressure or the head of the injection layer (Umm er Radhuma layer) should first be calculated and the value of II then calculated based on the relation between the injection pressure and the injection water volume, taking the depth of the injection well into account. Depending on the value of II, water will be immobilised in the injection layer.

2) Extraction Layer

With regard to the extraction layer, the productivity index (PI) should be calculated in advance. The PI index shows the productivity of one extraction well (or production well) and the oil volume (or water volume) produced per day (kl/day) when the oil pressure (or water pressure) of the extraction layer is reduced by 1 kg/cm^2. This index is required to determine the number and the distribution of the extraction wells, which in turn are required to lower the groundwater level in the extraction layer (Upper Dammam layer) to the desired level and to maintain that level. (Similarly, with regard to the injection layer, the number and distribution of the required wells can be determined based on the permissible water volume for one well.)

$$PI = \frac{q_o}{Pws - Pwf}$$

where q_o : Produced Oil Volume (kl/day) Measured at Ground Tank (Stock Tank Oil - STO)

Pws : Sealed Well Bottom Pressure (kg/cm^2)

Pwf : Artestian Well Bottom Pressure (kg/cm^2)

Psw - Pwf = Pi : Differential Pressure (draw down pressure)

4.2 Groundwater Drainage Methods and Application

The facilities required for groundwater drainage basically consist of water collection facilities, water transfer facilities and disposal facilities. Here, the direct drainage methods to lower the groundwater level are examined. In principle, the following items should be considered to establish the judgement criteria for the applicability of drainage systems.

1. Implementation feasibility
2. Collection efficiency
3. Water quality (prevention of infiltration by seawater)
4. Land use constraints
5. Workability
6. Cost

4.2.1 *Classification of Groundwater Drainage Methods*

Groundwater drainage methods are largely classified into vertical and lateral drainage, depending on the direction the collection system runs. The collection efficiency improves in proportion to the surface area size and various techniques are employed to achieve the maximum efficiency. Fig. 4.2.1 shows a systematic classification of the main methods and Fig. 4.2.2 gives illustrations of these methods.

4.2.2 Groundwater Mound Drainage Methods

(1) Tunnel Type

The tunnel type of drainage has been a strong candidate from the early stage of the study. Lateral collection channels are provided in the tunnel and the groundwater is collected in the tunnel and drained by gravity flow.

i) Feasibility

Based on the field study results and the available geological data, the tunnel should be located more then 20 m below the ground surface to avoid cave-ins during the construction work. A geological survey, however, is required in view of conducting a concrete examination.

ii) Collection Efficiency

Even if there is anisotropy or permeability fluctuations in the area where the groundwater mound is located, systematic drainage over a wide area is possible due to the adequate contact area and facility length.

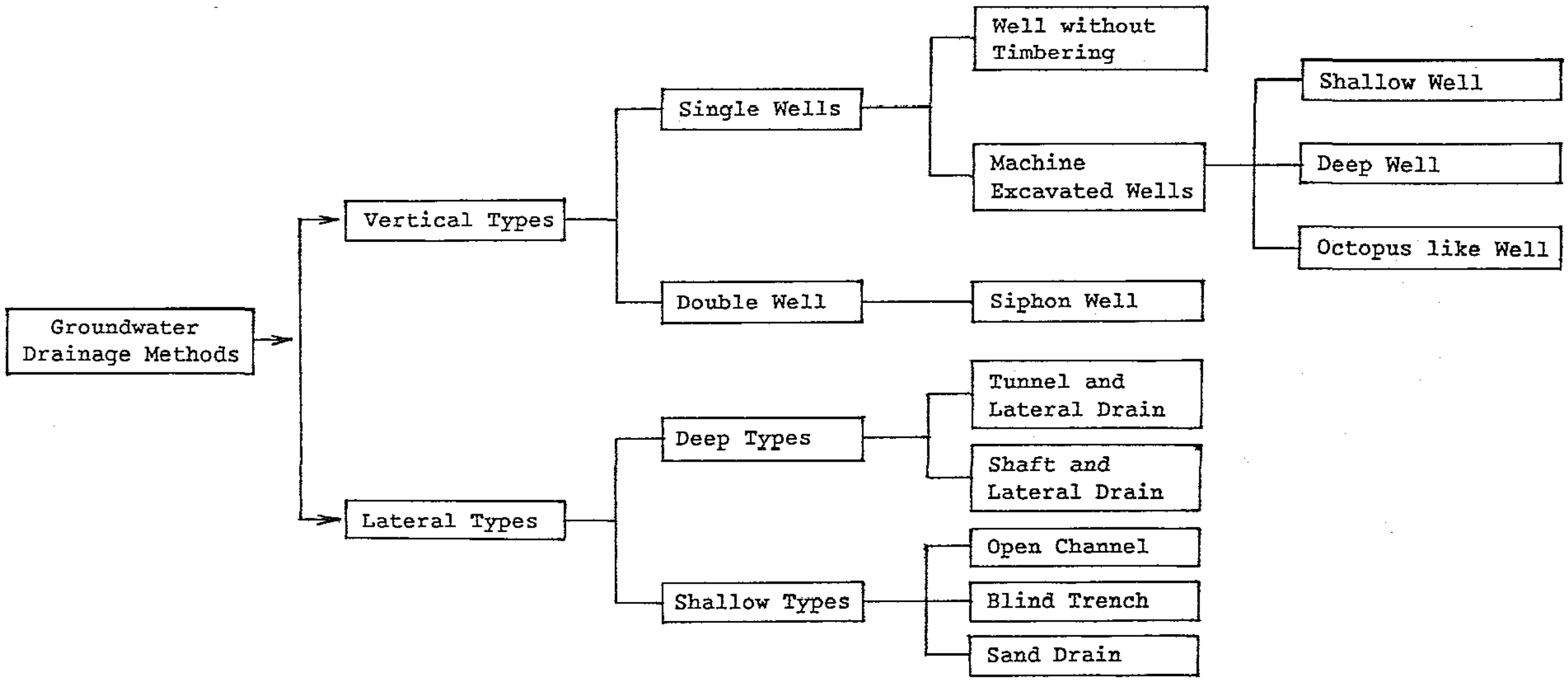


Fig. 4.2.1 Systematic Classification of Groundwater Drainage Methods

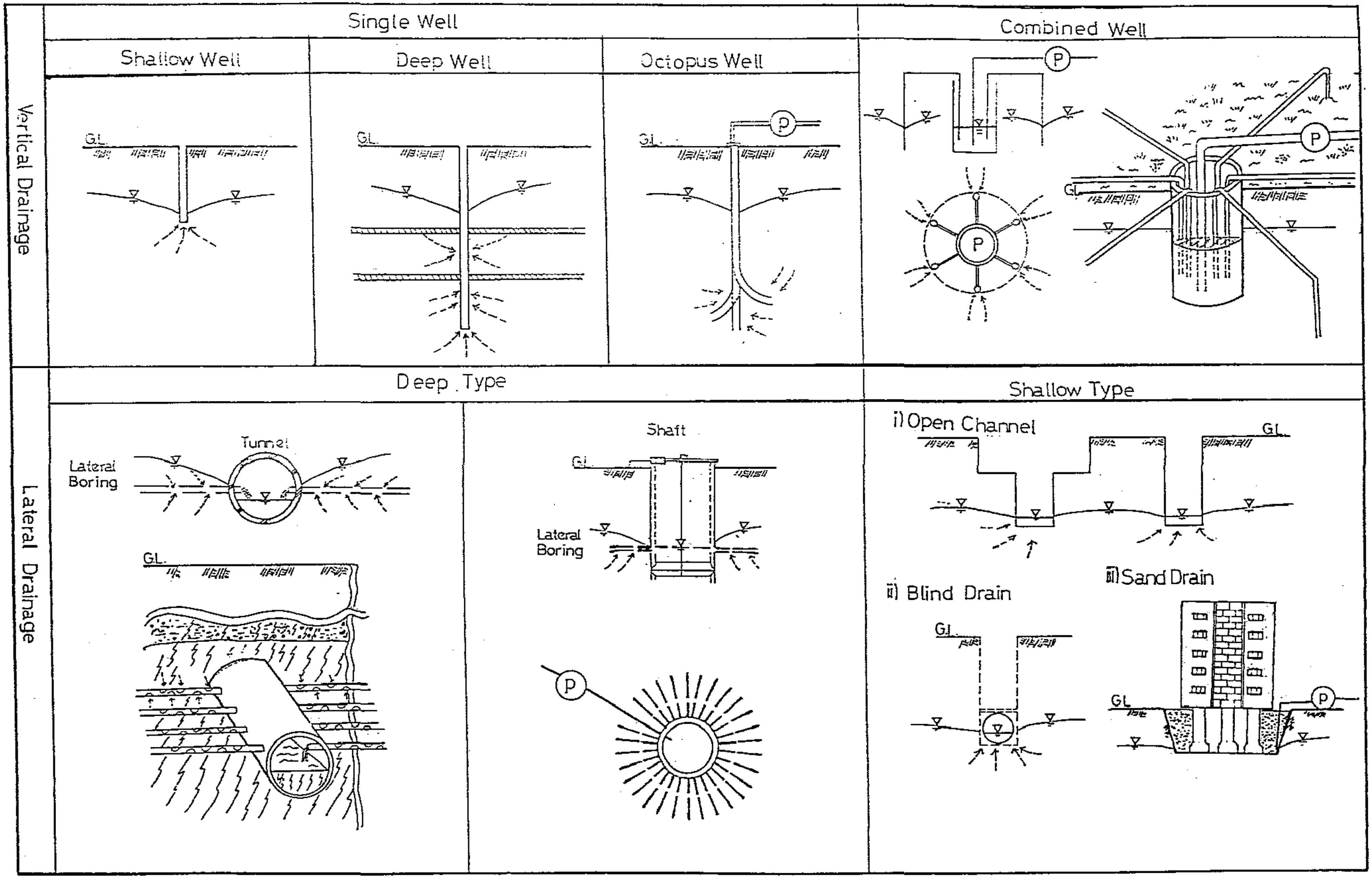


Fig. 4.2.2 Sketch of Drainage Methods

iii) Water Quality

As the excavation depth is deep in view of the necessity of providing the tunnel with adequate earth cover, the water quality tends to be poor. In addition, collection using a tunnel, may provoke the intrusion of seawater.

iv) Land Use

There are few constraints on the land use due to the adoption of the tunnel type of drainage.

v) Workability

As excavation is conducted by blasting, the work will generate vibrations. Water drainage is required as the work is carried out below the groundwater level.

vi) Cost

Assuming a total tunnel length of 16 Km and a unit cost of 2 million yen/m, the total construction cost is 32 billion yen (approximately 800 million Qatar riyals). This is the most expensive method of the currently proposed methods.

(2) Shaft Type

The shaft type of drainage requires lateral collection holes of 20 - 30 m in length which radiate from the bottom of the shaft to increase the catchment. The groundwater flows into the shaft via these holes and is then transported by either tankers or through conveyance pipes.

i) Feasibility

As the permeability of the groundwater mound area is not uniform, appropriate sites for efficient water collection by the shaft type drainage method must first be selected.

ii) Collection Efficiency

The shaft type drainage method is basically a local measure although it may be considered as a main measure if a large number of appropriate sites can be found. In terms of the maintainability, any reduction in the collection volume due to the clogging of the lateral holes can be easily dealt with by excavating new holes. When tankers are used to transport the collected groundwater, however, the effects of this method on lowering the groundwater level are minimal due to the constraints posed by the limited transportation capacity of the tankers.

iii) Water Quality

The quality is the same as the quality of the water in the groundwater mound.

iv) Land Use

Only a small area of land is required for the shaft. If a tanker transportation system is adopted, however, sites will be required for a tanker refuelling facility and parking.

v) Workability

No specified problems are anticipated.

vi) Cost

Shaft (10 x 10 x 15 m)	18 million J YEN
Lateral Channel (30 x 30 M)	54
Tanker Station	24
Total	96 million J YEN

(3) Well Type

This type of drainage is excluded from the present examination as it is difficult to excavate a number of wells with a high collection efficiency in a groundwater mound area.

(4) Open Channel and Blind Trench Types

The distance between the ground surface and the groundwater level is deeper in a groundwater mound area than in a lowland area, resulting in a high construction cost. In view of the fact that groundwater problem does not arise in the area, these types of drainage are excluded from the present examination.

4.2.3 Lowland Drainage Methods

(1) Tunnel Type

Since the weathered layer in a lowland area is thicker than that in an area of high ground, the location of the tunnel should be more than 15 m below the sea-level in view of the required earth cover thickness. The tunnel may also collect seawater at this depth and, therefore, this type of drainage is judged to be unsuitable for a lowland area.

(2) Shaft Type

A shaft type drainage facility could be possibly introduced at a roundabout or on public land. The structure has already been described in 4.4.2 and this type is suitable for the local drainage of groundwater.

(3) Well Type

There are two conceivable ways of utilising well type drainage facilities, i.e. the use of existing dug wells and drilled wells or the excavation of new wells for the sole purpose of extracting groundwater. A huge extraction amount is required if the groundwater level is to be lowered over a wide lowland area. The groundwater extracted from the wells can be disposed of by the following two methods.

1. Well ⟶ Branch Transfer Pipe ⟶ Pump Station ⟶ Trunk ⟶ Transfer Pipe

2. Well ⟶ Branch Transfer Pipe ⟶ Water Treatment before Injection ⟶ Injection to Deep Well

i) Feasibility

As the lowering of the groundwater level by the use of wells is a well-known phenomenon, this method is quite feasible.

ii) Collection Efficiency

As the subject area is characterised by limestone, there is a question of how often the wells can successfully hit the groundwater veins such as fissures, vugs, etc. in the layer. In regard to the wells, unless the drawdown depth in the wells is very deep, the influence radius of drawdown is smaller than that of open channels or blind trenches due to the flow characteristics of the groundwater. Therefore, these wells tend to be quite deep, often collecting water from outside the subject area where the lowering of the groundwater level is originaly intended. The extraction amount consequently tends to be large.

iii) Water Quality

As the wells tend to be rather deep, as described in ii) above, it is assumed that the quality of the water will be poor.

iv) Land Use

Although no specific problems are anticipated, the cooperation of the private sector should be obtained in the case of a wide subject area.

v) Workability

There is a great amount of experience to draw on in regard to the well drilling.

vi) Cost

Although the construction cost involved is the cheapest of the proposed methods, the maintenance cost is high due to the large number of wells.

(4) Open Channel Type

The groundwater is collected in the channel from the sides and is drained by gravity flow. In the case of a lowland area, a pump station is required to transport the collected groundwater.

i) Feasibility

Open channel drainage is practiced in limestone on the agricultural field. It can cope with any size of subject area by changing the size of the channel network.

ii) Collection Efficiency

As fissures run through the limestone with an interval distance between them of some 10 m, the influence of permeability deviations on the collection efficiency is minimal. The open channels are easily maintained and the collection is easy to check. As these channels mainly collect water from shallow layer, an increase of the drainage volume due to additional drainage from deeper layer can be largely minimized.

iii) Water Quality

In general, shallow groundwater tends to have better quality than deep groundwater, reflecting their specific gravities, and the water in question is therefore good.

iv) Land Use

There are some constraints on the land use in view of securing sites for the channels. In addition, some damage may be caused to the local scenery by the dumping of rubbish in the channels.

v) Workability

In view of past experience, the workability is judged to be good.

vi) Cost

Both the construction and maintenance costs of open channels are higher than those of blind trenches.

(5) *Blind Trench Type*

Since there are some constraints on the land use when open channels are used, these open channels could be changed into blind trenches by filling them with permeable material. The feasibility, collection efficiency, water quality and workability of blind trenches are almost the same as in the case of open channels. However, while blind trenches are advantageous in terms of land use, their construction cost is higher than that of open channels. Nevertheless, as the land use value is given priority over the construction cost in the implementation of necessary measures for a rising groundwater level in an urban area, the blind trench type of drainage is considered to be superior to the open channel type.

4.2.4 Summary of Drainage Methods

Table 4.2.1 compares the various drainage methods which could be used to prevent the rise of the groundwater level over a wide area. If the land use constraints are ignored, the open channel method is the most suitable as it has been extensively used in agriculture and is of proven reliability. In the case of drainage in an urban area, such as Doha, however, the possible locations for the facilities are limited. Therefore, the blind trench method, which is superior in terms of land use, should be adopted.

While the well method also has advantages, there is a difficulty in effectively lowering the groundwater level in the shallow layers which has a direct relation to the rising groundwater level problem. In addition a number of wells are required to lower the groundwater level. As a pump should be installed at each well, maintenance is difficult and expensive. The tunnel method is the most appropriate method of lowering the level at a groundwater mound but has a much higher construction cost than the other methods and carries a high risk of seawater intrusion.

If the main objective is to lower the groundwater level at specific local sites or groundwater reuse, the shaft and lateral drain method, siphon method and sand drain beside a building, etc. may be used. The shaft or the main well of the siphon method can act as a reservoir and, therefore, there is a strong likelihood of their being employed if the reuse of groundwater is intended.

Table 4.2.1 Comparison of Drainage Methods

	Tunnel	Well	Open Channel/ Blind Trench
Application Area	Groundwater mound	Lowland	Lowland
Collection Method	Lateral drain inside tunnel	Strainer section of well	Sides and bottom
Collection Style	Gravity flow	Pumping up	Gravity flow
Feasibility	Geological survey required	Possible	Previously used
Collection Efficiency	High risk of sea-water infiltration	Inferior collection efficiency of shallow layer groundwater	Efficient collection of shallow layer groundwater
Water Quality	Poor	Not so good	Passable
Land Use	No problems	Cooperation of private sector required	Blind trench: no problems Open channel: problems exist
Workability	Vibrations during work, drainage	Previously used	If blind trenches are constructed under existing roads, the temporary closure of these roads is necessary
Cost	Highest	As many wells are required, the maintenance cost is high	Open channels are cheaper than blind trenches

4.3 Groundwater Recharge Reduction Measures

The measures to reduce the groundwater recharge would have no direct effect on the lowering of the current high groundwater level. However, they would have indirect effect by preventing the further rise of the groundwater level. These measures can be classified as shown in Fig. 4.3.1.

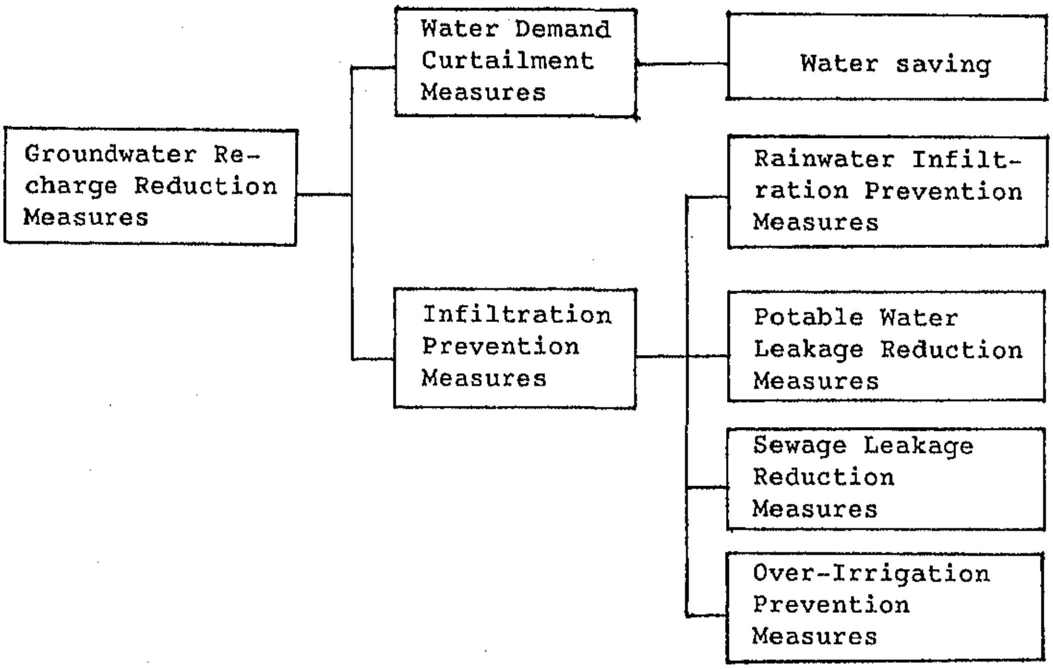


Fig. 4.3.1 Classification of Groundwater Recharge Reduction Measures

4.3.1 Water Saving

Water saving has the following effects in terms of groundwater recharge reduction.

1. The potable water supply volume is reduced in accordance with a reduction in the water demand, thus reducing the leakage volume.

2. The generated sewage is reduced in accordance with a reduction in the water consumption, thus reducing the leakage from cesspits and septic tanks.

3. The irrigation volume is reduced in accordance with an increased awareness among citizens of the problem of over-irrigation, thus reducing the irrigation return.

In order for water saving to be effectively carried out, public relations activities and educational activities in schools and offices, etc. are necessary in view of informing the public of the seriousness of the rising groundwater level problem and how water saving can be achieved. However, since most people derive great satisfaction from using plenty of water, it may be difficult for them to accept water saving measures. In addition, as Doha's population consists of a variety of races with different languages, these activities should be carried out in various languages using expressions which are easy to understand.

Although public relations activities are the basic starting point for water saving, the introduction of a water charge may achieve the same objective. Water is currently supplied to the Qatari citizens free of charge and the introduction of a water charge may, therefore, raise the public's consciousness of the value of water but may also impose a financial burden on some people. Therefore, the utmost care should be taken in any decision to introduce a water charge.

According to the Japanese experience, the effect of water saving measures on the water demand is a maximum of 10%. In view of the fact that the water consumption is steadily increasing in Doha at the present time, the water demand will remain steady at best or even show a slight increase even if water saving measures are successfully implemented .

4.3.2 Rainwater Infiltration Prevention Measures

Groundwater recharge due to rainwater is much less than recharge due to potable water. Consequently, rainwater infiltration prevention measures do not qualify for a separate project. The current Wadi Musherib Stormwater Drainage Project aims at preventing the occurrence of standing water at roundabouts in view of preventing traffic disturbance. So far, the installation work on the section between the Arab Bank Roundabout and the Cable and Wireless Roundabout has been almost completed and work is expected to continue upto Wadi Musherib Dam. As the stormwater trunk channels are designed to have a capacity capable of accommodating strong rainfall intensity during storms in dry areas, they can also be used as transfer channels for groundwater drainage.

4.3.3 Potable Water Leakage Reduction Measures

The objectives of potable water leakage reduction measures are the effective utilisation of valuable water and the reduction of potable water infiltration underground by preventing leakage. As water supply facilities deteriorate with age, new leakage tends to occur despite the repair of existing leaks. In the case of the Tokyo Metropolitan Government, the leakage rate is still some 15% despite the fact that leakage prevention measures and maintenance checks are implemented every year. According to the existing survey results, the rate of leakage from potable water distribution system in Doha is some 20%. Therefore, the room for improvement of the leakage rate is considered to be a maximum of 5-10%.

4.3.4 Sewage Leakage Reduction Measures

Groundwater recharge due to sewage takes two forms, i.e. the infiltration of sewage from cesspits and septic tanks and the leakage from the sewerage system located higher than the groundwater level. Sewage leakage reduction measures are more effective in the case of the former than the latter.

The current capacity of the sewerage system is based on $0.27m^3$/day/person for a subject population of 160,000 while the future expansion plan is designed for a subject population of 210,000. At present, the overflow of sewage onto roads is sometimes seen due to the reduced infiltration from cesspits, in turn caused by the rise of the groundwater level.

4.3.5 Over-Irrigation Prevention Measures

Trees provide the urban environment not only with beautiful scenery and greenery but they are also restful for the eyes. Irrigation, therefore, is indispensable for their upkeep. As Doha has a very dry and hot summer, irrigation water quickly evaporates and the salinity of the soil water tends to increase, thus necessitating additional irrigation.

Over-irrigation means the recharge of the groundwater due to excess irrigation water which is not required or absorbed by the plants and trees. While it is not difficult to understand the concept of over-irrigation, such questions as how a reduction in the groundwater recharge can be achieved and whether or not plant growth is affected by reduced irrigation, etc. must be answered. A study in this field has not yet been sufficiently carried out in Doha and, therefore a systematic study will be necessary in the future with the following main study items.

1. Actual irrigation water volume
2. Improvement of irrigation method
3. Improvement of plants to increase their salt-resistance
4. Development of salt density control method depending on quality of water used
5. Measurement of underground infiltration originating from over-irrigation
6. Establishment of adequate irrigation water requirements for each species of plants and trees

As described above, measures should be established to effectively prevent over-irrigation and the public should be persuaded to accept these measures.

4.3.6 Comparison of Groundwater Recharge Reduction Measures

As many groundwater recharge reduction measures have overlapping objectives, the comprehensive implementation of these measures will contribute to a long-term and wide area solution to the problem of the rising groundwater level in Doha. The characteristics of the measures described in 4.3 are summarised in Table 4.3.1.

Some of the leakage prevention measures almost permanently require maintenance or improvement in view of the characteristics of the facilities involved. In addition, public relations activities for water saving and technical instruction on over-irrigation prevention, which may be described as the software aspects of these reduction measures, should also be implemented. In this context, it is wrong to assume that water saving measures are very inexpensive as the preparation of public relations videos and the management of a water saving campaign office, etc. require specialists in the respective fields.

Table 4.3.1 Comparison of Groundwater Recharge Reduction Measures

	Water Saving	Rainwater Drainage	Potable Water Leakage Reduction	Sewage Leakage Reduction	Over-Irrigation Prevention
Main Objective	Effective water utilisation	Solution for traffic jams due to rainfall	Effective water utilisation	Improvement of daily life infra-structure (improvement of public hygiene)	Effective water utilisation
Minimum Effect in Terms of Groundwater Recharge Reduction	Suppression of water demand increase. Maintenance of current demand level	Drainage from road sections only. Infiltration from bare ground cannot be prevented	Although 5-10% of the current supply may be saved, it will be offset by the increase in the demand	Only the sewage in areas with a low sewerage service ratio is affected	Accurate under-standing of the current situation is first required. Measurement of the effect is is difficult
Time Required for Measures to be Effective	A long time is required for water saving PR activities to show their effects	Construction of the required facilities takes a long time	The effects depend on the balance between repaired sections and damaged sections. These measures may take the longest time of all measures to show their effects	Because of the good workability and economical aspects, can be easily implemented	A long time is required for the basic study
Cost	Cheapest as no facilities are required	The most expensive as large size facilities are required	Semi-permanent facility improve-ment work will be required as long as potable water is used	Cheap	Relatively cheap as only the study cost and partial improvement of the existing facilities are required

4.4 Urgent Measures for Damaged Areas

4.4.1 Urgent Measure Qualifications

The longer the implementation of an urgent measure is delayed, the more damage is incurred. The objective of an urgent measure is the prevention of the further rise of the groundwater in order to prevent the progression of damage as early as possible. To achieve this, the direct control of the groundwater level in damaged areas is necessary and, therefore, an urgent measure should have the following qualifications.

1 Necessity of the measure
2 Positive results in the past (reliability)
3 Safety

As the design, implementation and testing processes are required after the decision has been made on a measure, 2 or 3 years are required for the measure to start showing its effect. If a lot of time is spent on carrying out a study to prepare the urgent measure, the measure's original effect is lost. It is too late to be sorry if the propagation of mosquitoes in standing water or the solidification of salt on the ground surface cause a social problem.

An urgent measure should, therefore, be based on the drainage method which is deemed the most reliable method at present and should also conform with wide area measures to be implemented in the future. As described in 4.2.4, the blind trench method is the most suitable for an urban area.

4.4.2 Judgement Criteria for Adoption of Urgent Measure

The following criteria may be used to judge whether or not a specific measure should be implemented.

1 Whether or not it is more economical to abandon the damaged area and move buildings, etc.to new locations.

2 Whether or not a provisional measure, such as banking or the local pumping of the groundwater, can cope with the situation for an interim period.

3 Whether or not the required drainage volume necessitates the construction of groundwater drainage facilities.

4 Whether or not the structures in the area will be affected by the lowering of the groundwater level.

Criteria 1 - 3 are also criteria for the selection of a subject area for the implementation of an urgent project. With regard to 1 , the relocation of an urban area where a large amount of capital has already been invested is rather difficult although the decision may depend on the area's current land use. However, if the level of land use in a suburban area is low, the policy to abandon the area may be chosen. The provisional measures described in 2 above are only effective for a short period of time and, therefore, cannot provide a fundamental solution to the problem. As they can delay the progression of the damage, however, they are effective in the case where the damaged area is relatively small. The required drainage volume referred to in 3 can be calculated using the water balance or the simulated flow of the groundwater. The judgement on whether or not an urgent measure should be implemented may be made by considering a measure which corresponds to the required drainage amount. Fig. 4.4.1 shows the flow of the judgement criteria for the implementation of an urgent measure.

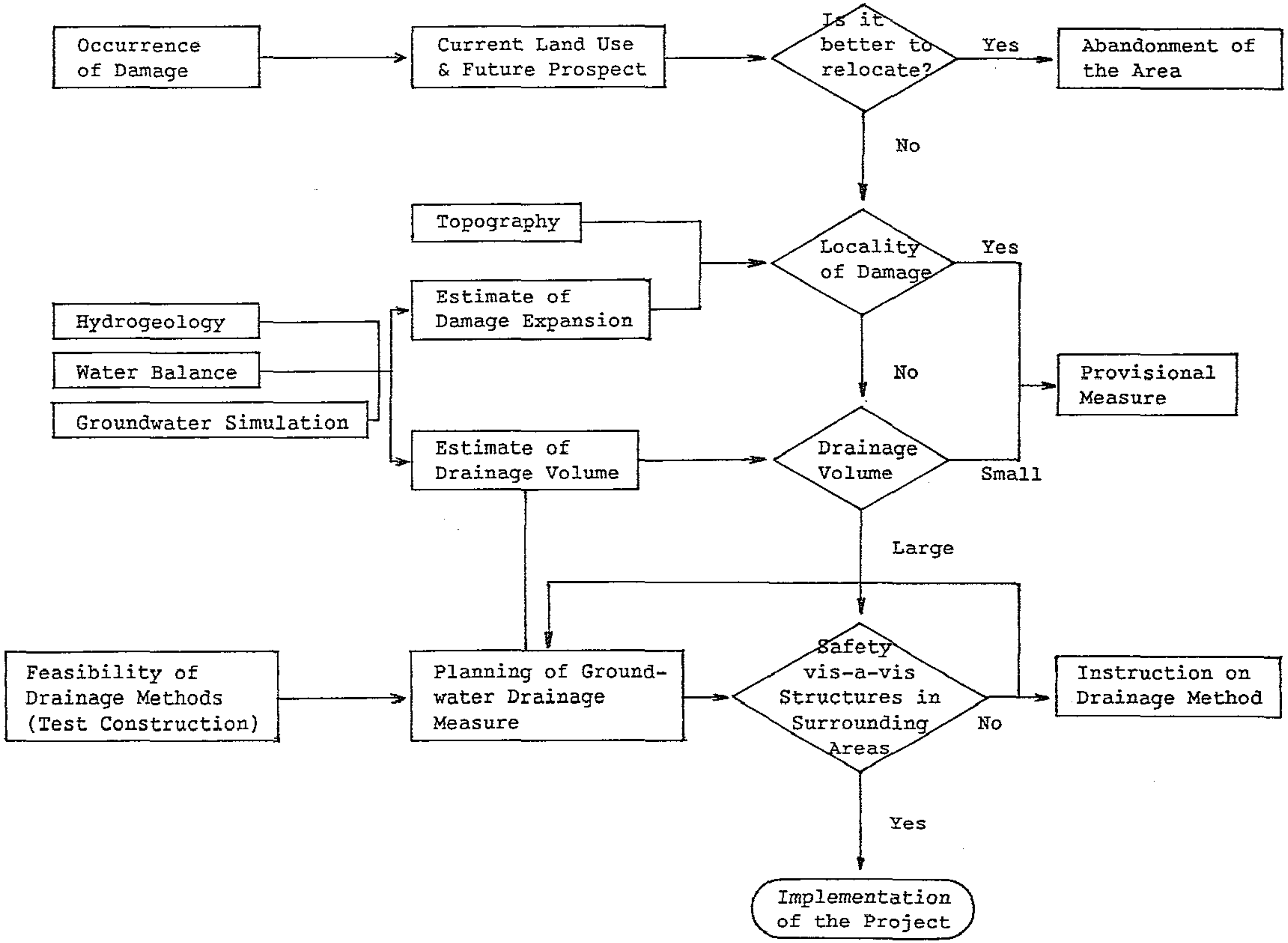


Fig. 4.4.1 Flow of Emergency Measure Implementation Process

4.4.3 Effects of Drainage on Existing Building Foundations

(1) Dissolution of Limestone

In the limestone layer, cracks are enlarged and grottos developed by the dissolution of the limestone. This process of dissolution is generally divided into the following 4 stages.

The $CaCO_3$ contained in the limestone layer is firstly dissolved into the water running through cracks.

$$CaCO_3 = Ca^{+2} + CO_3^{-2} \quad \ldots\ldots \quad (1)$$

$CaCO_3$ is a salt which is fairly hard to dissolve and which has a solubility product of (Ca^{+2}) (CO_3^{-2}) = 1.2×10^{-8} (room temperature). When the CO_2 in the air is dissolved in a saturate solution of $CaCO_3$, the following dissociation results and the solution shows acidity.

$$H_2O + CO_2 = H_2CO_3 \quad \ldots\ldots \quad (2)$$

$$H_2CO_3 = H^+ + HCO_3^- \quad \ldots\ldots \quad (3)$$

$$HCO_3 = H^+ + CO_3^{-2} \quad \ldots\ldots \quad (4)$$

As the value of the ionization constant is extremely small, the Co_3^{-2} produced by the dissolution of $CaCO_3$ in (2) produces the reverse reaction shown in (4) because of the influence of the common ion and, therefore, bonds with the H^2 to produce HCO_3^-. As a result, the equilibrium of reaction (1) is broken and the equilibrium subsequently shifts to the direction of increasing (Ca^{+2}) to compensate for the decrease of the (CO_3^{-2}) to maintain the value of the solubility product, i.e. the dissolution of $CaCO_3$.

In the reverse case where H^+ is removed from the solution, the $CaCO_3$ precipitates. The largest influencing factor in the dissolution of $CaCO_3$ is temperature. The dissolution of $CaCO_3$ accelerates with a low temperature, causing a lowering of the pH, which in turn results in an increased $CaCO_3$ solvency. Although a pressure increase facilitates the dissolution of $CaCO_3$, it is considered that pressure is not an important factor in an actual situation.

(2) Dissolution Speed

Since many factors are involved in the dissolution process, it is generally difficult to theorise the dissolution speed. However, a number of researches have been carried out in this field and several examples are introduced here.

Corbel (1959) established an expression to show the dissolved thickness of limestone using a unit of a millimetre per 1,000 years. A similar expression for limestone dissolution has also been presented by Groom and Williams (1965). Groom and Williams calculated the dissolution speed of limestone in an area in South Wales to be 15.77 mm/1,000 years using their expression. Using the same samples, however, a result of an average 40 mm/1,000 years was obtained using Corbel's expression. M. M. Sweeting (1966) conducted research on the dissolution speed of limestone in the north of England and obtained a result of 0.083 mm/year using Corbel's expression. All these results present no specific problem in terms of civil engineering geology.

(3) Subsidence Monitoring at Test Sites

The elevation of the groundwater level observation holes was surveyed both before and after the pumping test, i.e. April and December, 1986, at both test sites in Rayyan and Wadi Musherib. Although some of the observation wells were destroyed by vehicles during this period, making surveying at these wells impossible, the results at all other holes were within the surveying tolerance for civil engineering work (± 5 mm), showing no specific change in elevation.

(4) Aggregate Test

An aggregate test was conducted at each test site in Rayyan and Wadi Musherib to determine whether or not the weight of the aggregate placed in the groundwater would change due to the flow caused by the pumping test. The types of aggregate used were the Grade-II aggregate of the Civil Engineering Material Laboratory of the Ministry of Public Works and 20 mm diameter aggregate for concrete. Approximately 5 kg of samples were exposed in the groundwater from early July to late October during the pumping test period and they were then weighed. In short, the weight of the aggregate did not show any specific change in the 4 months of the test period (see Supporting Report E for details).

(5) Construction of Wadi Musherib Stormwater Trunk Line

Construction in work at Lower Wadi Musherib Stormwater Trunk Line is giving almost same effect as the proposed drainage scheme and there is no such claim or damage reported as far as study team investigated.

(6) Conclusion

Based on the examination above, no specific problem is anticipated with regard to the limestone layer which constitutes the foundation of Doha. It is, therefore, concluded that the groundwater discharge will not affect the existing buildings which are built on this foundation. However, it is clear that a local piping phenomenon may occur in those places where the foundation consists of a strongly weathered layer, a layer containing clayey sand or refilled ground. With regard to an urban area, such as Wadi Musherib, if a direct link between the potable water leakage and the drainage is established an excessive flow speed or flow rate may result. In view of this possibility, the monitoring of the flow speed and the flow rate will be necessary during the drainage work at the time of construction and after the commencement of full-scale drainage, as well as the adoption of geotextile sheeting to prevent the run-off of fine sand grains from blind trench walls. Supporting Report F and G give details of these aspects.

5. TEST WORK

5. TEST WORK

5.1 Preparation of Test Work

(1) Purpose of the Test

Pumping test in Trench aimed at obtaining more detailed information related to the effect of groundwater level lowering by Trench type dewatering and the reuse of abstracted groundwater.

The effect of groundwater level lowering by pumping test in Trench was evaluated through the groundwater level and abstraction amount observation.

With regard to water quality, the shallow part of the aquifer has low salinity in general according to the stratified structure of salinity and the Test Trench aimed at collection of low saline groundwater. The possibility of groundwater reuse and the necessity of water treatment were examined from the chemical analysis data.

For this purpose, the Test Work consisted of the following work.

- A trench having approximate length of 100 meters, width of 1 metre and depth of 4-5 metres
- Open holes for observation
- Core boring for geological information

(2) Selection of Test Sites

From the very beginning of JICA's commitment to the rising groundwater problem, this test work was considered as an actual demonstration test for lowering groundwater level in critical areas. Therefore at the stage when the existing studies were reviewed and preliminary reconnaissance was done, Lower Wadi Musherib and Old Rayyan were selected as test work sites where groundwater levels are actually almost at ground surface or flooded at some of the lower points.

Although locations for potential test sites were very limited, especially in Lower Wadi Musherib because of its proximity to the city center, two sites were kindly arranged by Doha Municipality, as follows:

- an open area just north of the S.E.D building at Lower Wadi Musherib
- an old farm of H.E. Sheik Khalid at Old Rayyan

(3) Undertaking on Test Work Construction

For the execution of the Test Work, the following required preparation works were assumed by both JICA and Qatar Government:

(i) JICA

- Test Trench construction including observation house and ground tanks
- Testing equipment such as pumps, flowmeters
- Hydrological measurement equipment

(ii) Qatar Government

- Open hole drilling for observation holes
- Electricity supply facility
- Road tankers for transportation of abstracted groundwater at Rayyan and sewerage line arrangement at Lower Wadi Musherib
- Automatic recording equipment for groundwater levels

(4) Program of the Test Work

The test work construction and pumping test were carried out according to the schedule in Table 5.1.1 and general plans in Fig. 5.1.1 and Fig. 5.1.2 herein attached at both sites respectively. Key events are dated as follows:

- Site selection - January '86
- Design and Specification of test schemes - January '86
- Test equipment preparation - February-April '86
- Trench excavation - March-June '86
- Open hole drilling - April-May '86
- Equipment installation - May-June '86
- Electricity supply - May-July '86
- Disposal arrangement - June-August '86
- Pumping test - July-October '86
- Reinstatement - November '86

(5) Handover of the Test Equipment

At the end of the test works, all the equipment and instruments listed below which were imported or procured by JICA for the test works were donated to the Ministry of Electricity and Water.

1) Submerged pumps with control panel and flowmeter

2) Engine pumps

3) Observation equipment
   - Temperature-humidity-barometer recorder
   - Wind recorder
   - Rain gauge
   - Evaporation pan

4) Chemical analyser
   - HACH
   - COD meter

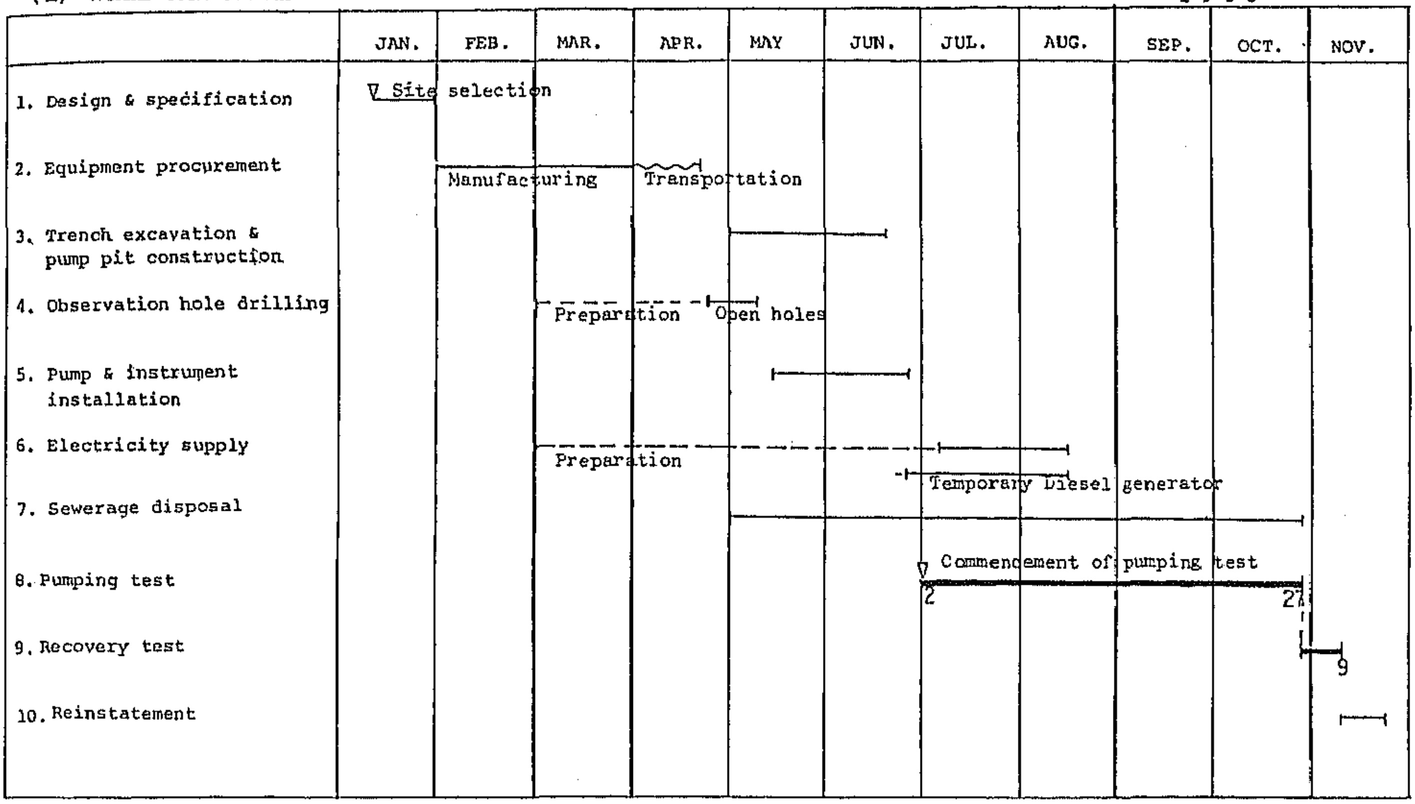
5) 5,000 Imp. gallon fibreglass water tanks

6) Room air conditioners

Table 5.1.1 Construciton and Pumping Test Schedule

(1) Wadi Musherib

1 9 8 6



(2) Rayyan

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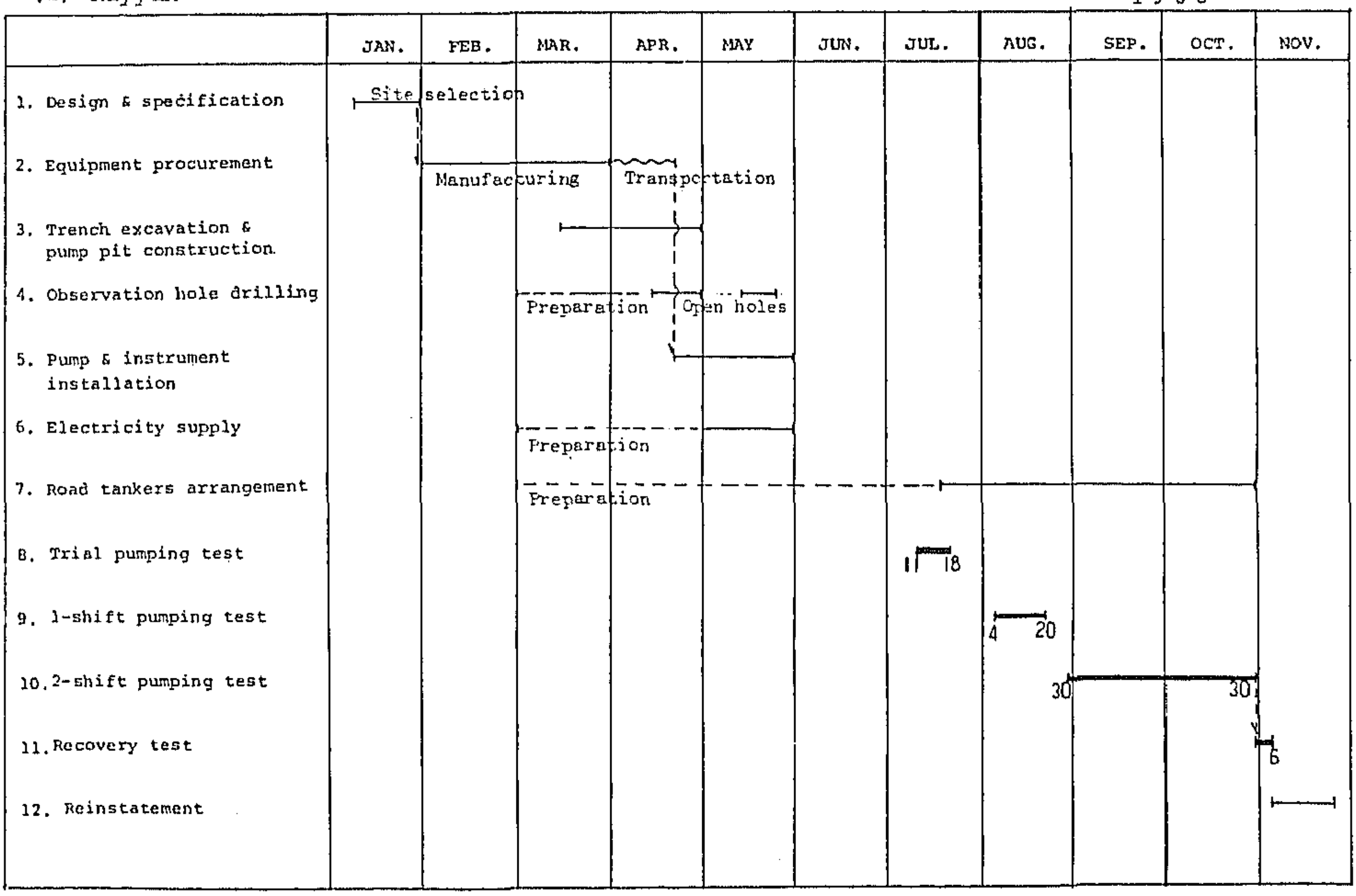

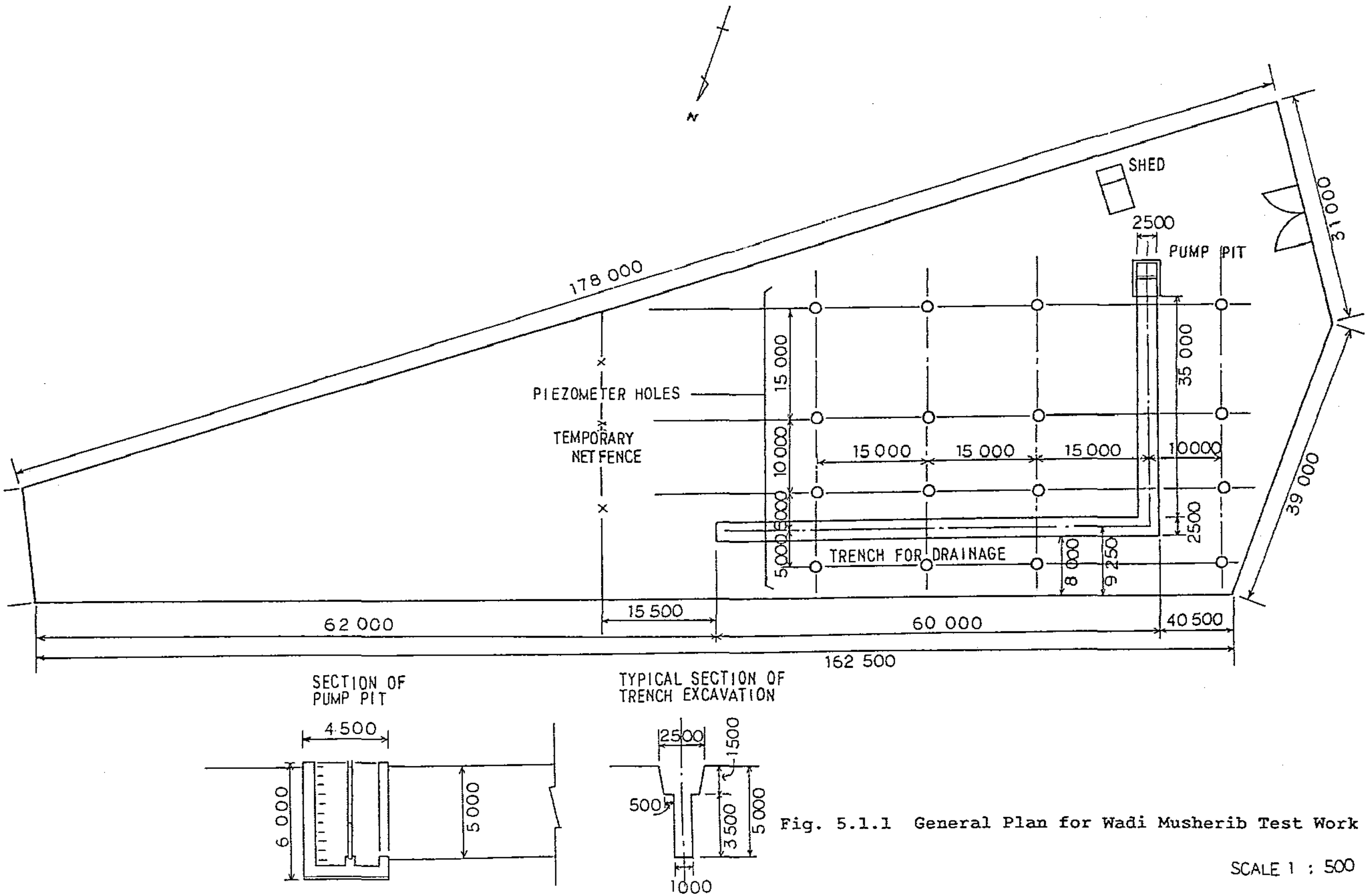

Fig. 5.1.1 General Plan for Wadi Musherib Test Work

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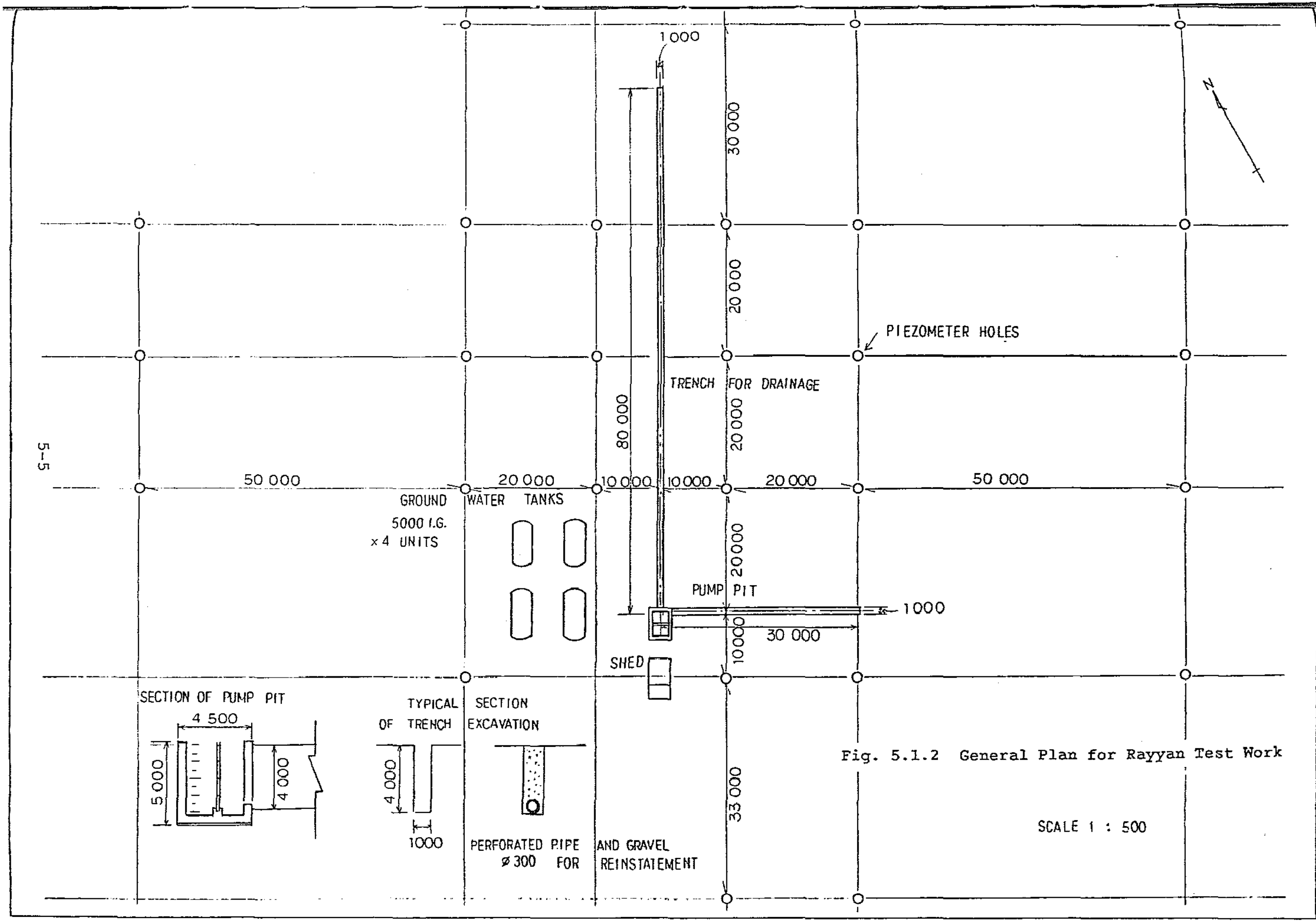


Fig. 5.1.2 General Plan for Rayyan Test Work

SCALE 1 : 500

5.2 Pumping Test at Lower Wadi Musherib

5.2.1 Continuous Pumping Test

(1) Activity

Pumping test commenced on the 2nd of July 1986 at Lower Wadi Musherib and continued until the 27th of October 1986 without any major stoppage. On the 16th of July, an abrupt partial blockage occured at the sewage outlet and since then the discharge amount was controlled to avoid any overflow from the sewage manhole. This blockage was cleared in mid-October and the discharge amount recorded thereafter was as the beginning of test.

(2) Groundwater Inflow to the Trench

At the end of the longer leg of the Trench, groundwater freely flowed out but in other parts of the Trench, distinctive flowing water channels were not found. However, small amounts of groundwater did seep out along bedding planes and irregular clay veins.

(3) Discharge Amount

The relationship between the water level at Trench and discharge amount can be summarized as follows:

- When the water level at the Trench was maintained nearly at the bottom, i.e., (1.05 - 1.20 m in QND), the discharge amount varied in the range of 300 - 400 m^3/day (0.21 - 0.28 m^3/min), as shown in Fig. 5.2.1.

- During the discharge limitation period from the 16th of July to mid October, most of the time the water level was in the range of 2.30 - 2.60 m in QND and the corresponding discharge amount was 230 - 260 m^3/day (0.16 - 0.18 m^3/min).

(4) Drawdown Depth

The outline of pumping test condition was as follow:

a. After 95 hours from the start of the pump test, the water level in the Trench reached the bottom i.e. 3.3 m draw-down.

b. From 95 to 320 hours elapsed time, a steady state was maintained with the water level at the bottom of the Trench.

c. At 320 hours, a blockage caused an abrupt decrease in the capacity of the sewage outlet. This resulted in a rapid increase in the Trench water level by 1.5 meters.

The drawdown depth distribution around the Trench and its profile are respectively shown in the Fig. 5.2.2 and Fig. 5.2.3.

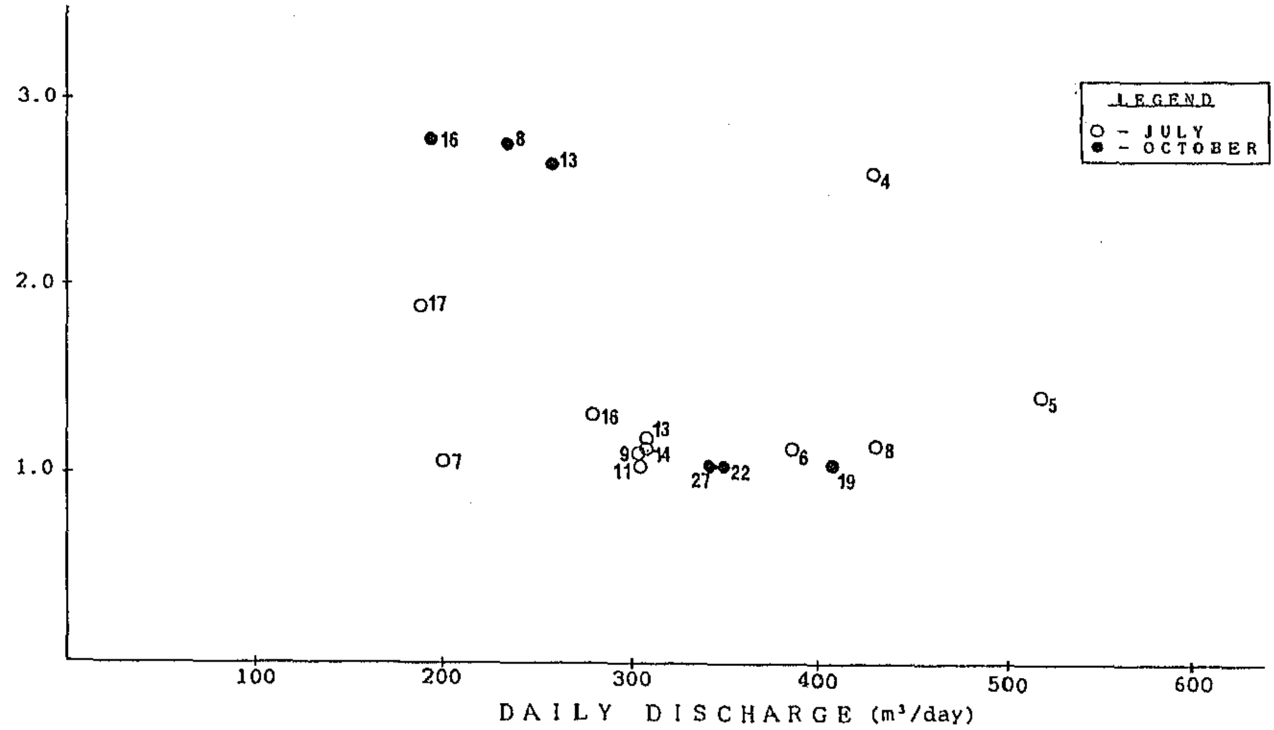


Fig. 5.2.1 Daily Discharge Amount at Wadi Musherib

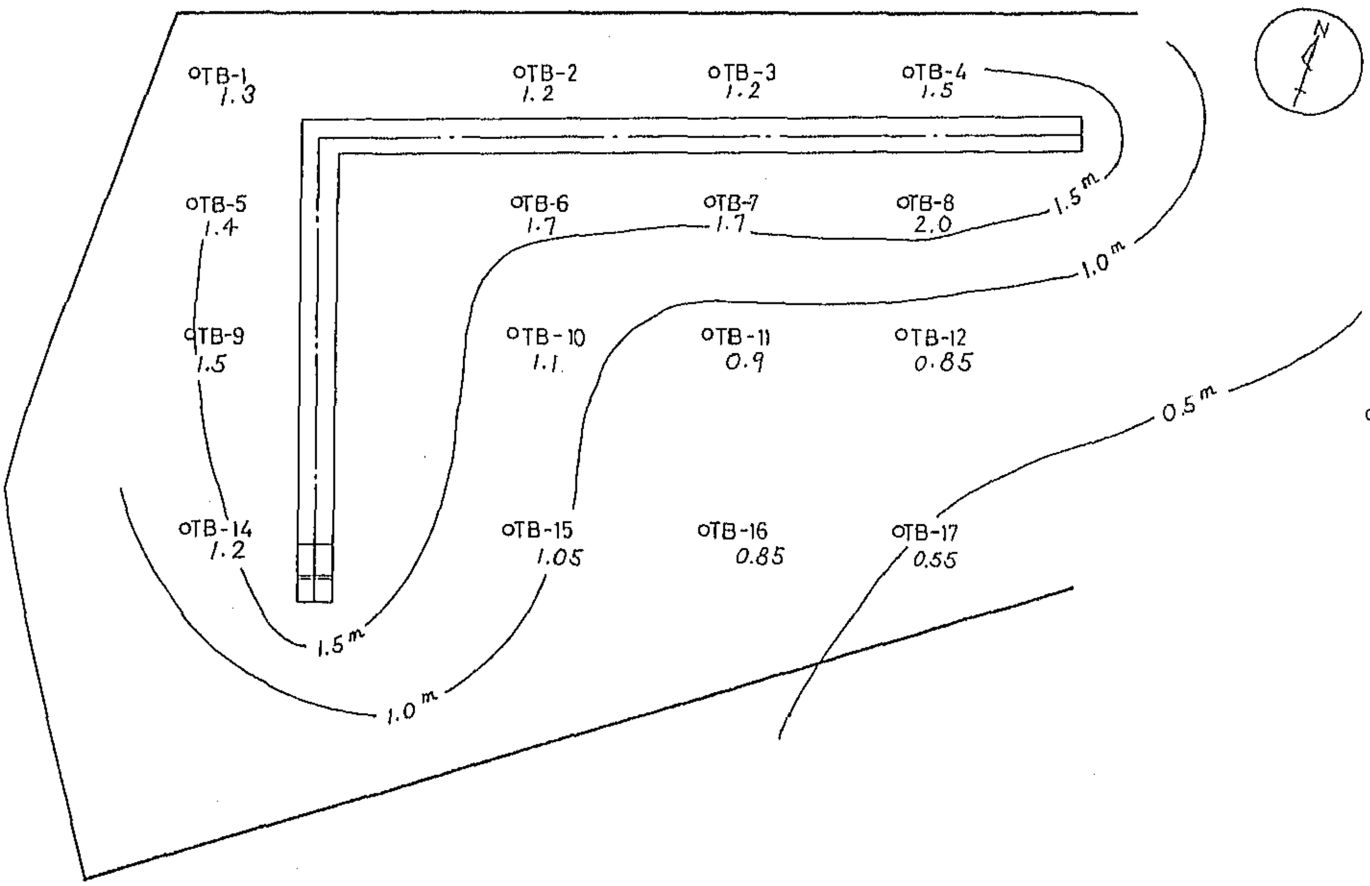


Fig. 5.2.2 Drawdown Depth Distribution around the Trench of Wadi Musherib

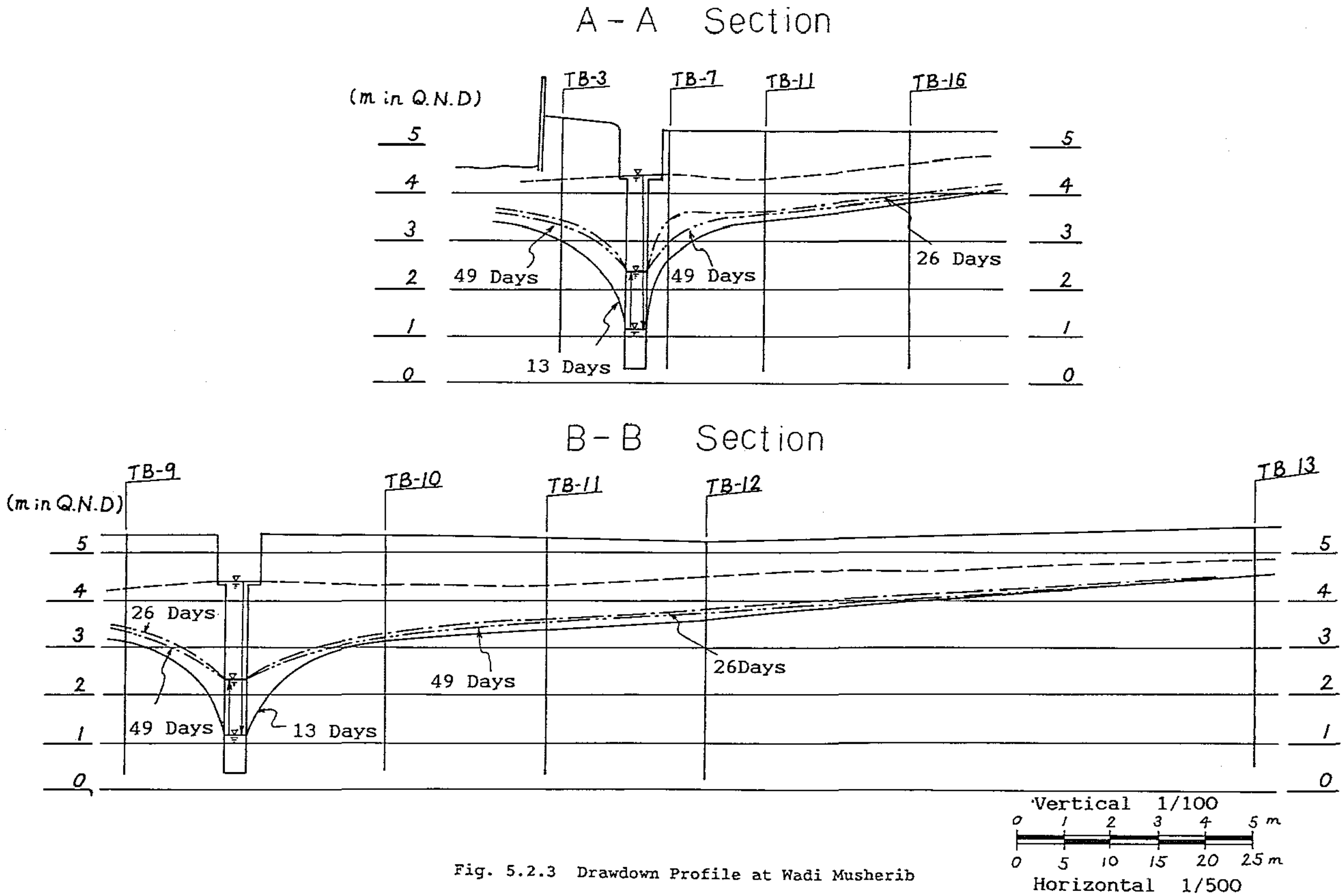


Fig. 5.2.3 Drawdown Profile at Wadi Musherib

5.2.2 Recovery Test

(1) Test period

The recovery test was planned for 1 (one) week period, but recovery rates observed at Wadi Musherib were relatively slow, so the test was then extended until the recovery rates became less than 1 cm per day. Finally the test period was from October 27th to November 9th 1986.

(2) Total recovery height

Total recovery height at the Trench was 3.40 m and those of the open holes became smaller according to the distance from the Trench, as shown in Fig. 5.2.4.

Rough figures between the recovery height and the distance from the Trench are:

Distance from Trench	Recovery Height
5 m	2.0 - 2.5 m
10 m	1.3 - 1.8 m
15 m	1.1 - 1.2 m
30 m	0.7 - 1.0 m

(3) Recovery percentage in Time

Recovery percentages at the Trench after 47 hrs, (approx. 2 days) and 119 hrs. (approx. 5 days) were 67 and 87 percent respectively. Corresponding recovery percentages of the open holes were smaller than that of the Trench.

Especially, recovery percentage after 47 hrs at TB-10 was too small and the surrounding area of TB-10 may have low permeability.

5.2.3 Water Quality

(1) Sampling

The quality of the groundwater drained to the Trench is an important factor for determining the utilisation of groundwater, water treatment and dumping destination. Water sampling was, therefore, carried out regularly from the Trench and observation wells at the four corners. The water samples were then analysed with the cooperation of the Doha South Sewage Treatment Works Laboratory.

The items for analysis were as follows.

- pH
- Electric Conductivity (EC)
- Chemical Oxygen Demand (COD)
- Chlorine Ion
- Sulphate Ion
- Calcium Ion (As $CaCO_3$)
- Magnesium Ion (as $CaCO_3$)
- Sodium Ion
- Potassium Ion

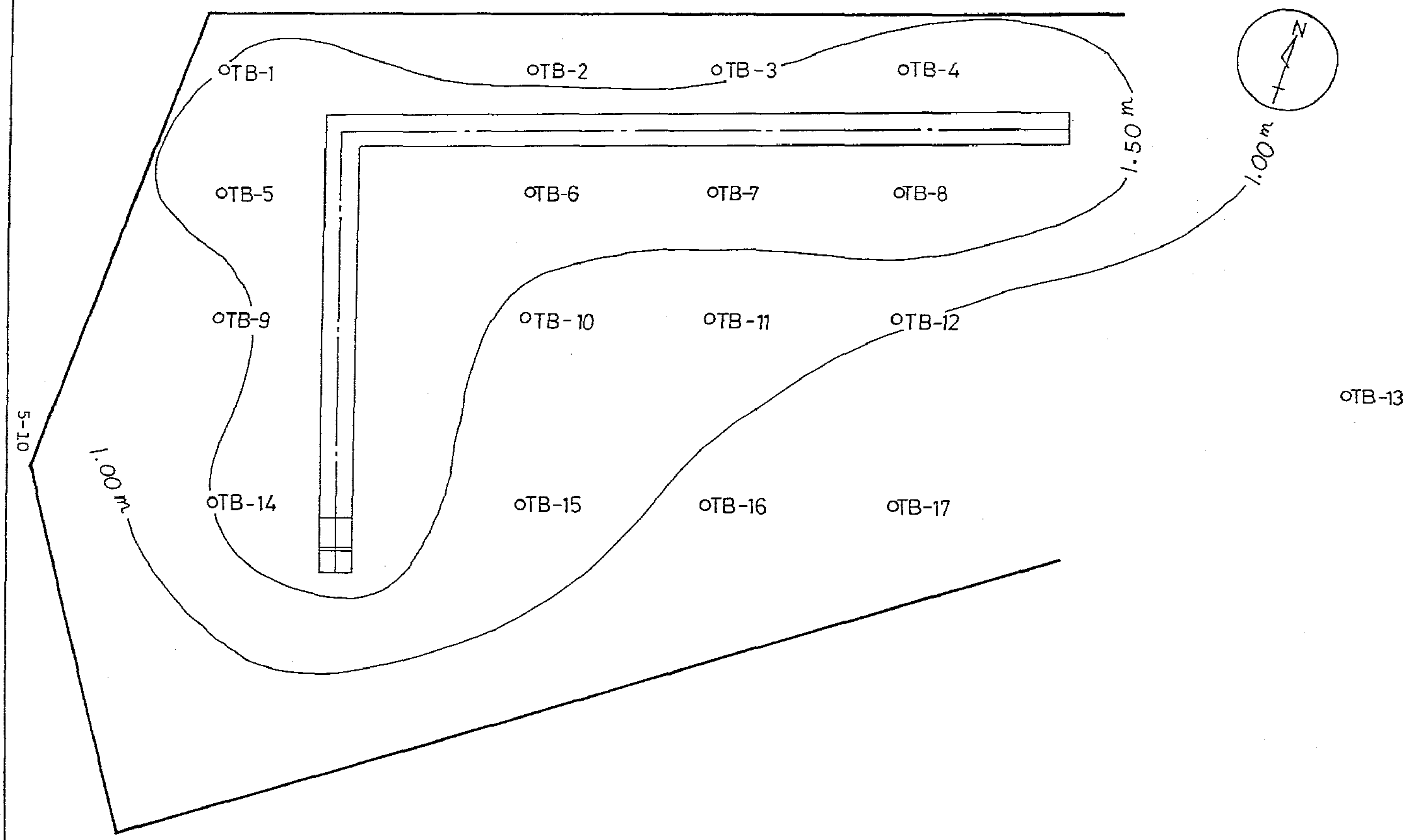


Fig. 5.2.4 Recovery Height Distribution around the Trench of Wadi Musherib

(2) Analysis Results

The results of the analysis are shown in Fig. 5.2.5 and the characteristics of each water sample are summarised as follows.

1) Quality of Trench Water

The quality of the Trench water was both the best and the most stable. While there was some small fluctuation in the quality during the first few days following the commencement of the test pumping, the quality later stabilized.

Some values in both initial and final stages of the test were

o	EC	:	4000 to 5000 micro mhos/cm
o	COD	:	1 to 3 mg/l
o	Ca	:	900 to 1200 mg/l
o	Na	:	200 to 500 mg/l
o	SO_4	:	1200 to 1400 mg/l
o	Cl	:	600 to 750 mg/l

2) Quality of TB-1 Water (NW corner)

There was little fluctuation in the quality of the TB-1 water in regard to the pH and EC values but the COD value showed specific fluctuation. The COD value before July 16th was 0.7 - 2.7 mg/l, reaching 7.3 mg/l on July 23rd and then gradually dropping with a final reading of 3.1 mg/l on October 27th.

3) Quality of TB-4 Water (NE corner)

There was a big change in the quality of the TB-4 water before and after July 16th. Before July 16th, the EC and COD values were as high as 6,500 - 8,370 micro mhos/cm and 5.2 - 8.8 mg/l respectively. After that date, however, the EC value dropped to 4,300 - 4,970 micro mhos/cm and the COD value gradually dropped reaching 1.1 mg/l on August 10th, then began rising evenly to 6.7 mg/l on October 19th, finishing at 2.4 mg/l at the end of pumping test. Cl values up to July 16th ranged between 1300 to 900 mg/l while after that value was relatively steady at 590 mg/l, finishing at 630 mg/l at the end of test.

4) Quality of TB-14 Water (SW corner)

The EC value of the TB-14 water was 12,040 - 13,050 micro mhos/cm before July 16th but dropped to 6,760 - 8,510 micro mhos/cm after July 23rd. The COD value increased from 1.1 mg/l to 7.3 mg/l in the period from the commencement of the sampling test to July 16th, dropping to 2.4 - 4.6 mg/l after July 23rd. Cl value initially was 1750 mg/l on July 2nd, peaking to 2450 mg/l on August 31st, before falling to 743 mg/l on October 27th.

5) Quality of TB-17 Water (SE corner)

The quality of the TB-17 water was worse than that of the other water samples and showed much fluctuation. EC value had a rather steady average rate of 16,700 micro mhos/cm up to July 9th, rising sharply to 49,040 micro mhos/cm on July 16th, falling rapidly to 5,040 micro mhos/cm on October 27th. COD value initially 10 mg/l on July 2nd peaked to 72.7 mg/l on July 16th, falling to 1.8 mg/l at the close of the test. Cl also showed fluctuating values of 4,050, 17,000 and 458 mg/l on July 2nd, July 23rd and October 27th.

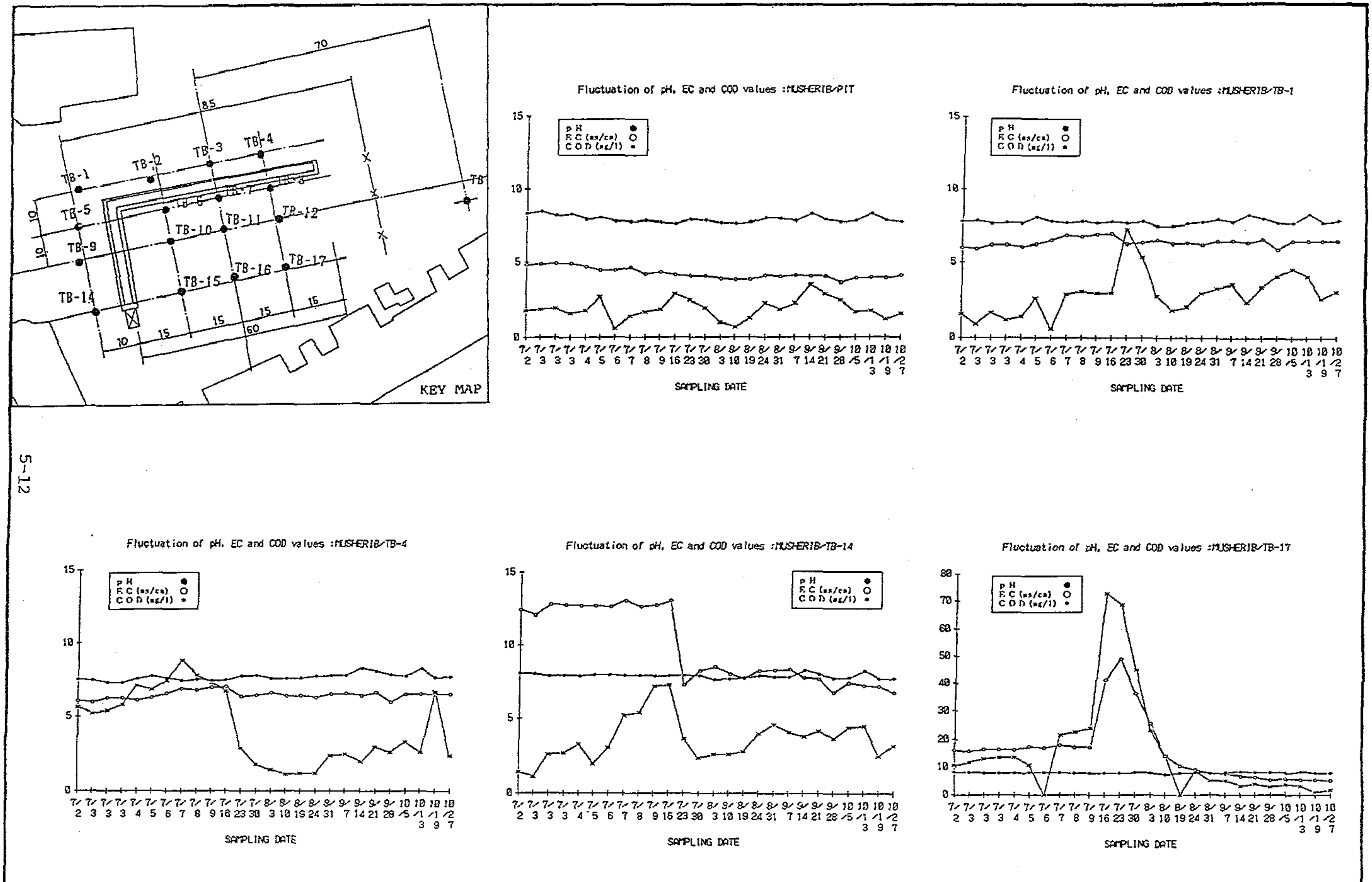


Fig. 5.2.5 Water Quality Fluctuation at Wadi Musherib

(3) Water Quality Assessment

EC values for all observation wells were measured both in the initial and final stages, and the values of each well respectively changed slightly in both stages, however, the distribution of EC values in the area of the test work site showed the same tendency, No. 10 was the highest at both times (EC 18,220 and 20,200 micro mhos/cm), the next highest was No. 6 (EC 13,640 and 16,670 micro mhos/cm). The Trench water showed the lowest values regarding all items throughover the test period.

After a period of showing some water quality fluctuation for both Trench and the four observation wells where water samples were taken for chemical analysis, almost all water quality data for Trench and observation wells water except TB-17 became stable and tended to show each characteristic level. In the case of TB-17 well, polluted water entered it from a building located on the southern side, showing very high values of EC, COD and Cl in the mid July, these values sharply decreased by August 10th.

5.3 Pumping Test at Old Rayyan

5.3.1 Continuous Pumping Test

(1) Activity

Due to the delay of transportation to remove the abstracted groundwater and of decision for its disposal, the pumping test had many difficulties to start and the following three plans were made.

1 Preliminary test

Prior to the main pumping test scheduled to start in August, a preliminary test was carried out from 11th to 18th July. This allowed testing of the apparatus and pumps and enabled investigation of the hydrogeological conditions of the test site.

2 Trial run with 1 shift transportation

Test pumping began on August 5th, 1986 and was continued until August 20th. The test was carried out with only 4 hours discharging per day, between 7 am and 11 am. Since the volume of water abstracted was so small, the water level in the Trench dropped by only about 1.2 m per day and almost recovered to its original level within twelve hours. As only a small volume of water was abstracted and discharging intermittent, it has been assumed that the groundwater level did not reach a steady state.

3 Continuous run with 2 shifts transportation

Owing to the kind help provided by the Ministry of Public Works (MPW), a longer pumping test began on August 30th, 1986 with 14 hours discharging per day, and consequently it became possible to obtain more reliable results. This test was continued until October 30th.

(2) Groundwater Inflow to the Trench

In the Trench many groundwater discharge points were observed. Two discharge mechanims could be distinguished, one along the bedding planes and another along the fissures. Large amount of discharge was observed along the fissures, especially in the shorter leg on the Trench (30 m). In case of discharge along bedding plane it was obvious that the discharging rate increased toward the fissures.

(3) Discharge Amount

The relationship between the water level at the Trench and the discharge amount can be summarized as follows:

1 Pumping was 14 hours daily (Friday holiday) and relationship is examined between Trench water QND level at 06:00 hr on Thursday and weekly discharge as shown in Fig. 5.3.1. From second to fourth week there was a drop in Trench water level of around 50 cm with a corresponding fall in weekly discharge amount. From the fifth to eighth week the water level fluctuated within 10 cm, while discharge amount showed an increase from 5,900 to 6,300 m^3/week dropping to 6,200 m^3/week on the eighth week.

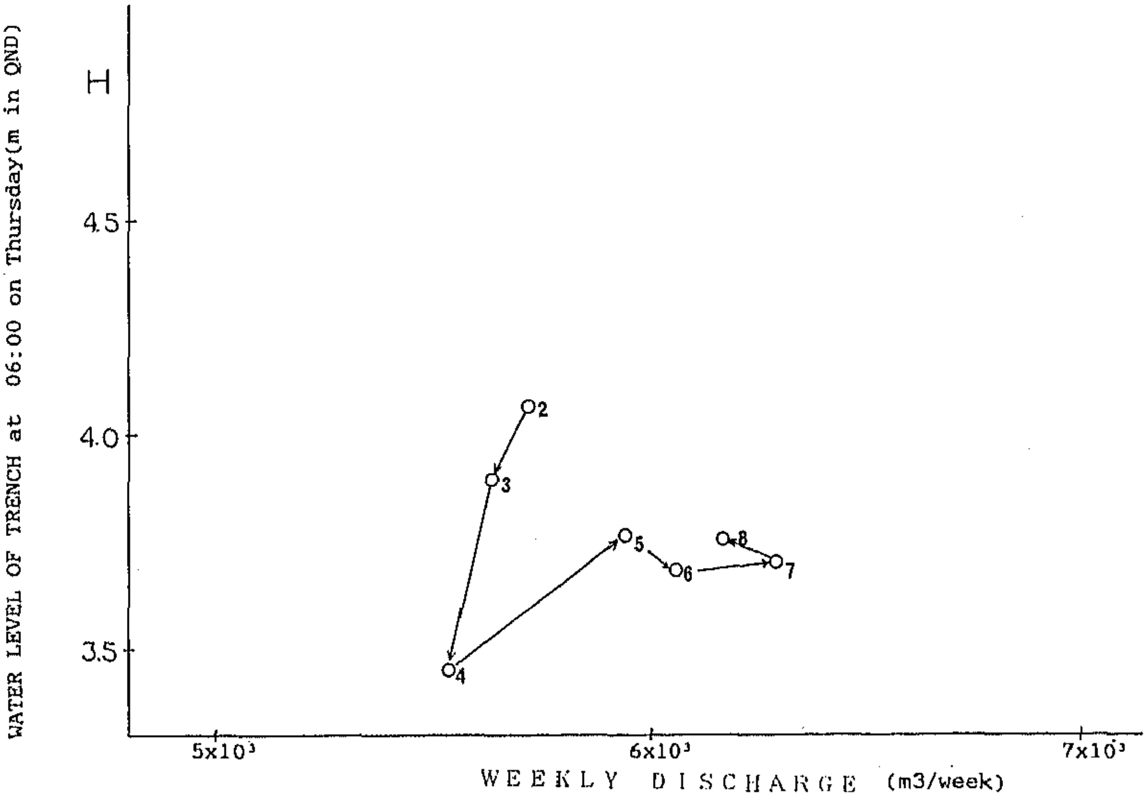


Fig. 5.3.1 Weekly Discharge Amount with 2 Shifts Transportation at Rayyan

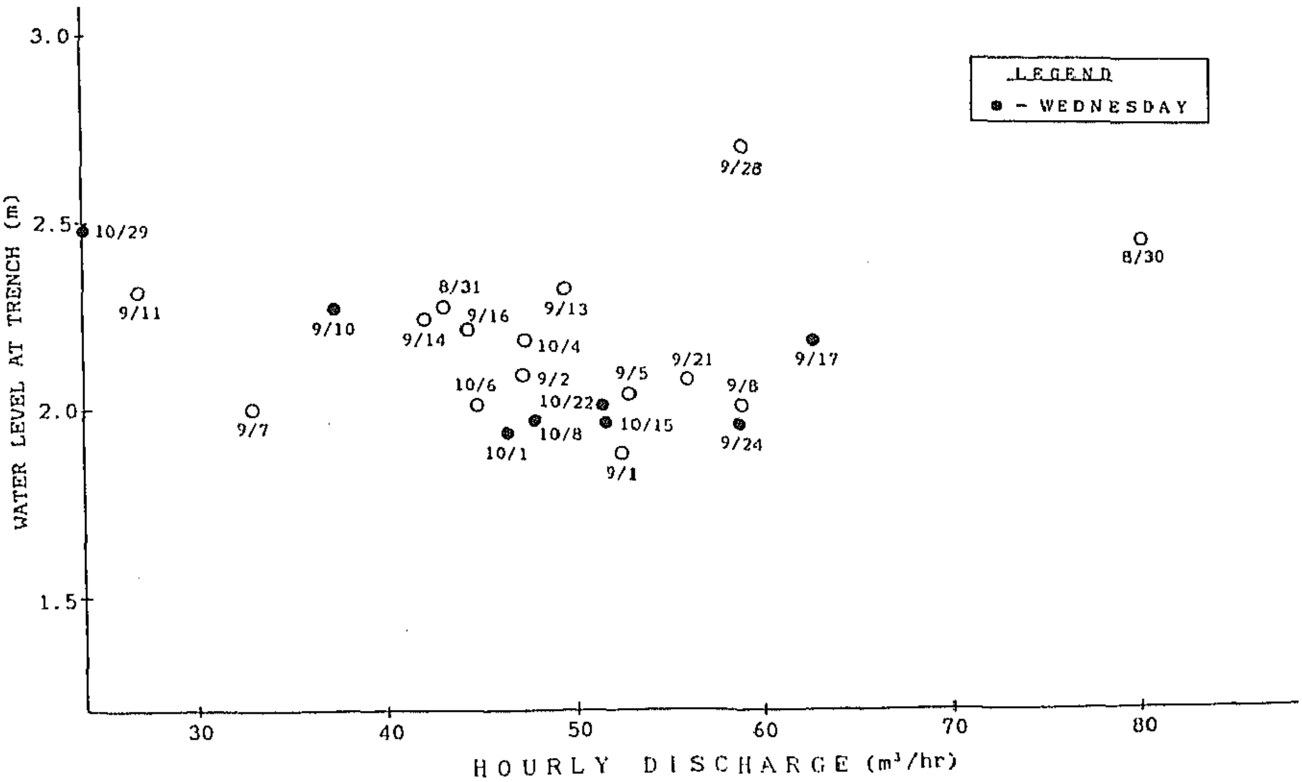


Fig. 5.3.2 Hourly Discharge Amount with Bottom Water Level of Trench at Rayyan

2 Fig. 5.3.2 shows relationship between hourly discharge and Trench water levels. During the period from 13th September up to the last few days in October hourly discharges ranged from 45 to 60 m^3/hr (0.75 - 1.0 m^3/min) with bottom water levels of Trench measured at 20:00 hrs ranging between 2.0 to 2.2 m in QND.

(4) Drawdown Depth

The outline of pumping test condition was as follows;

In July a preliminary pumping test was executed with the following results.

(a) After 18 hours from the beginning of the pumping test, the water level in the Trench reached the bottom i.e. 2.9 m drawdown.

(b) The groundwater levels at the boreholes reached a steady state at about 18 to 24 hours. The drawdown from initial groundwater level was approximately 1.0 metres in the vicinity of the Trench, (i.e. within 30 m of the Trench).

With two shifts operating daily for a total of 14 hours, the water inside the Trench, recovered in the night, can be drained in 7 - 8 hours and drainage at the Trench bottom level can be carried on for a further 6 - 7 hours. At distances of 30 and 90 m from trench, groundwater level was lowered by 0.5 - 0.7 m and 0.1 - 0.2 m respectively.

The drawdown depth distribution around the Trench and its profile are respectively shown in Fig. 5.3.3 and Fig. 5.3.4.

5.3.2 Recovery Test

The recovery test was carried out from October 30th to November 6th.

Observation results at open holes and the Trench are summerized as follows.

(1) Total Recovery Height

Total recovery height at the Trench was 2.48 m and those of the open holes became smaller according to the distance from the Trench as shown in Fig. 5.3.5. Rough figures of the recovery heights according to distance from the Trench are:

Distance from Trench	Recovery Height
10 m	0.9 - 1.7 m
30 m	0.5 - 0.8 m
90 m	0.2 - 0.5 m

Recovery height contour lines show that drawdown by abstraction was developed in the eastern side of trench rather than in the western side.

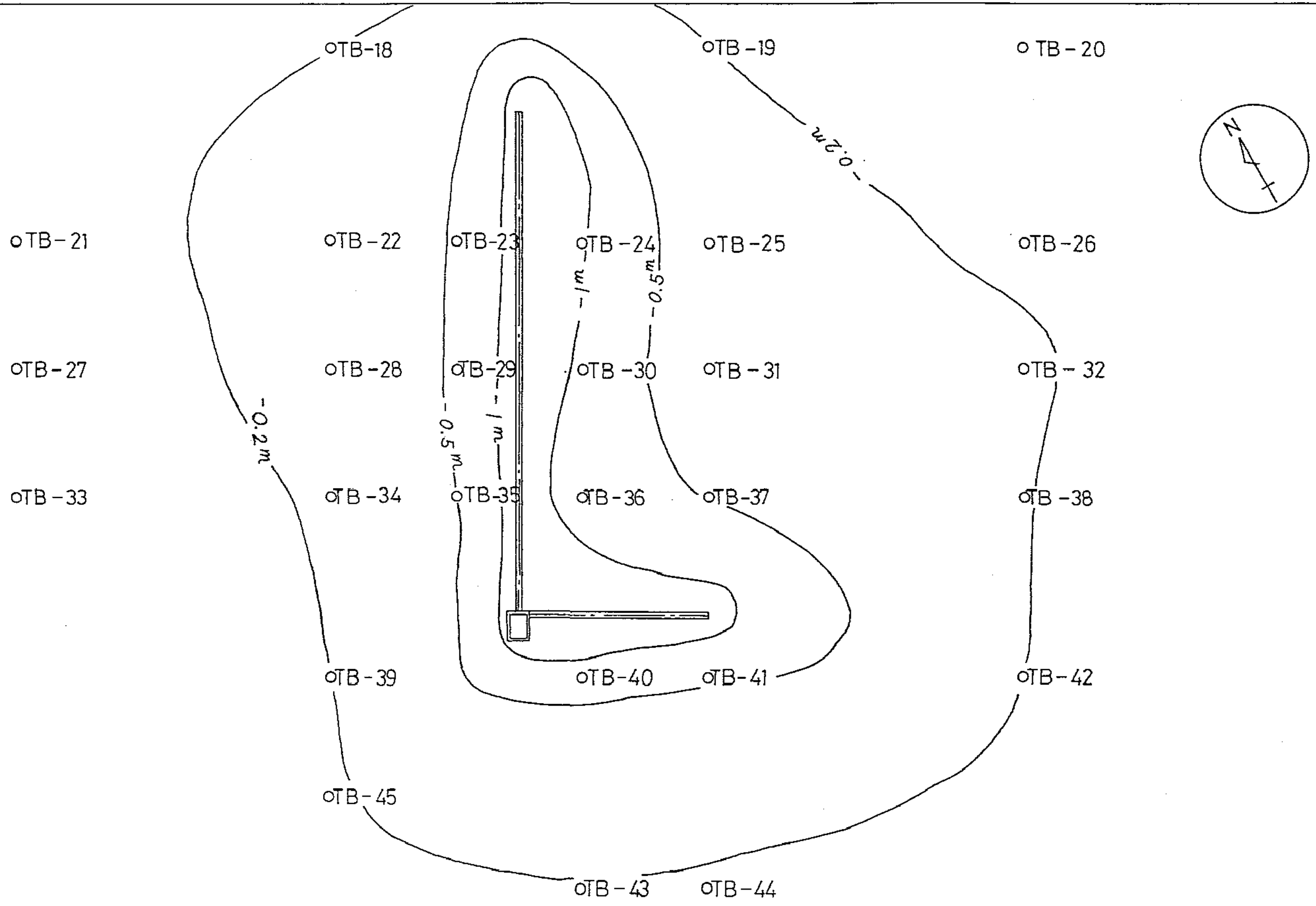


Fig. 5.3.3 Drawdown Depth Distribution around the Trench of Rayyan

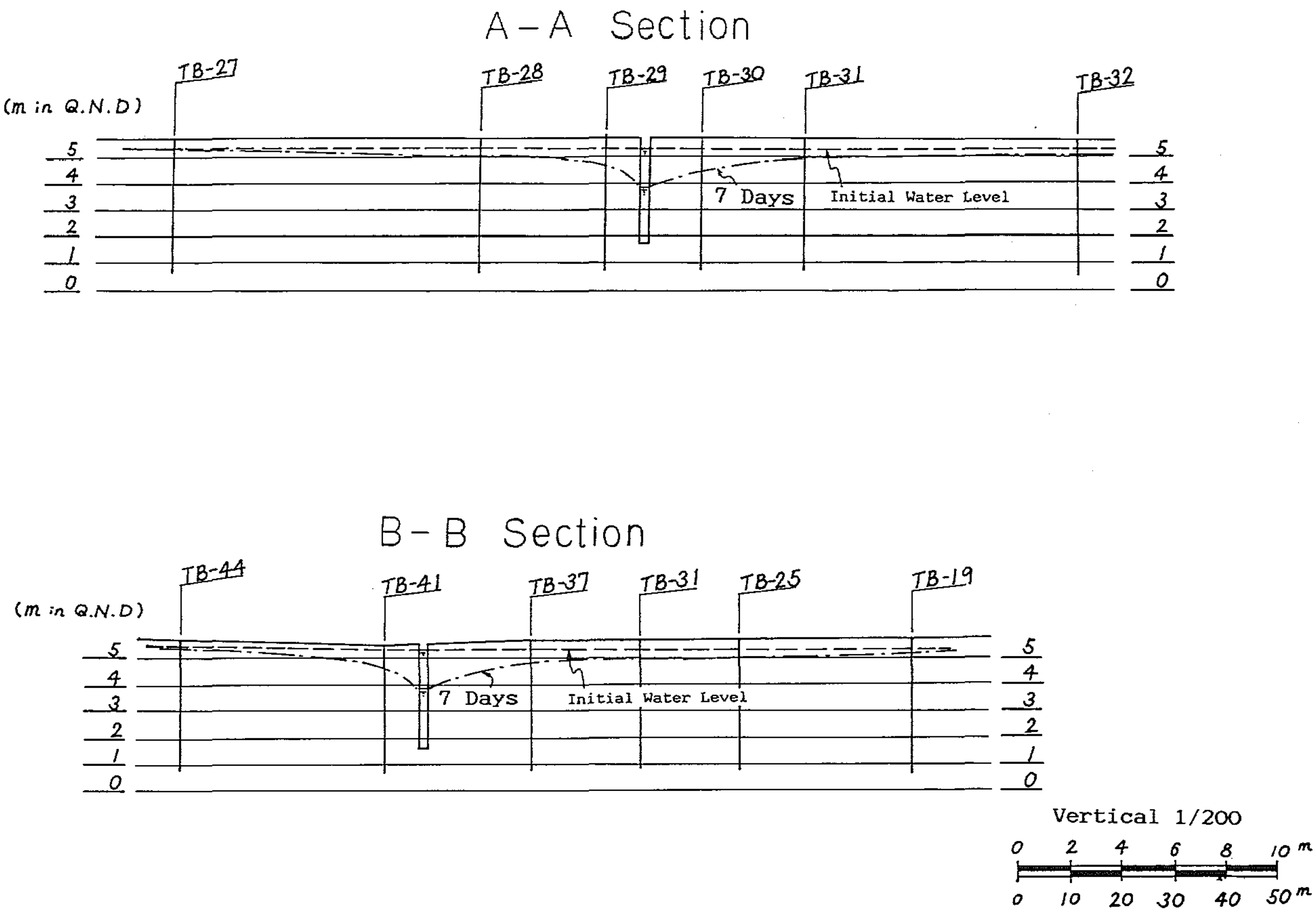


Fig. 5.3.4 Drawdown Profile at Rayyan

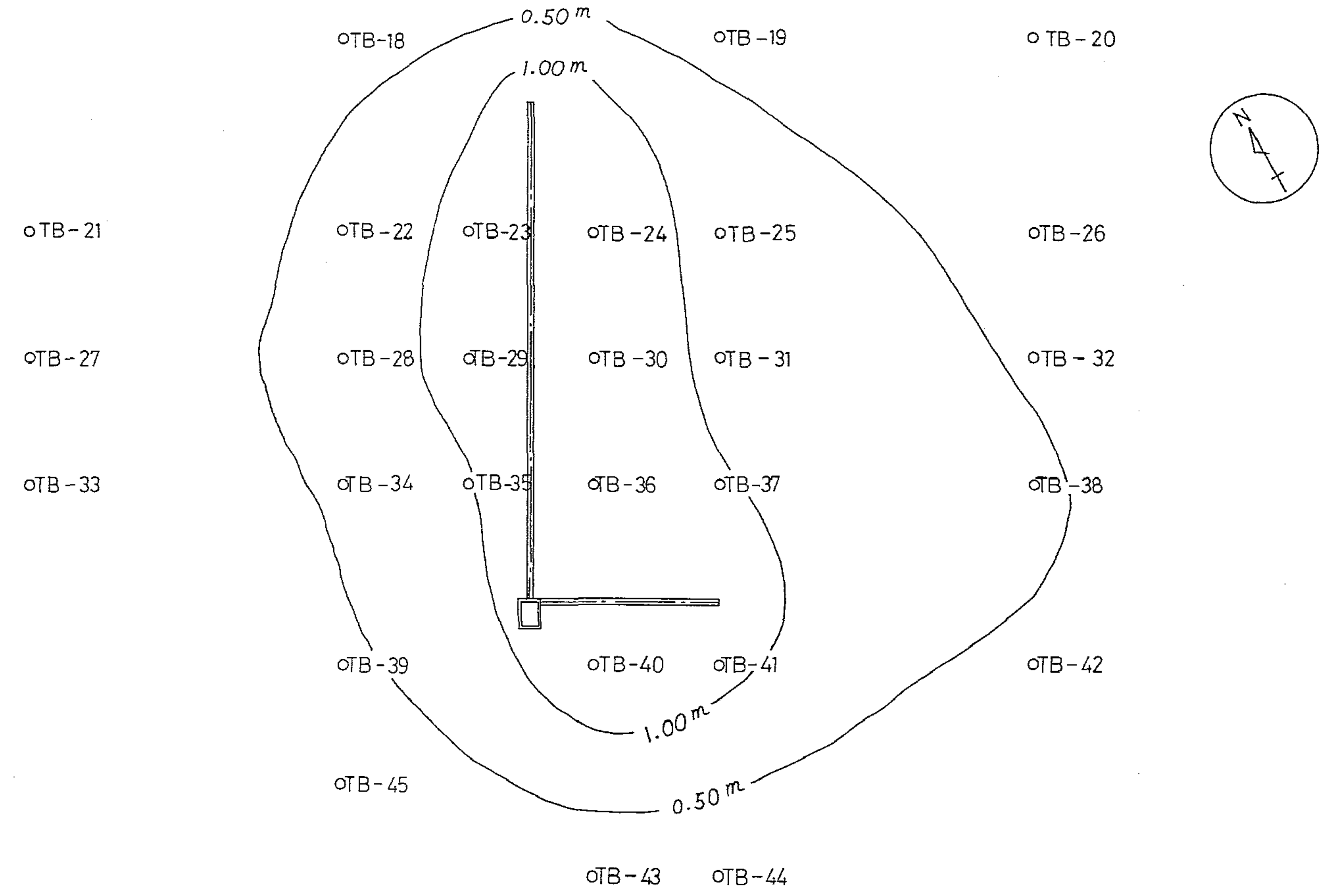


Fig. 5.3.5 Recovery Height Distribution around the Trench of Rayyan

(2) Recovery Percentage in Time

Recovery percentages at the Trench after 25 hrs. (approx. 1 day) and 49 hrs. (approx. 2 days) were 85 and 93 percent respectively. The recovery percentage of an open hole is generally small when its location is removed from the Trench.

5.3.3 Water Quality

(1) Sampling

As the means of transporting the pumped-up water from the Trench in Rayyan took some time before being firmly established, the water sampling schedule was affected, as in the case of the pumping tests. As table 5.3.1 shows, the schedule accordingly became very irregular.

Table 5.3.1 Sampling at Rayyan Test Work Site

Period	Site	Type of Pumping Test
7/11 - 7/18	Pit, TB-23, TB-39	Pumping test without transportation
8/5 - 10/30	Pit, TB-21, TB-25 TB-42, TB-45	Pumping test with transportation

The pumping test in July was carried out by dumping the water pumped from the Trench to the east side of the Trench. Although the volume of pumped-up water was sufffficient for sampling, this method was not preferable due to the possibility of the dumped water being recirculated.

Transportation by a road tanker commenced in August but the pumped-up volume was low for nearly a month because of there being only one shift. Full-scale pumping tests with two shifts started on August 30th.

The items for the water quality analysis were the same as those for Musherib, as follows.

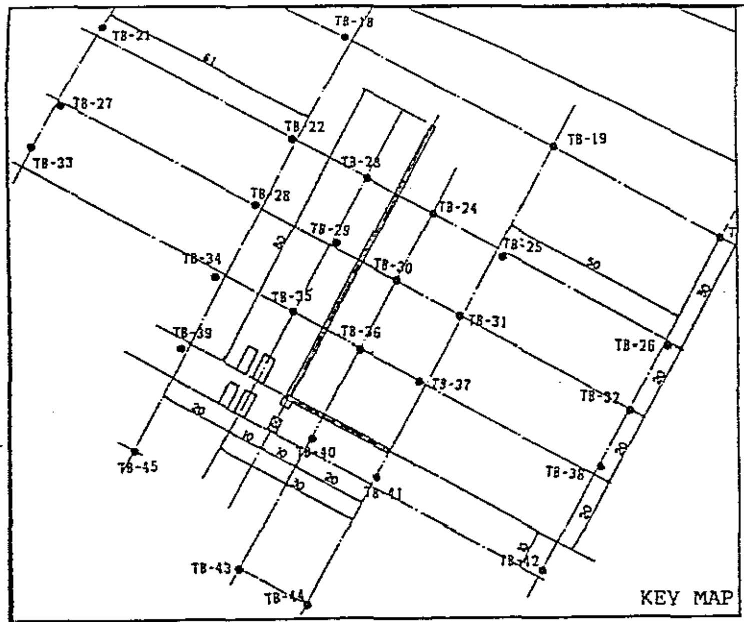
- o pH, EC, COD
- o Cl^-, SO_4^{--}
- o Ca^{++}, Mg^{++}, K^+, Na^+

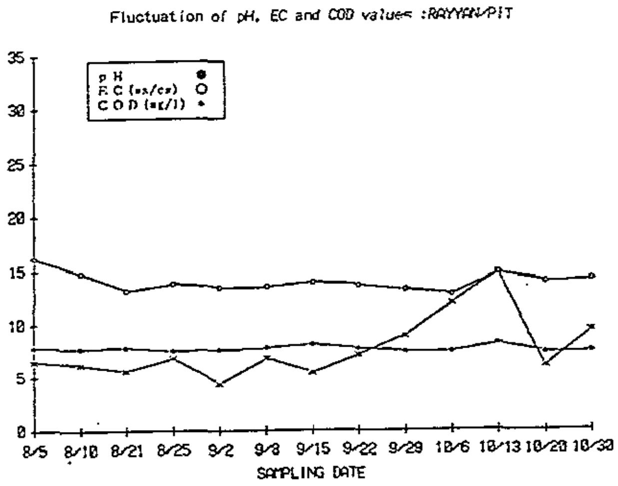
(2) Analysis Results

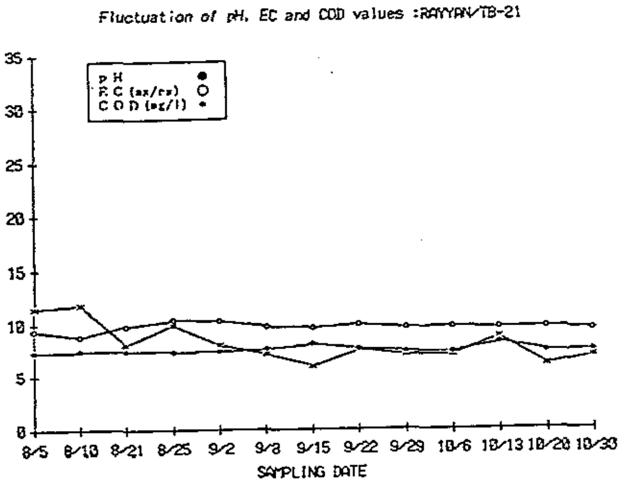
The results of the analysis are shown in Fig. 5.3.6.

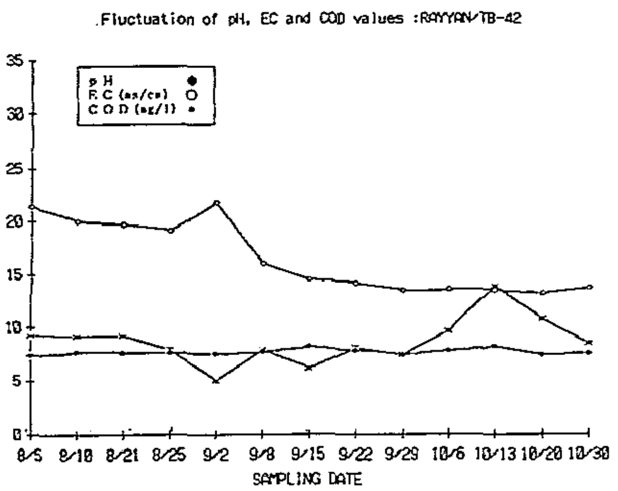
1) Quality of Trench Water

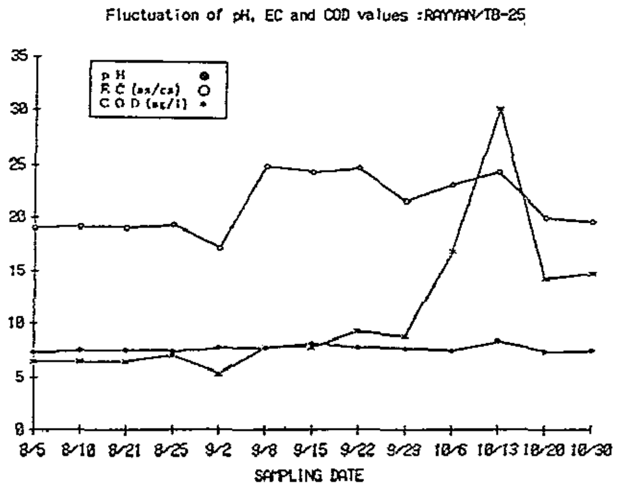
The EC value fluctuated between 13,150 and 17,810 micro mhos/cm in the tests between July 11th and October 30th. The COD value fluctuated between 3.9 and 14.8 mg/l, hitting its highest point on October 14th becoming lower to 9.4 mg/l at the end of test.











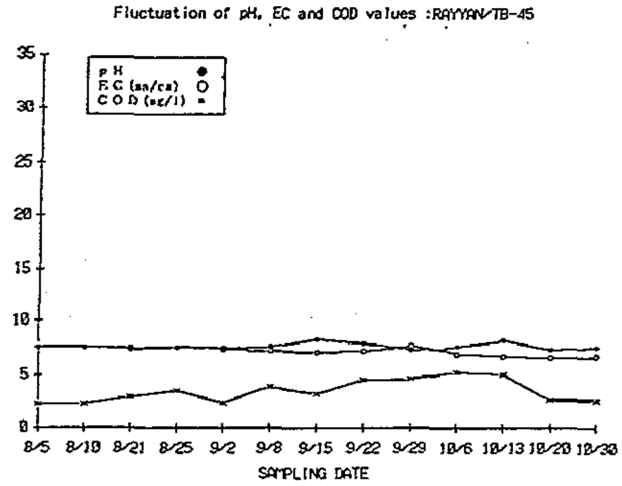


Fig. 5.3.6 Water Quality Fluctuation at Rayyan

2) Quality of TB-21 (NW corner)

Initial and final EC values were 9,400 micro mhos/cm, slightly lower than the average value of 9,700 micro mhos/cm. COD values ranged between 11.7 to 6.0 mg/l. Initial and final values for Cl, SO_4 and Ca showed very slight change.

3) Quality of TB-25 (NE corner)

Initial and final EC values were 18,960 and 19,440 micro mhos/cm. During the period from September 8th to October 13th values averaged 23,800 micro mhos/cm with a peak value of 24,840 micro mhos/cm on September 8th, which was the highest EC value recorded at Rayyan. COD values fluctuated between initial and final values of 6.4 and 14.7 mg/l respectively with a peak reading of 30 mg/l on October 13th. Initial and final values for Cl, SO_4, Ca, and Mg showed little differences.

4) Quality of TB-42 (SE corner)

The highest EC value of 21,340 micro mhos/cm was on August 8th which gradually fell to its lowest value of 13,120 micro mhos/cm on October 20th. COD values fluctuated between 5.0 to 13.8 mg/l. Cl value averaged 7,000 mg/l during the month of August, and 4,600 mg/l during September and October with little variations.

5) Quality of TB-45 (SW corner)

EC values did not vary much ranging from 7,690 to 6,640 micro mhos/cm. Initial and final values of COD were 2.2 and 2.5 mg/l respectively, with a peak of 5.3 mg/l on October 6th. Final values of Cl, SO_4 and Ca showed small decrease from those measured at the start of the pumping test.

(3) Water Quality Assessment

After the start of the continuous pumping up of groundwater from the Trench, the EC of Trench water and those of observation wells TB-21 and TB-45 (west of the trench) seemed stable, in comparison to those for observation wells TB-25 and TB-42, both of which are situated east of the trench.

In the case of Rayyan, the values for Ca and SO_4 seemed more stable than the values for Na-Cl, maybe by the reason of higher solubility of Na-Cl or lower solubility (and saturation) of Ca-SO_4. The water quality of wells in the east side of the Trench seemed to be showing tendency of almost always higher salinity with EC than those on the western or west-south side of the Trench.

The water quality of Trench seemed almost the average of the four observation wells as shown in Table 5.3.2. Figures in the table are the averages of 23 samples tested during August 5th to October 27th.

Table 5.3.2 Water Quality from Test Trench and Four Observation Wells on Rayyan Test Work Site

Analytical Items		Test Trench	Average values of four obs. wells
EC (micro mhos/cm)		13,985	13,657
Cl	(mg/l)	4,261	4,802
SO_4	"	2,659	2,621
Ca	"	2,382	2,477
Mg	"	1,517	1,486
K	"	98	91
Na	"	1,762	1,582
COD	"	7.8	7.8

This fact suggests that permeability of the underground conditions is nearly homogenous thus facilitating the mixing to shallow surrounding groundwater in various directions.

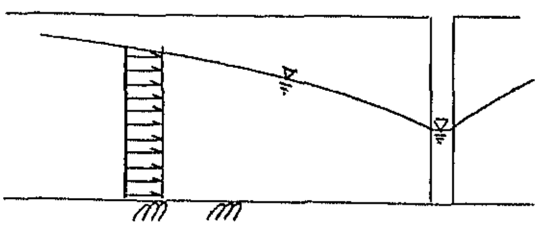
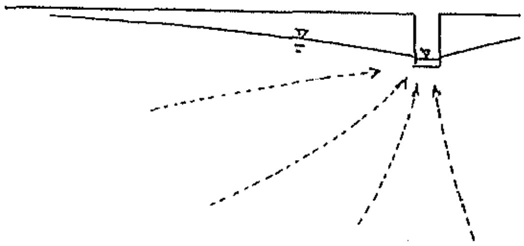
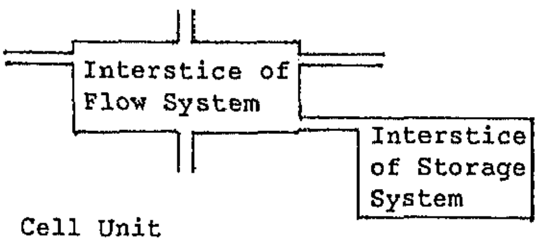
5.4 Numerical Analysis for Drawdown Simulation

5.4.1 Mathematical Models

The purpose of the drawdown simulation is to determine the hydraulic constants and to evaluate the drawdown effects of drainage system.

The principal simulation models are shown in Table 5.4.1. In the case where the trench length is much longer than its width, the vertical plane which is at the right angle with the length direction is usually used as the calculation domain.

Table 5.4.1 Mathematical Methods for Drawdown Simulation

Model	Required Dimension(s) and Calculation Method
Horizontal Model  Base Rock	o One Dimensional Analysis o Finite Difference Method
Vertical Plane Model 	o Two Dimensional Analysis o Finite Element Method
Rock Infiltration Model  Cell Unit	o Two Dimensional Analysis o Finite Difference Method

The following facts should be considered in the selection of a mathematical model in view of the pumping test results.

a. As the trench partially penetrates into the aquifer, the vertical flow element is dominant near the trench bottom.

b. If fissures only exist in the shallow part of the aquifer, the discharge should quickly diminish. However, the actual discharge stabilised a few days after water level of the trench reached the bottom. This fact indicates that the fissures have developed not only in the horizontal direction but also in the vertical direction.

The horizontal model is not adequate due to its strong deviation from the assumed uniform horizontal flow. In comparison, the vertical plane model is widely used in the civil engineering field and can express vertical flow element. The rock infiltration model is occasionally referred to in the research papers but its application examples are very poor.

Generally speaking, the aquifer of limestone has strong heterogeneity as porous media and its permeability should be treated in the stochastic phenomena. This kind of approach demands much information and time for analysis.

The model selected here aims to seek the mean value of permeability corresponding to the stochastic distribution of permeability in the heterogenous field.

In the present study, the two dimensional vertical plane model is adopted. The governing equation is expressed as follows.

$$\frac{\partial \rho}{\partial t} = \mathrm{div}\ (k\ \mathrm{grad}\ \psi)$$

Where, ρ : Mass Volume in Porous Media
t : Time
K : Permeability Coefficient
ψ : Hydraulic Head
div : Divergence
grad: Gradient

The finite element method is used for the present purpose of the study. The initial conditions and the boundary conditions are set in accordance with the purpose of the analysis.

5.4.2 Simulation of Drawdown

(1) Calculation Domain

The calculation domain includes the following three layers which are strictly related with the pumping test by trench abstraction method.

1 First Layer : Strong Weathered Zone of Upper Dammam Formation
2 Second Layer: Weak Weathered Zone of Upper Dammam Formation
3 Third Layer : Lower Dammam Formation

The drawdown effect at a point of 100 m distance from trench is estimated to be negligible and this distance is sufficient to set up the boundary condition.

The calculation domains of two dimensional vertical plane model for drawdown simulation for Wadi Musherib and Rayyan are expressed by the grid systems as shown in Fig. 5.4.1 and Fig. 5.4.2.

(2) Calculation Procedure

- The tentative permeability coefficient of each layer was evaluated from the mean value of Lugeon value obtained from the Lugeon test.

- The judgment condition is the concordance of drawdown depth curve in function of the distance from trench.

- The final solution was sought by the trial and error method.

- This calculation procedure is shown in Fig. 5.4.3.

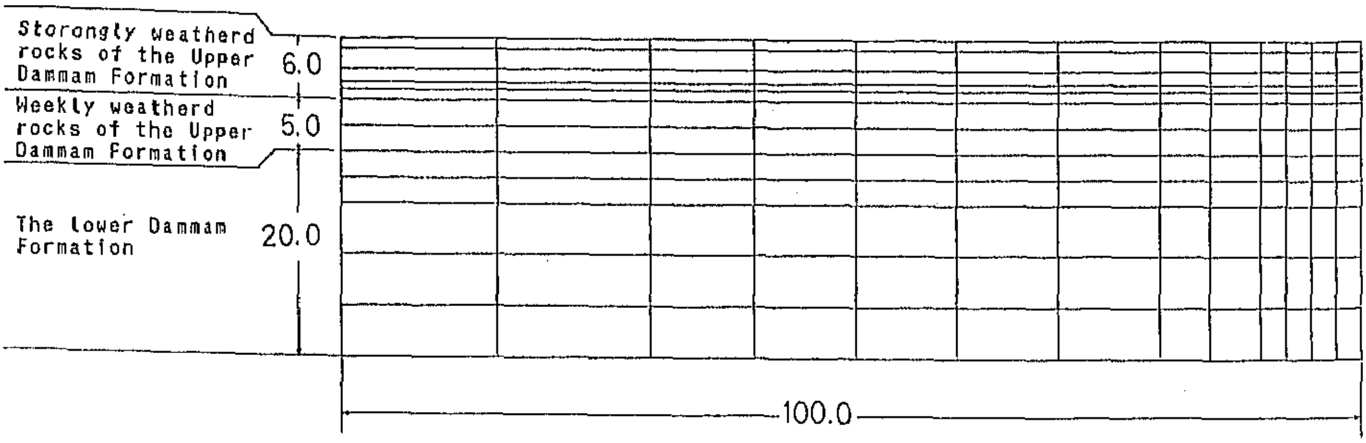


Fig. 5.4.1 Grid System of Drawdown Simulation for Wadi Musherib

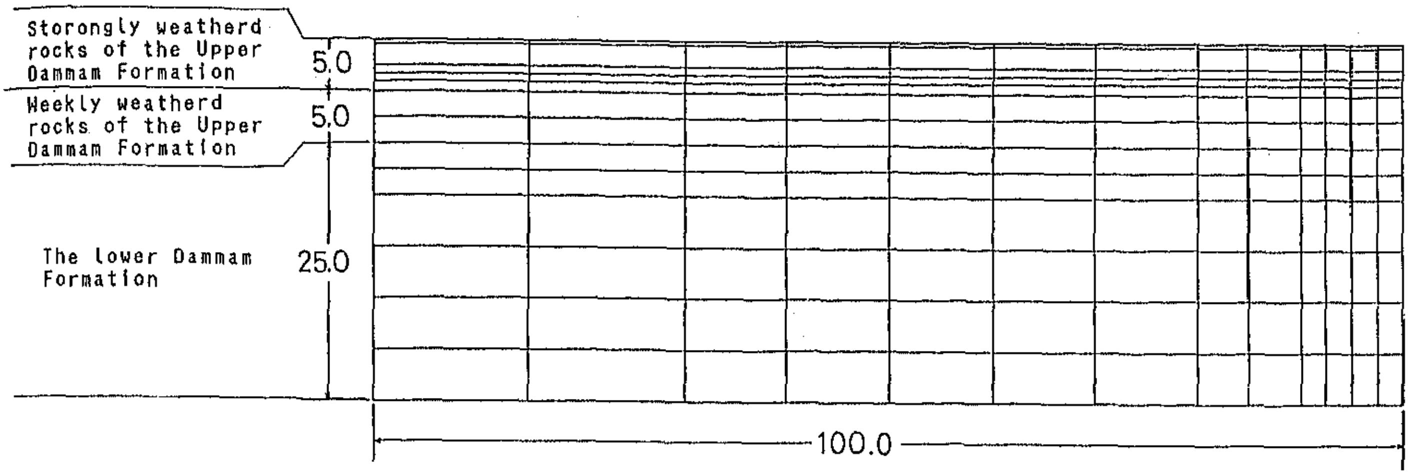


Fig. 5.4.2 Grid System of Drawdown Simulation for Rayyan

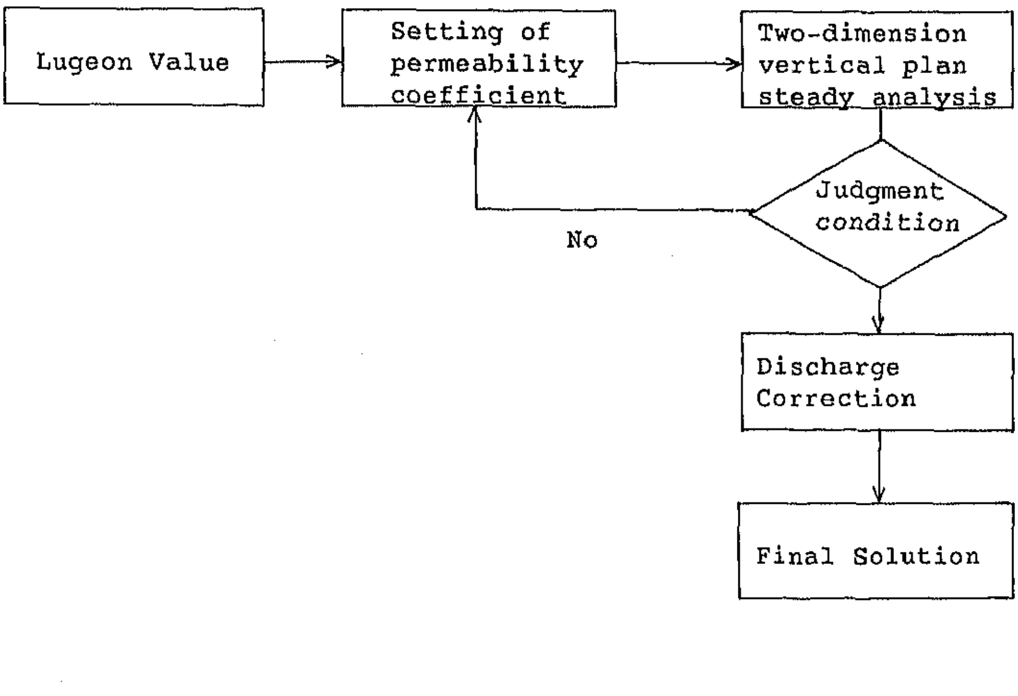


Fig. 5.4.3 Identification Procedure of Permeability Coefficient

(4) Calculation Results for Rayyan

Eight runs were carried out, as shown in Table 5.4.3.

Table 5.4.3 List of Test Runs for Premeability Coefficient Identification: Rayyan

	Permeability Coefficient (m/sec)		
	1st Layer	2nd Layer	3rd Layer
Case 1	2.62×10^{-4}	2.44×10^{-4}	8.11×10^{-5}
Case 2	5.28×10^{-4}	4.93×10^{-4}	1.64×10^{-4}
Case 3	3.95×10^{-4}	3.69×10^{-4}	1.23×10^{-4}
Case 4	9.17×10^{-5}	8.54×10^{-5}	2.84×10^{-5}
Case 5	9.14×10^{-5}	8.54×10^{-5}	4.00×10^{-6}
Case 6	9.14×10^{-5}	8.54×10^{-4}	8.50×10^{-6}
Case 7	9.17×10^{-5}	8.54×10^{-5}	3.60×10^{-5}
Case 8	8.36×10^{-5}	7.79×10^{-5}	3.28×10^{-5}

The permeability coefficient of each layer obtained from final solution is as follows,

- First layer (Strong weathered layer): 8.36×10^{-5}m/s
- Second layer (Weak weathered layer) : 7.79×10^{-5}m/s
- Third layer (Lower dammam) : 3.28×10^{-5}m/s

The drawdown result of the final solution is as follows;

Distance from Trench	Drawdown Depth
10 m	1.56 m
30 m	0.75 m
70 m	0.17 m

(3) Calculation Results for Wadi Musherib

During the delay of the core boring, the fundamental nature of the vertical plane model with approximately 20 runs was studied; Different kinds of boundary condition, setting of calculation domain, etc.

After having obtained the Lugeon values and core logs, eight runs were carried out, as shown in Table 5.4.2.

Table 5.4.2 List of Test Runs for Premeability Coefficient Identification: Wadi Musherib

	Permeability Coefficient (m/sec)		
	1st Layer	2nd Layer	3rd Layer
Case 1	4.52×10^{-4}	1.31×10^{-4}	5.99×10^{-5}
Case 2	4.04×10^{-4}	1.18×10^{-4}	5.36×10^{-4}
Case 3	2.25×10^{-4}	6.54×10^{-4}	2.98×10^{-4}
Case 4	4.17×10^{-5}	1.20×10^{-5}	5.51×10^{-5}
Case 5	4.17×10^{-5}	3.00×10^{-5}	1.90×10^{-6}
Case 6	4.17×10^{-5}	6.00×10^{-4}	3.10×10^{-6}
Case 7	4.17×10^{-5}	2.00×10^{-5}	1.50×10^{-5}
Case 8	2.71×10^{-5}	1.30×10^{-5}	9.74×10^{-6}

The permeability coefficient of each layer obtained from final solution is as follows,

- First layer (Strong weathered layer): 2.71×10^{-5}m/s
- Second layer (Weak weathered layer) : 1.30×10^{-5}m/s
- Third layer (Lower dammam) : 9.74×10^{-6}m/s

The drawdown result of the final solution is as follows;

Distance from Trench	Drawdown Depth
5 m	2.34 m
10 m	1.98 m
30 m	1.12 m

6. BASIC CONCEPT FOR URGENT DRAINAGE PLAN

6. BASIC CONCEPT FOR URGENT DRAINAGE PLAN

While the control of the groundwater recharge sources is the most appropriate measure to fundamentally solve the rising groundwater problem, a long period of time is required in order for it to be successfully implemented. Unfortunately, however, the appearance of standing water in lowland areas and salt accumulation in capillary wet areas do not wait for preventive measures to be implemented and, therefore, the introduction of urgent measures is necessary to prevent damage originating from these phenomena.

The introduction of urgent drainage measures in Wadi Musherib and Montazah in Doha has been recommended since the time of the ASCO Report (1983). The basic requirements for these urgent measures are described in Item 4.4 and the various conditions to be considered in the planning of effective drainage facilities and their conformity are examined here.

6.1 Selection of Project Areas

The following areas which have a shallow groundwater level and where direct damage is currently in progress have been selected from the Study Area of the JICA Study as project areas for the Urgent Drainage Improvement Project.

a. Those areas in Wadi Musherib where the groundwater level is less than 1.5 m below the ground surface (some 150 ha, wadi type).

b. Those areas in Rayyan where the groundwater level is less than 1.3 m below the ground surface (some 70 ha, inland depression type).

c. Those areas in New District where damage to structures and vegetation is likely to be caused (coastal type).