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Water in the Gulf Region toward integrated management

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Ground Water Information System of Bahrain

Dr. Jasminko Karanjac

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GROUND WATER INFORMATION SYSTEM OF BAHRAIN

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GROUND WATER INFORMATION SYSTEM OF BAHRAIN

INTRODUCTION

Bahrain is in the process of establishing a state-wide ground water information system known as the **Bahrain Ground Water Information System** (BGWIS). The establishment of BGWIS is one of activities of the United Nations Development Programme (UNDP) supported project **"Water Resources Management Plan of Bahrain."**

The water sector of Bahrain is heading for a major crisis as early as by the year 2010 if the existing trends in the development of water resources are not reversed. This if due to severe over-exploitation of the existing resource base - ground water for agricultural, industrial and domestic water supply (Samoilenko, 1994). The total withdrawal of the ground water exceeds more than twice the natural recharge. This leads to the depletion of fresh water resource, decline of water levels, sea water intrusion and leakage of deep saline water; all resulting in the deterioration of water quality.

The development of ground water, its conservation, monitoring and filing of data and information about ground water is under jurisdiction of the *Water Resources Directorate* from the *Ministry of Commerce and Agriculture*. The Directorate maintains files on most of production wells, the number of which is estimated at about 2000. The Directorate operates about 70 ground water monitoring points (observation wells), monitors water abstraction and evaluates the status of the water resource base.

The new BGWIS will store most of available data on location of wells, lithology and stratigraphy of geological formations, water quality, water levels fluctuations over the period of time, abstraction from individual wells, testing of water-bearing formations (pumping and step-drawdown tests), and the like. It is expected that the fully established BGWIS will contain about 2000 production and exploration wells, about 2000 samples with chemical analyses, about 70 observation wells with long-term observation period, 50 pumping tests, and more than 100 step-drawdown tests.

The *Ground Water for Windows* (GWW) software is used to create the BGWIS. The software, which is in public domain and of which the writer of this presentation is a co-author, is the United Nations contribution to the ground water data information processing and management.

GROUND WATER FOR WINDOWS SOFTWARE

Ground Water for Windows (GWW) software is an object oriented, relational data base and *ground water information system* (GWIS). The software integrates classical ground water information (lithology, hydrographs, chemistry, pumping tests, etc.) with geographic displays. The software is programmed as a Windows™ application, and it uses all Windows resources:

memory management, fonts, colors, screen, printer and plotter drivers, etc. The user designs the structure of the data base and creates various data entry forms. One of entry forms, tailored for BGWIS, is shown in Figure 1. The data transferred into the data base using this "Master" form are general data on well location and surface elevation, and some Bahrain-specific data on geometry of major lithostratigraphic units (Khobar, Alat, Umm Er Radhuma, etc.). (This and any other form can be modified, revised or updated at any time, adding new data fields or deleting existing ones.)

GWW is a highly integrated ground water information system. The current version has the following applications built in:

- Chemical data including concentration-time and concentration-depth series.
- Pumping test data.
- Well logs and well construction data.
- Water level data.
- Step drawdown test data.
- Grain size distribution curves and calculations of hydraulic conductivities using empirical formulas.
- Various hydrogeological calculations, such as well functions, drawdowns, and miscellaneous well construction data.
- Cross sections and fence diagrams.
- Mapping.

Using the data entered through the *Well Log and Well Construction application*, GWW will create lithologic cross sections, or lithologic and stratigraphic fence diagrams.

The data base can be filtered; that is a large set of information can be reduced to subset having user-selected attributes on any part of the data. Figure 2 displays an example with a chemical data subset. Common to all these samples is that they are taken from the Khobar aquifer, and each has the total dissolved solids (TDS) content greater than 10,000 ppm. It is an easy step in GWW to create a site map showing the sites of these samples.

The software is primarily intended for the creation of large relational data bases on a site, region, province, county, or country wide basis. Figure 3 displays the map of Bahrain island with locations of all abstraction wells currently in the data base.

GWW is language independent. Hydrogeologists from the Palestinian Water Authority and its Water Resources Action Programme have created a bilingual Master data entry form as shown in Figure 4.

In addition to ground water data processing, GWW has its own powerful mapping utility. Various location and thematic content maps become an integral part of the ground water base. The map in Figure 3 is one of such maps. Other thematic maps that are stored in BGWIS are the following:

- TDS contour maps for each aquifer.
- Piezometric maps for various aquifers and parts of the island. These maps are automatically created by GWW for selected dates.
- Geometric contour maps showing the relief (topography) of the top or bottom of major lithostratigraphics units, such as Khobar limestone, Alat limestone, Rus anhydrite, Umm Er Radhuma formation, etc. These contour maps, which extend to the eastern coast of Saudi Arabia (Dhahran, Qatif, Al Khobar, and Dammam), will be used as input data files for the modelling exercise.
- Aquifer parameter test site and contour maps (transmissivity, e.g.), to be used also as an input to modelling.
- Location maps showing sites of major abstraction at various scales, etc.

RETRIEVALS FROM BGWIS

Following the philosophy that a picture is worth one thousand words, GWW emphasizes graphics on screen and printed in: maps, logs, cross sections, hydrographs, various chemical diagrams, etc. One of typical maps stored in BGWIS is the map, Figure 3, showing sites of all drilled wells. An enlarged portion of the same map is reproduced in Figure 5, showing wells drilled in the northwestern corner of the main island.

Currently BGWIS contains information on drilling, construction, and lithology from about 500 wells. To display on the screen or print a well log for any well is a matter of seconds or at best one or two minutes. One example of graphical retrieval is shown in Figure 6. Using a combination of symbols (patterns) for lithology and colours for statigraphic units, one may at a glance notice the lithology and stratigraphy of formations the well drilled through. Here again, one may customize the display and replace English words with equivalents in Arabic.

In GWW one may create lithologic cross sections directly from a map using a mouse and selecting points one by one, by selecting a hand-drawn area and adding wells within a certain range from the cross section line, or by selecting a polygon area. A lithologic cross section is normally drawn using the information from the well log data base. To cross sections one may add ground surface elevation line, static or dynamic water level lines, and lines separating lithologic and/or stratigraphic units. One example of cross section is shown in Figure 7. The cross section displayed on the screen is in full color. It displays the ground surface, the contacts between major aquifer and aquitard units, and one or more water level (head) lines.

A separate utility titled Fence Diagrams creates automatically block diagrams using selected wells. The user controls angles of rotation and of view. Using hand-drawn lines the user may interpret lithology, fill layers with lithologic symbols and patterns, add various lines created in other parts of GWW, etc. One example of "three-dimensional" modeling of lithology is shown in Figure 8. It emphasizes the Khobar aquifer under Manama city.

Aquifer Testing Application

Pump tests, both constant rate to define aquifer parameters, and step-drawdown tests to define well's characteristics and optimum pumping rates for long term abstraction, are routinely practiced in every drilling campaign in Bahrain. It is believed that there are hundreds of such tests in "paper" files and reports. This information is one of most important in creating and running a mathematical model of the ground water system of Bahrain.

GWW handles the pumping test data as a data base and as a field-data processing programme. In the case of BGWIS, the processed data will be used to automatically create transmissivity maps for major aquifers to be used as input into the modelling software.

Hydrographs Application

The Hydrographs application within the BGWIS will be used to display hydrographs for each of wells making the monitoring network. Two such hydrographs are displayed in Figure 9, along with a small location map showing the sites of monitoring wells. Viewing these and other hydrographs one can easily detect a declining trend within the major abstraction zones. Also one can create various piezometric maps for selected aquifers, at selected dates, and for selected abstraction areas. One of such maps is shown in Figure 10.

Chemistry Application

The Chemistry application is used to create the chemical portion of the BGWIS with limitless number (except for practical reasons) of constituents and parameters; and to display on the screen and print some common "clean-water" graphs, such as Piper trilinear diagram, Wilcox irrigation criteria diagram, Stiff diagram, and Schoeller diagram. One "Schoeller" diagram is shown in Figure 11. It is illustrative since it displays the relative and total mineralization of selected wells from the Khobar aquifer. For contaminated sites, one may create composite reporting forms displaying in a map form the locations and values of contaminants over time, or, for the same time period, contaminant by contaminant contour map. Using a depth interval of sampling as a search-and-select criterion, one can create contour maps showing the contamination at certain depth intervals.

Of importance to Bahrain will be the presentation of chemical data as a time series. The deterioration of water quality on account of overabstraction is suspected to have taken place in some parts of the island. With BGWIS it will be easy to locate such places, to present contour maps for selected dates. Of particular importance will be a display of the type shown in Figure 12.

CONCLUSIONS

The *Ground Water for Windows* software is a state-of-the-art software and it is a public-domain software. Created for and by the United Nations, it became a software of choice for establishing the *Bahrain Ground Water Information System*. When all currently available data will be transferred into BGWIS, the data base will contain up to 2000 wells, up to 2000 chemical samples, 70 observation wells with long-term monitoring record, hundreds of pumping and step-drawdown tests, etc. Thematic maps, that are or will be created by GWW, will be stored within the BGWIS making it an information system. The data stored and processed within the BGWIS will be reported in periodic intervals in the form of Basic Documentation reports. The data will be used to grant permission or reject an application for a new abstraction well. Using graphical displays within BGWIS and Basic Documentation reports, general public may be better educated in understanding the ground water system, its vulnerability, extensions of water-bearing or water-transmitting formations toward Saudi Arabia, the deterioration of water quality due to overabstraction, and much more.

BGWIS will be used as a source of information for the modelling exercises, as a pre-processor to prepare input data for the model, and as a post-processor to present the results of modelling in a clear form for water resource managers to make decisions on the future use of ground water in Bahrain.

Considering the proximity of other countries within the Gulf region, the ground water information system created in Bahrain can easily merge with similar systems created elsewhere, provided an intelligent and nonambiguous design of data base structures is maintained.

References

Samoilenko, A. 1994. Mission Report to Bahrain under TSS-1 "Bahrain Water Resources Management Plan." United Nations DDSMS, April 1994.

List of Figures

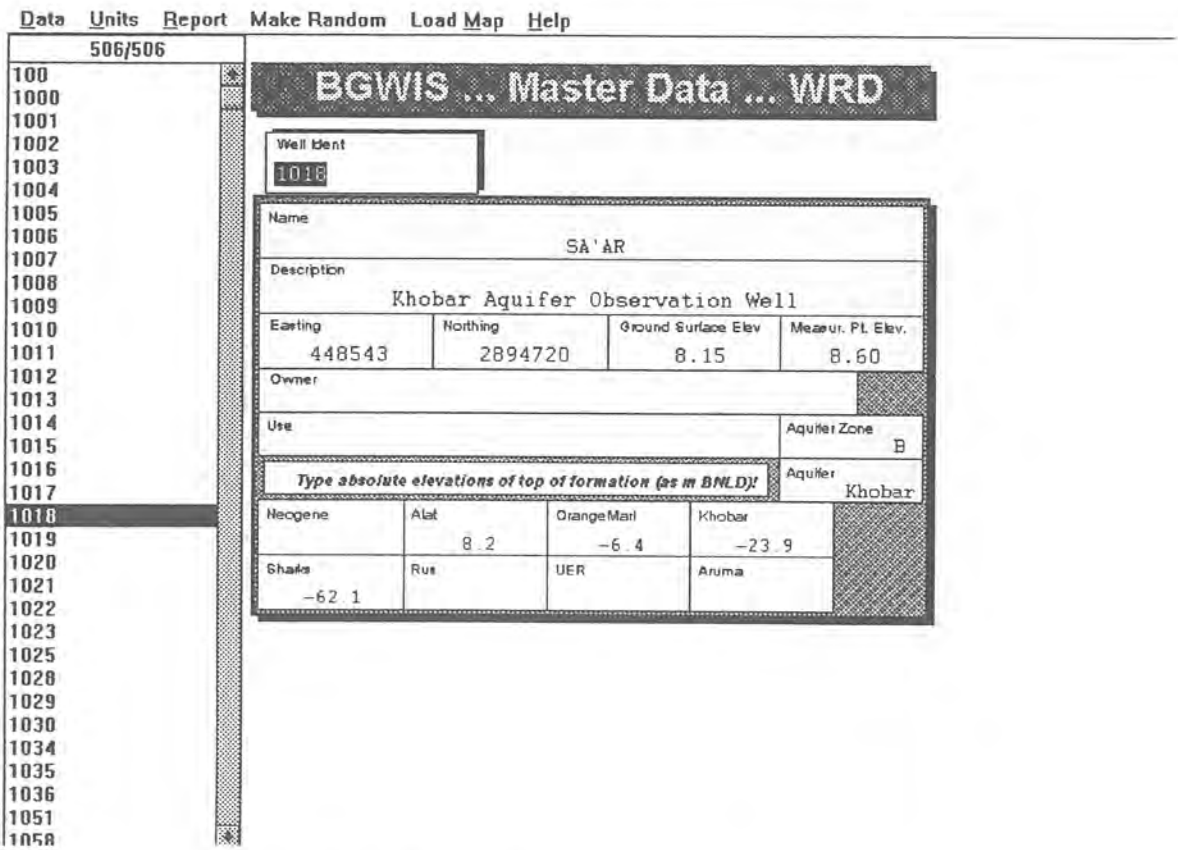


Figure 1. Master Data Entry Form.

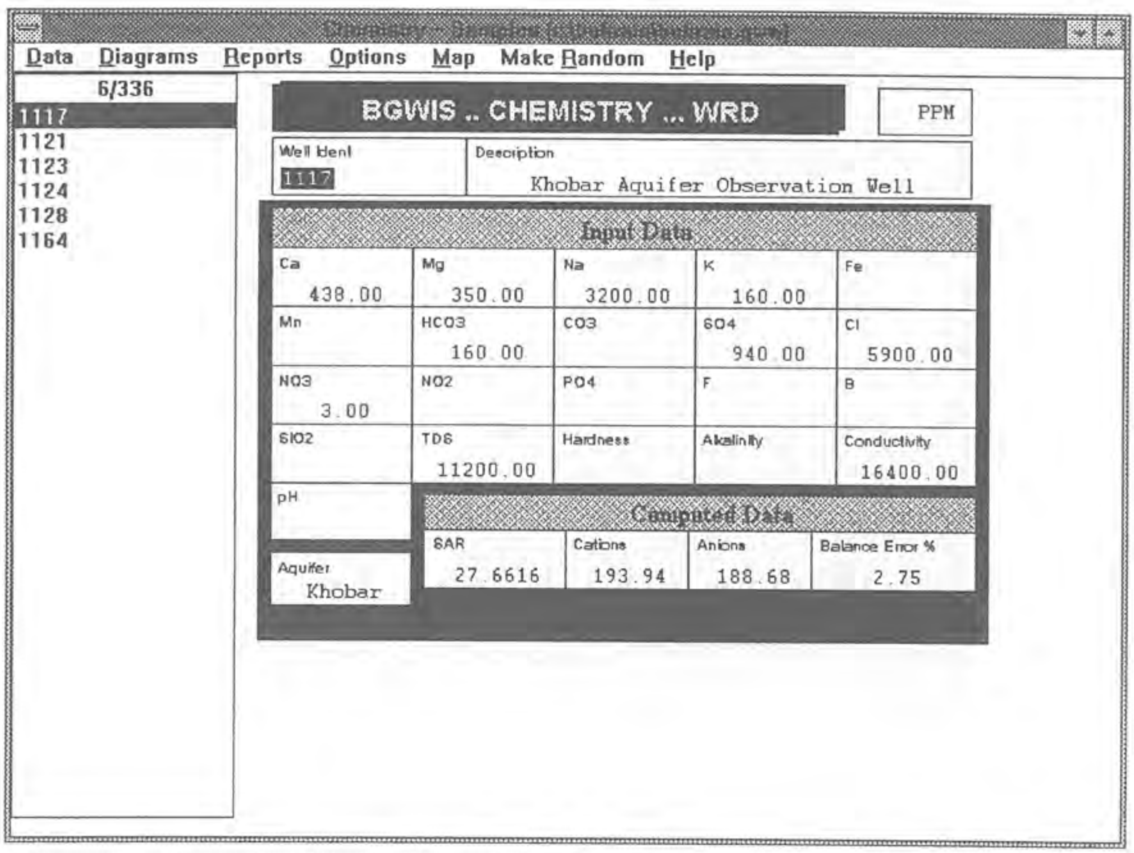


Figure 2. Chemical base filtered for Khobar samples with TDS>10,000 ppm

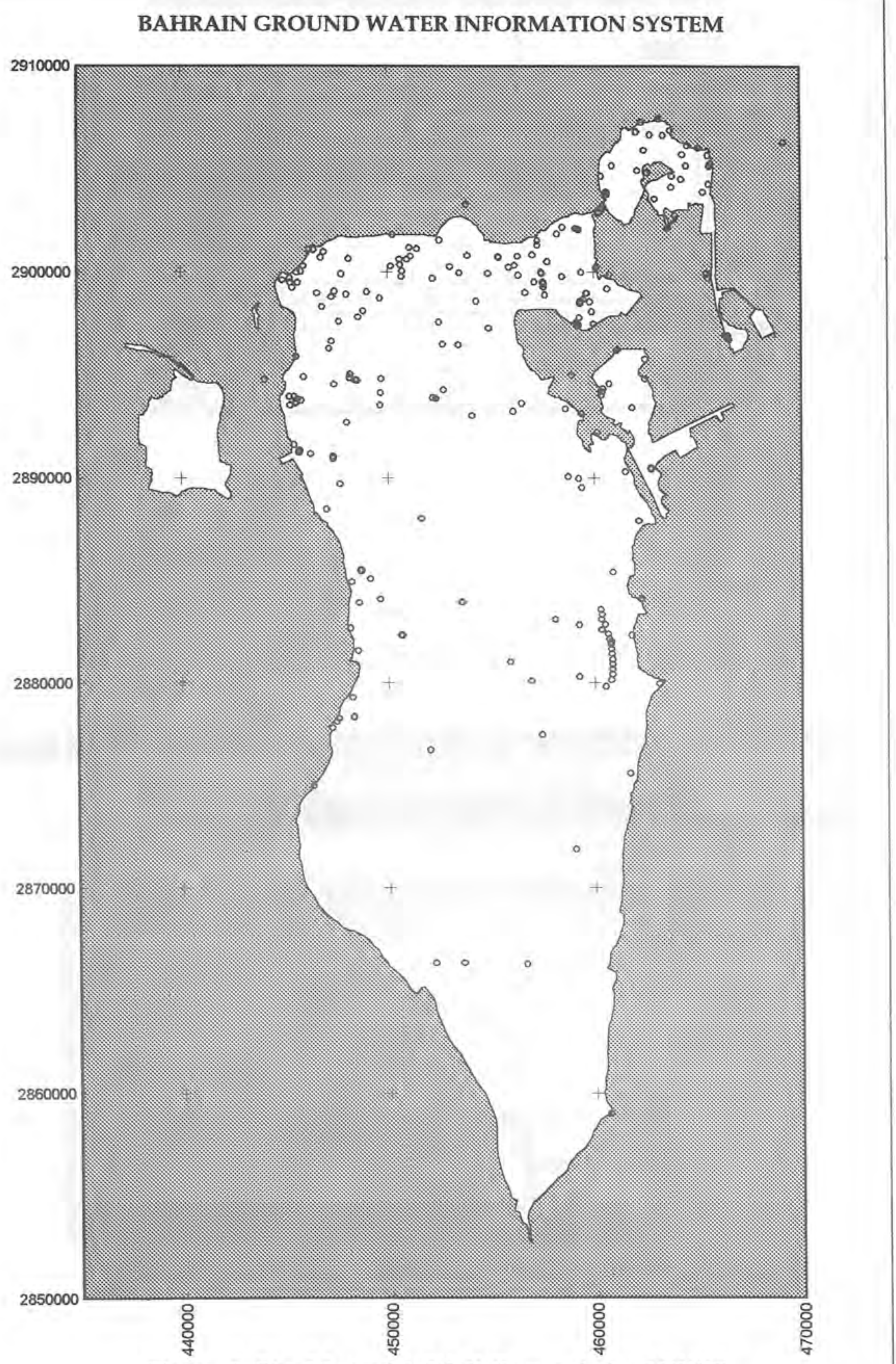


Figure 3. Site Map with Well Currently in BGWIS

Palestinian National Authority
Palestinian Water Authority
Water Resources Action Programme

السلطة الوطنية الفلسطينية
سلطة المياه الفلسطينية
برنامج إدارة مصادر المياه

بطاقة مورد مياه
Water Source Card

General Information: معلومات عامة:

Well ID: الرقم المصنّف للمورد:		Old (Israeli) ID:	الرقم القديم:
Source Name:			أسم المورد:
Type of Source: نوع المورد:		Ownership:	نوع الملكية:
Name of Owner:			أسم المالك:
Township:			البلدة:
Region:			المحافظة:
Easting: خط الطول:		Northing:	خط العرض:
Altitude (m.a.s.l.): الإرتفاع عن سطح البحر بالأمتار:		Use:	غرض الإستعمال:
License No.: رقم الرخصة:		Date Issued:	تاريخ صدور الرخصة:
Depth of Well: عمق البئر:		Date of Construction	تاريخ الحفر:
Drilling Method: طريقة الحفر:			
Drilling Contractor:			أسم متعهد الحفر:
Allowed Abstraction: كمية الضخ المسموح بها سنوياً:			
Datum Elevation: إرتفاع نقطة القياس عن سطح البحر:			
Depth to Water Level: العمق لمستوى الماء:		Date of Measurement:	تاريخ القياس:
Static Water Level (m.a.s.l.): المستوى الثابت للماء فوق سطح البحر:			
No. of Operating Days/year: عدد أيام العمل السنوية:			
Discharge (cu. m/hr): معدل الضخ بالمتر المكعب / الساعة:			
Drawdown (m): الهبوط في منسوب المياه بالأمتار:		Date:	التاريخ:
Type of Pump. نوع المضخة:		Date Installed	تاريخ التركيب:
Manufacturer:			أسم الصانع:
Power Source مصدر الطاقة:		Hourse Power:	القوة الحصانية:
Capacity (cu. m/hr): القدرة بالمتر المكعب/الساعة:		Depth of Pump:	عمق المضخة:
No. of Stages: عدد مراحل المضخة:		Intake Pipe Diameter (inch):	قطر ماسورة الولوج بالإنش:
Intake Pump Depth (m): عمق فتحة الماسورة بالأمتار:			

Figure 4. Example of an English-Arabic Entry Form

BAHRAIN GROUND WATER INFORMATION SYSTEM

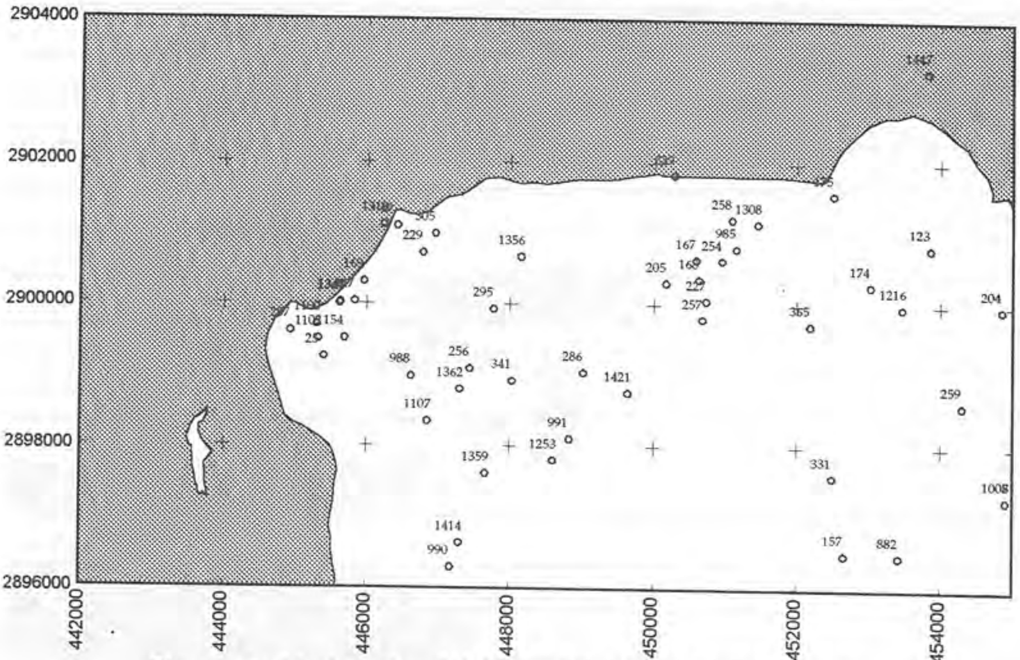


Figure 5. Abstraction Wells in NW Corner of Bahrain

BAHRAIN GROUND WATER INFORMATION SYSTEM

Well Ident	Description
1010	Umm Er Radhuma Observation Well

Drill. Method	Rotary with bentonite mud	Drill. Dates	16/05/79 to 17/06/79

X 448305	Y 2878350	Z 1.70	Meas. Pt. Elev. 2.60

All measurements are in meters.

Scales (1: xxx)

Water Level (MSL)	Level-Date	Vertical	Horizontal
-1.50		1400.0	50.0

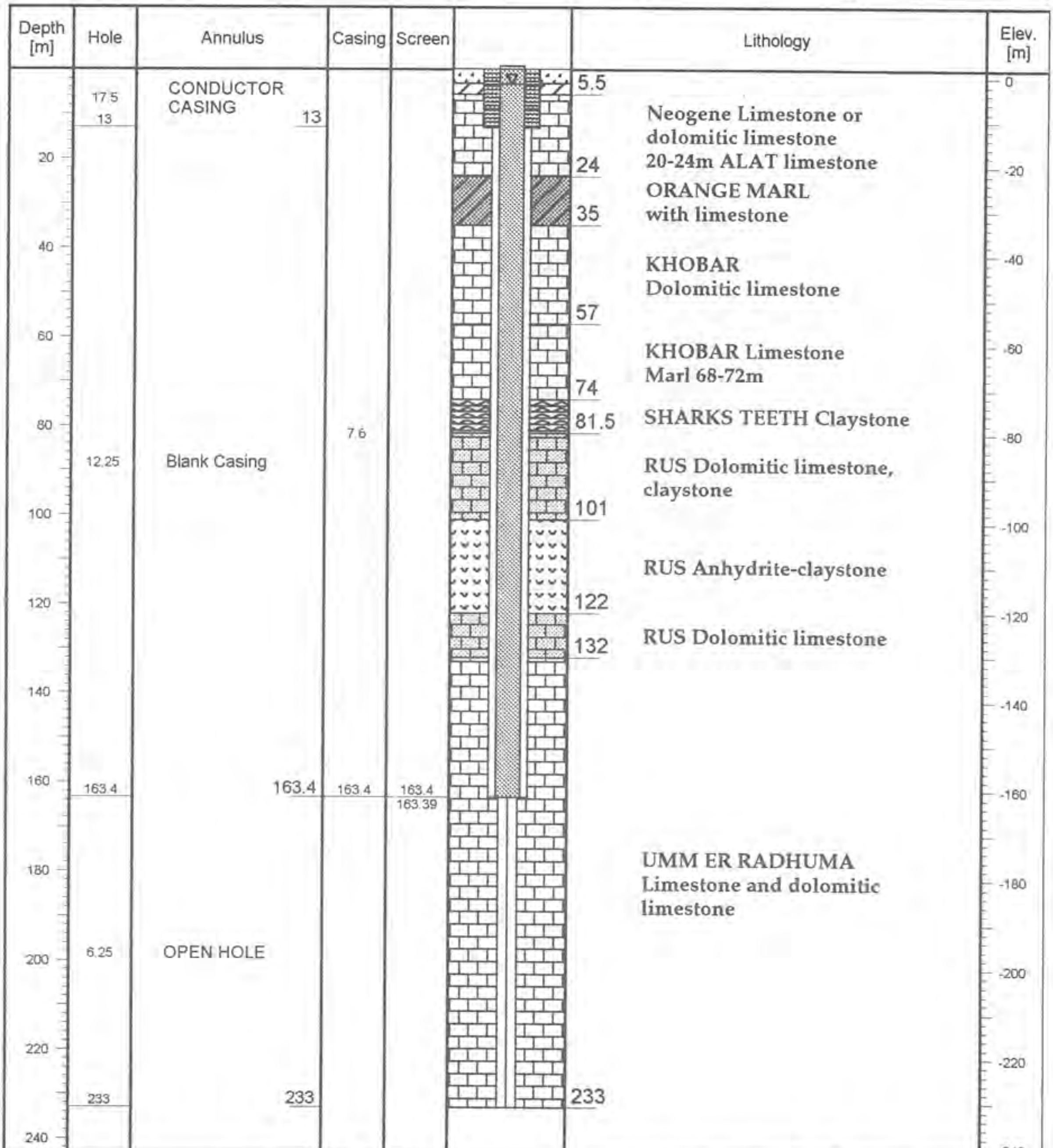


Figure 6. Example of Retrieval/reporting from BGWIS: A Well Log

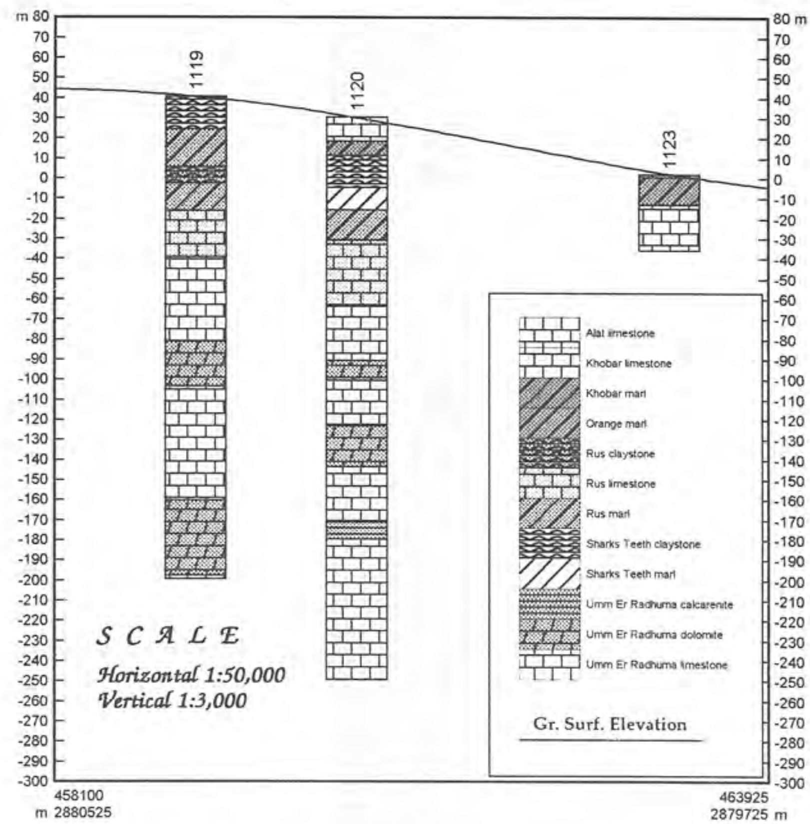


Figure 7. Litho-stratigraphic Cross Section

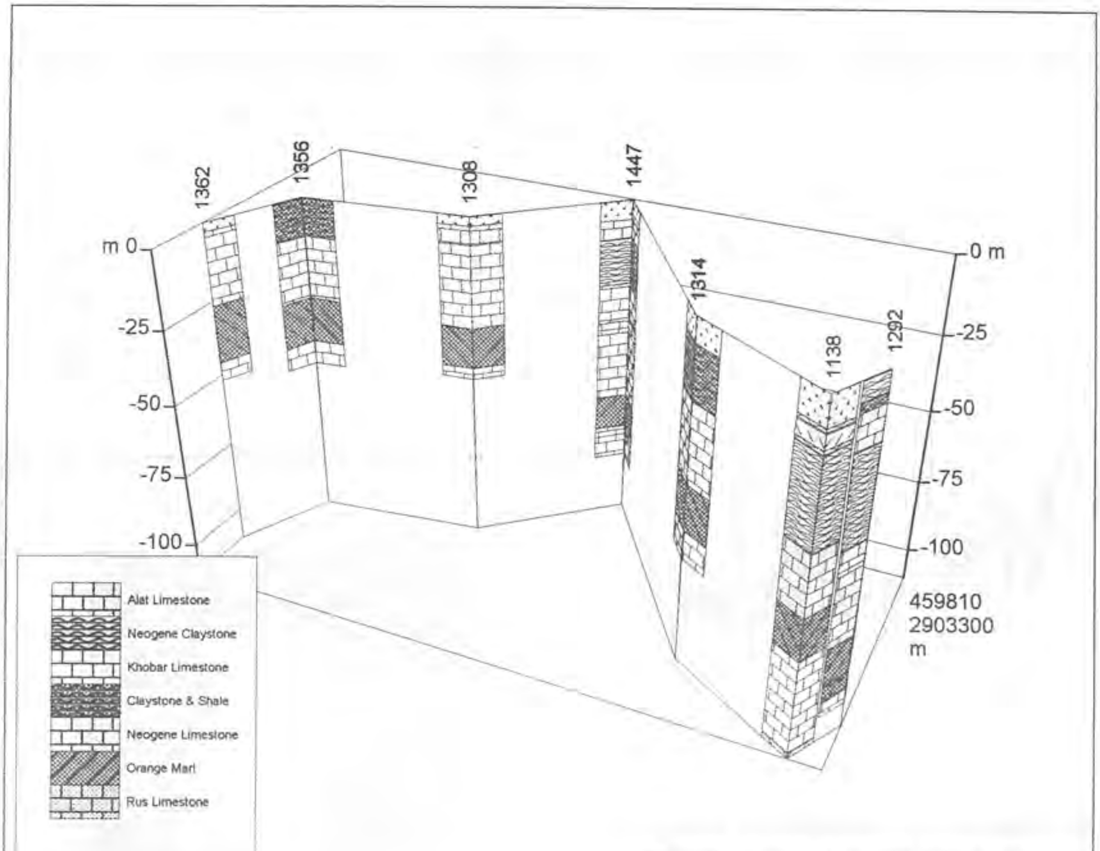
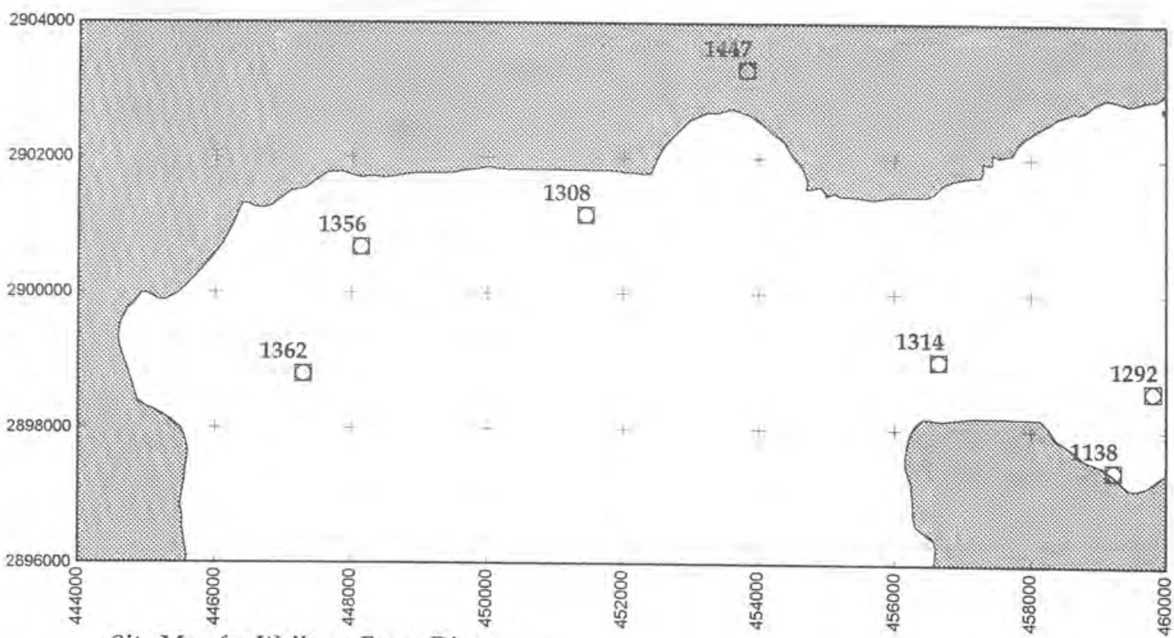
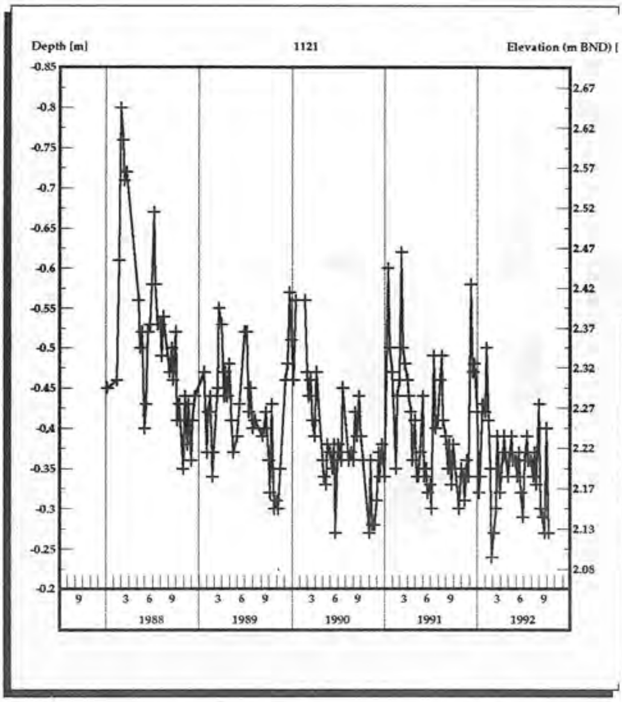


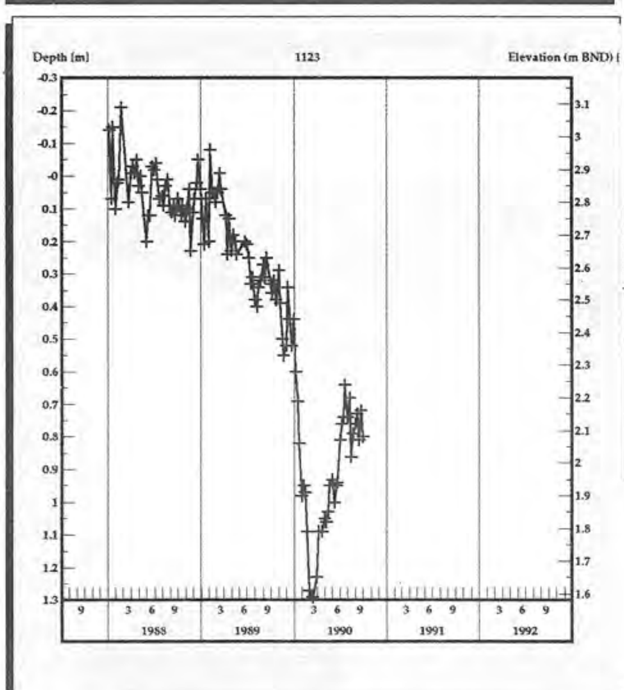
Figure 8. Fence Diagram under Manama



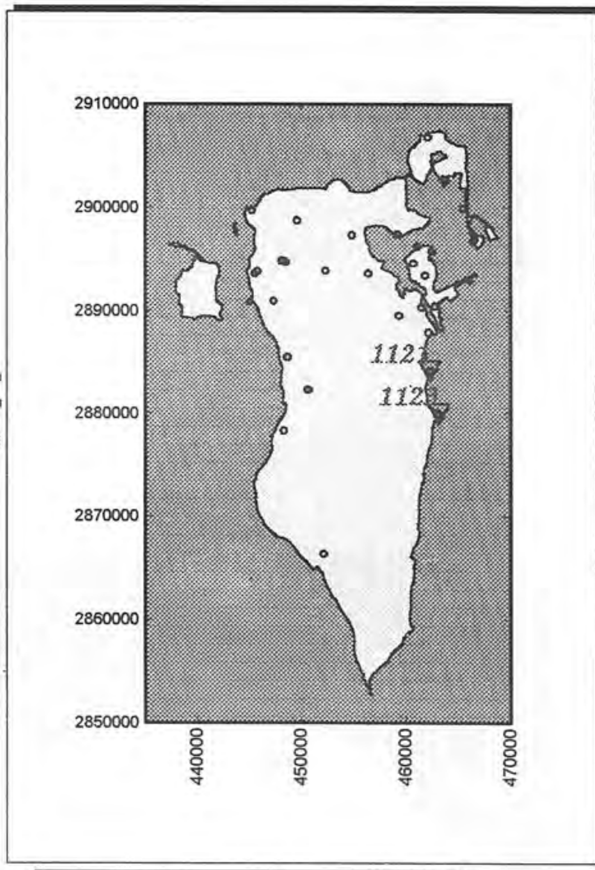
Site Map for Wells on Fence Diagram

Bahrain Ground Water Information System





Hydrograps from Monitoring Wells



Location Map

Figure 9. Hydrographs and Location Map

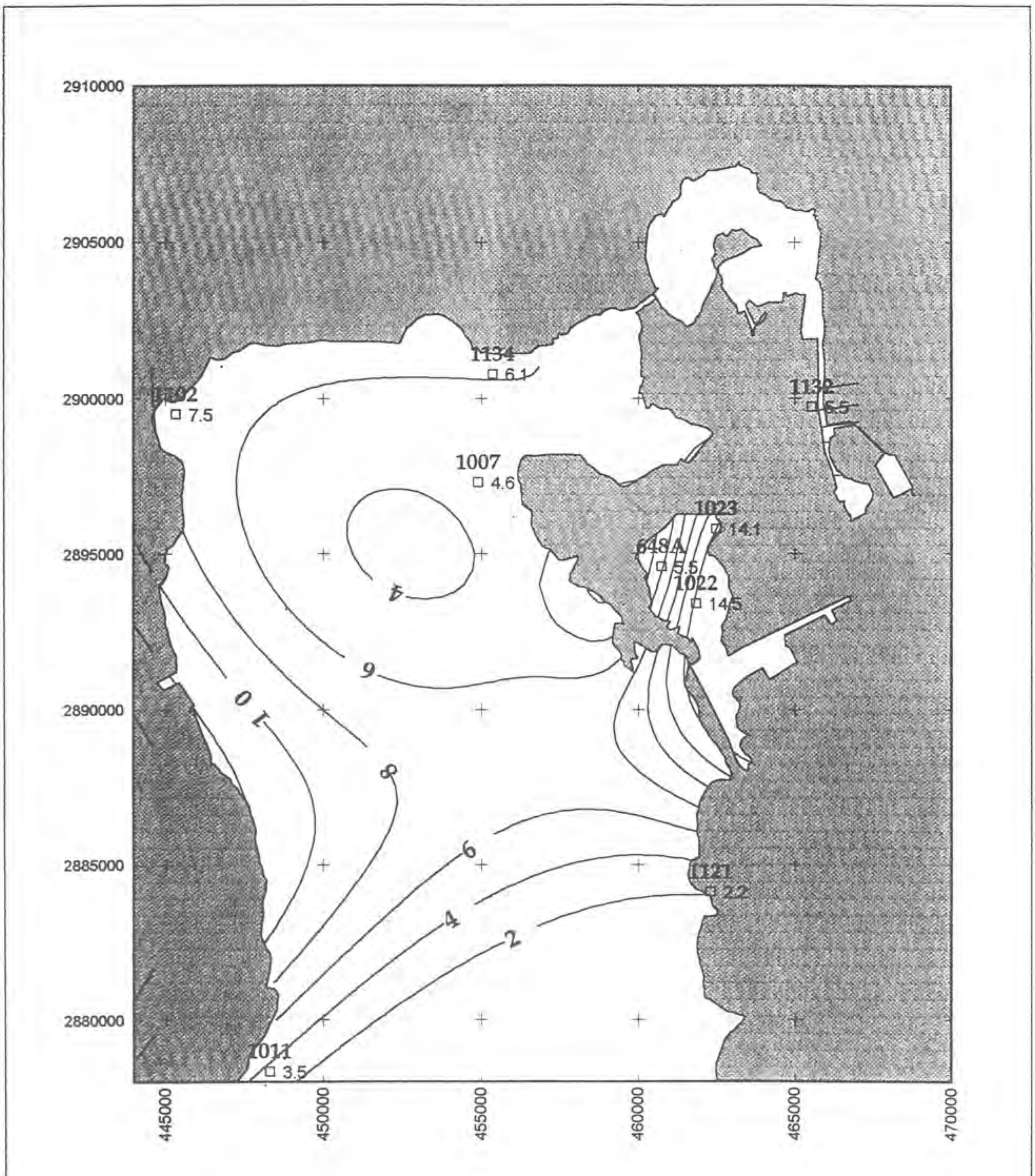


Figure 10. Piezometric Map of Khobar & Alat Aquifers for April 1992

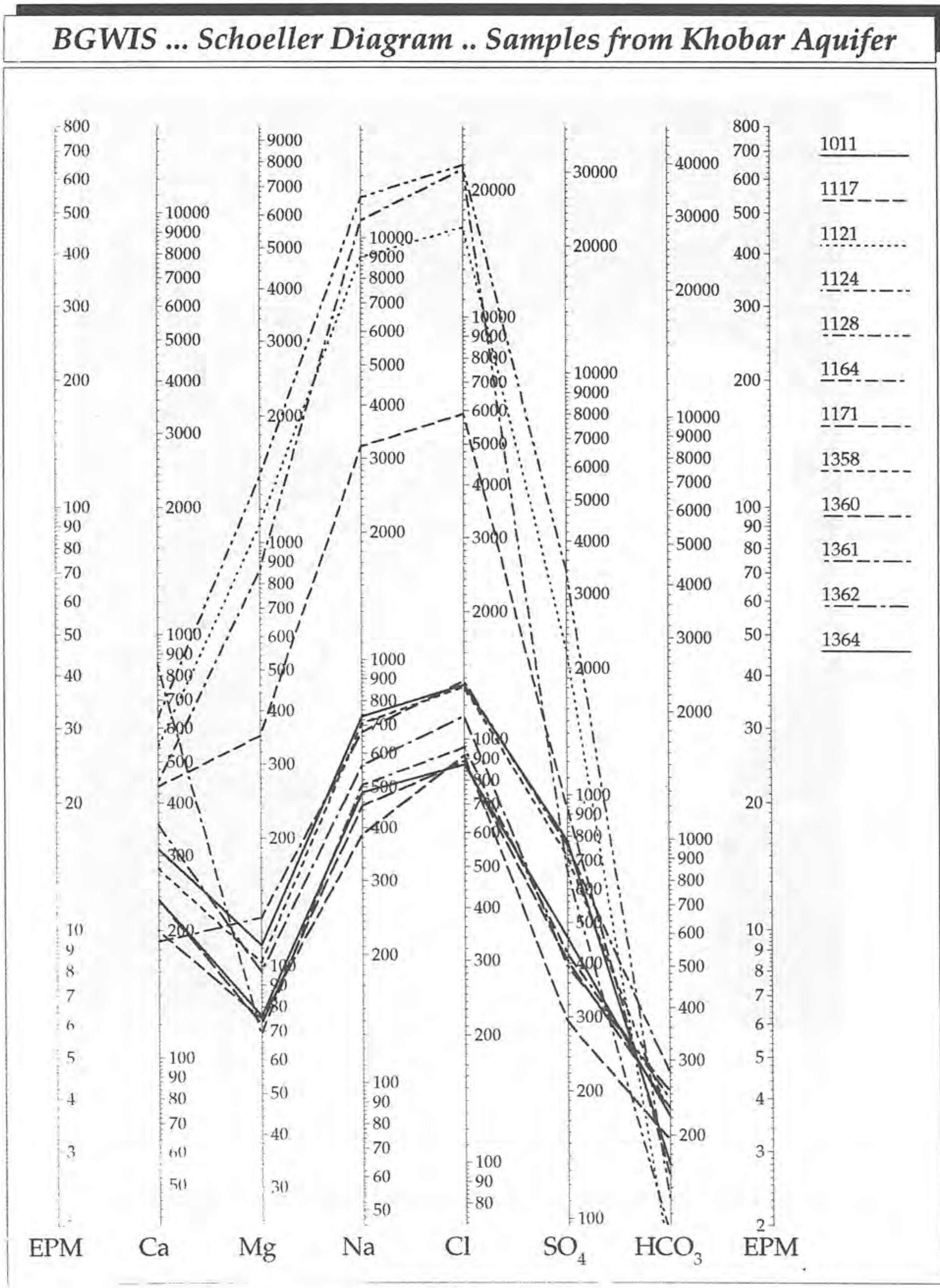


Figure 11. Schoeller Diagram for Khobar Samples

Bahrain GWIS .. Chemical Parameters Time Series

Well Ident	Description
1117	Khobar Aquifer Observation Well

X	Y	Z	ZM
461100	2896200	2.65	2.95

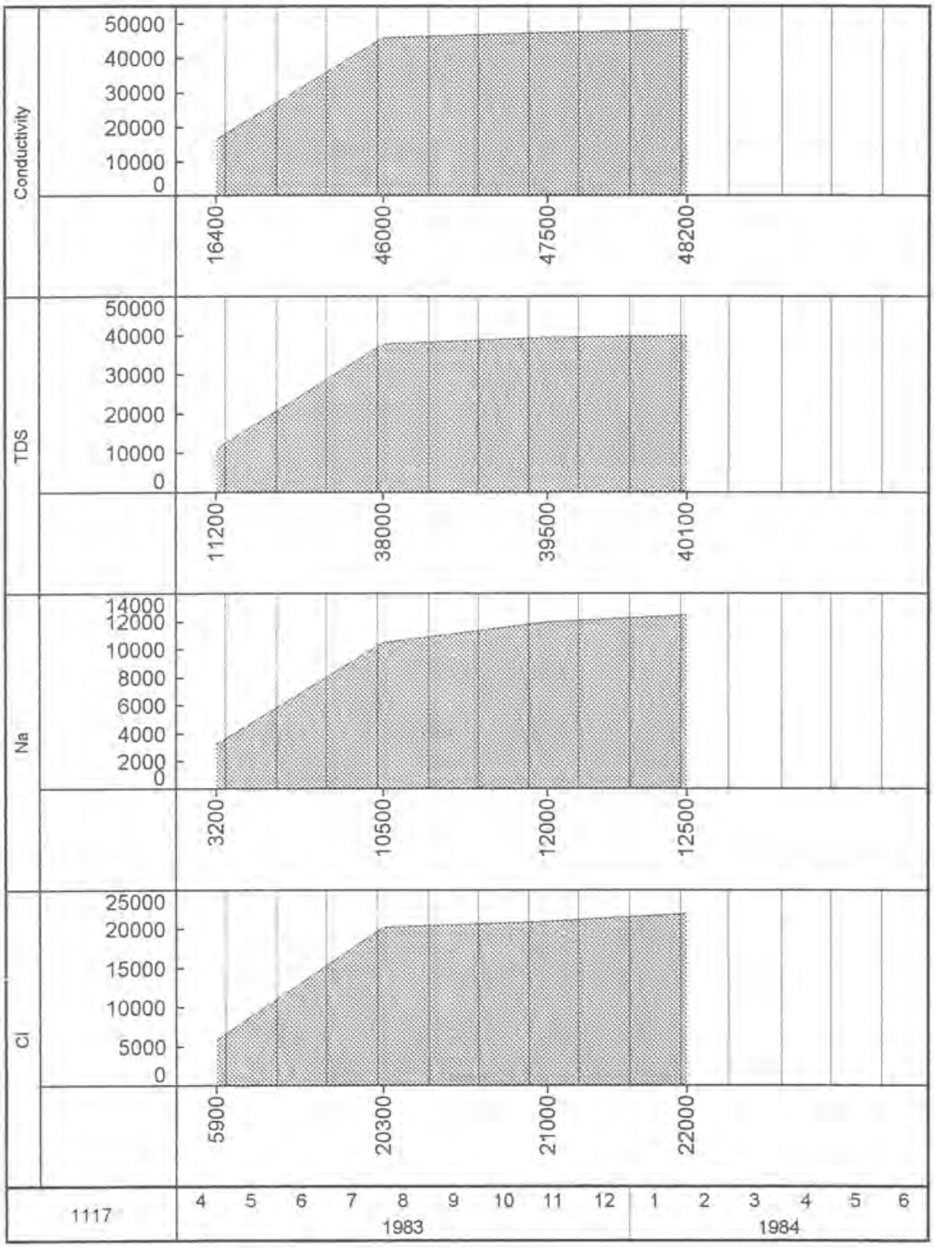


Figure 12. TDS and Other Constituents as a Time Series

Oman Water Resources: Management, Problems and Policy Alternatives

by

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Abstract

Oman is an arid country. Water abstraction exceeds annual recharge. Limited possibilities for new water sources exist. The government is working on augmenting water supplies, adopting conservation and improved management policies. This paper advocates flexible water policies responding to changing supply and demand. Involvement of communities is necessary for successful water management. Food security is recommended rather than a food-self sufficiency. Dependence on imported food may be resorted to in the short and/or medium term to alleviate pressures on strained water resources. Community education and awareness, administrative and pricing of irrigation water approaches are suggested as alternative integrated policies for water management.

Key words: Water resources, management, policy, Oman.

1. Introduction

Land and water resources are basic capital for mankind. They are largely location specific. Although, water is transferable to some degree, yet its transferability is governed by physical, economic, social, political and legal constraints. Proper use of limited water resources calls for a complex judgement on water resources supply and use planning, involving engineering, agricultural, economic, institutional and social factors that need to be integrated to reach optimum water resources use.

Each year increased demands are being placed upon Oman's scarce water resources. Agricultural, commercial, recreational, domestic, and growing industrial use are facing increasing problems of water quantity and quality. In response to the challenges posed by ground water overdraft in the major populated and crop producing region of the country, the "Batinah Area" (See Figure 1) and parts of other regions, the government started after mid-1970's extensive ground water studies, and took policy actions to increase supply, rationalize water consumption and call for water conservation. The purpose of this paper is to describe Oman's water resources, their exploitation and problems that are emerging due to water resource use and scarcity. Government policies and programs in the area of water management and conservation are also discussed. Alternative policies to manage scarce water resources are suggested.

2. Water Resources and their Exploitation

Oman is a country that has almost total arid climatic conditions. The country is highly dependent on rainfall for its fresh water supply and groundwater recharge. The main source of water is groundwater. Almost all fresh water in Oman, apart from domestic water supplies obtained from desalination plants, is from rainfall. Unreplenished fossil water aquifers are found in certain localities such as Al-Najd area in Dhofar region. Rainfall is limited and varies considerably throughout the country. Generally, it is described as sporadic and unreliable. A regular rainy season (monsoon) is experienced only on limited scale on Dhofar mountains. Frequent rains fall on Al-Hajar mountain range of Northern Oman (See Figure 1). The mean annual precipitation is estimated at 5,178 million cubic metres (MCM) of which 28 percent is lost as run-off. The hot climate causes 48 percent of total precipitation to evaporate, leaving 24 percent to recharge under ground water aquifers (equivalent to 1,240 MCM) [1].

Desalination plants were constructed to supplement domestic water supply in Greater Muscat Area and some other municipalities and villages. Recent discoveries of fossil water aquifers in Al-Najd area suggest that the estimated stock of water amounts to 5,000 MCM. The Ministry of Water Resources (MWR) estimates annual withdrawal of 100 MCM can be used annually to irrigate an area of approximately 2,500 feddans [2]. Figure 2 shows the annual use and supply of water in different regions of the Sultanate. Agricultural use amounts to 92 percent of the total water use. Most of the water used is in irrigation of crops, 94 percent, is consumed during the hot summer season (May to August) [3]. The consumption of wate- in agriculture has continuously grown since the 1970's, due to extensive and intensive increase in agricultural production. The area under crops was continuously growing as shown in Figure 3. The intensification of agricultural production is clearly demonstrated by the jump in total production by 328 percent between 1970 and 1993 [4]. During this period the Ministry of Agriculture and Fisheries (MOAF) estimated that the productivity per feddan tripled. Livestock numbers increased from 214,000 head to 1.4 million head, representing an increase of seven fold [5].

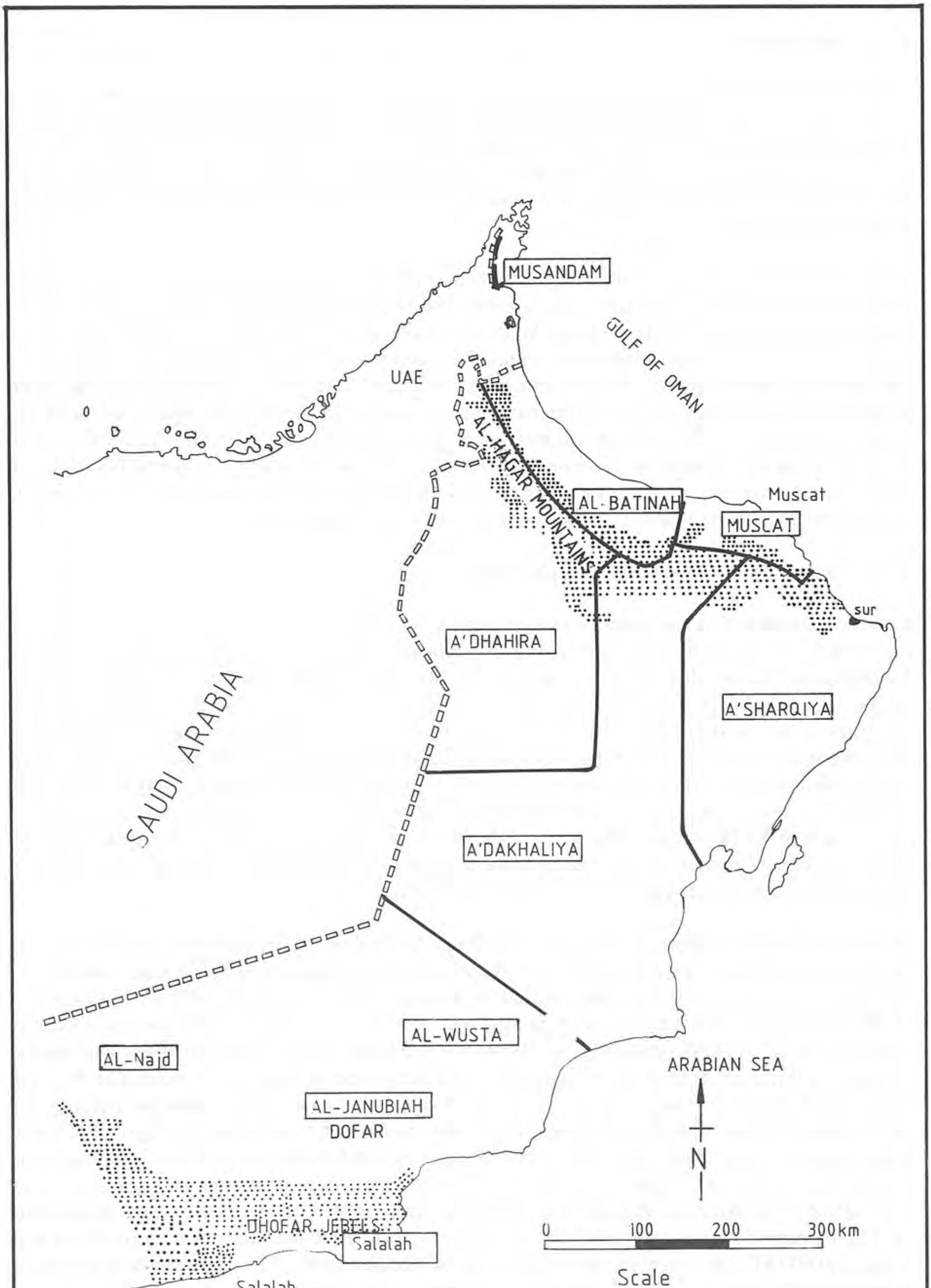

Figure (1) Sultanate of Oman Map

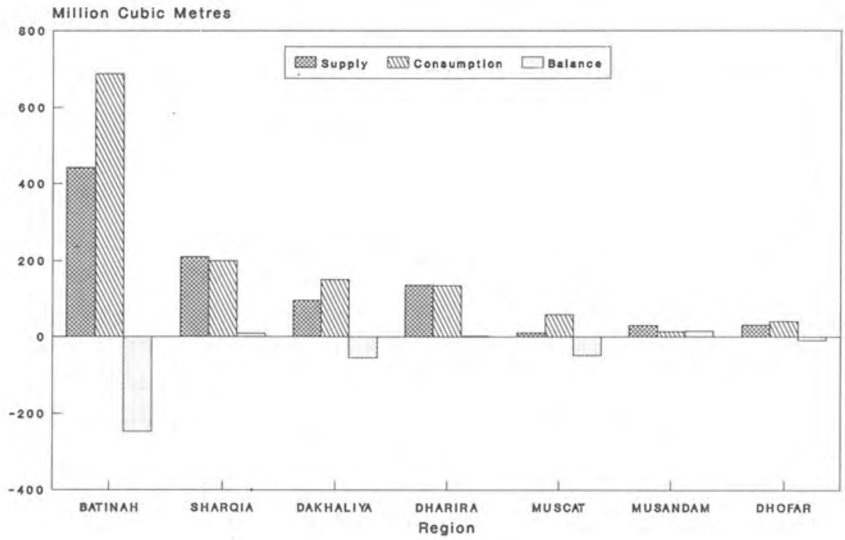

Fig. (2) Annual Supply & Use of Water in Oman [3]

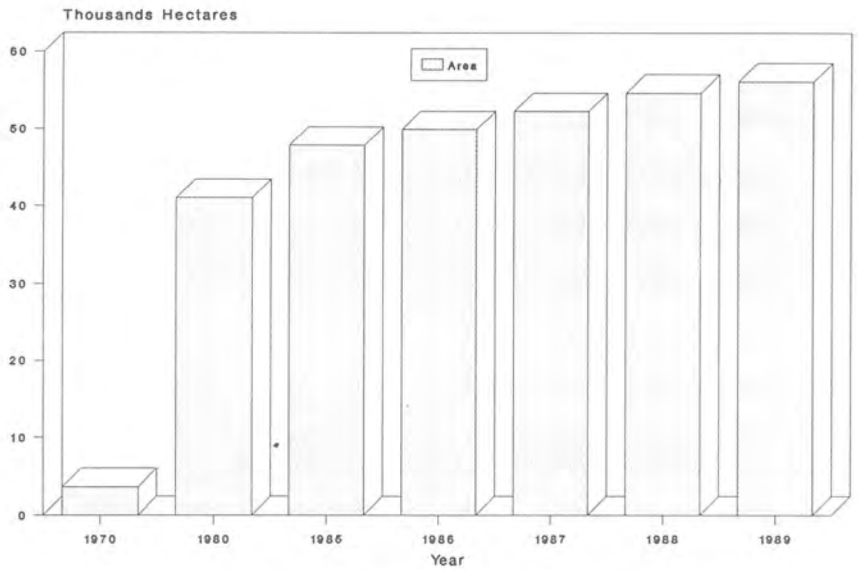

Figure (3) Area Under Crops in Oman [4]

Water used for domestic purposes increased tremendously over the last two decades. Piped water, which was not available before the 1970's, is now in common use in most urban areas. Net potable water use is now estimated at 70 MCM and expected to grow to 263 MCM by 2010 [6]. This being a result of the rising standard of living for the rapidly increasing population. Two main systems of irrigation are used in Oman, Falaj (plural Afalaj) and well systems. Falaj system is an old system of channelling irrigation water by gravity, sub-surface or in canals from springs and/or surface wells to farms. The use of irrigation water is controlled by the village(s) community in a very strict and well organized fashion. Water allocation is based on acquired rights, which can be passed through inheritance or rarely through purchase. It is estimated that approximately 70 percent of irrigation water in the Sultanate is delivered by Aflaj, which irrigate 55 percent of the cropped land [7]. Wells are found all over the country, with high concentration in Batinah.

3. Current Policies

The government awareness of the importance of sustaining water for agricultural and domestic use was clearly evident. A number of policies and programs were initiated over the last two decades to increase water supply, improve its use and conservation. These programs were mainly of institutional and technical nature. Technical work on improved water management practices included reduction of water loss in different uses, improvement of ground water supplies (through construction of many groundwater recharge dams, see Table 1), water harvest, and building of desalinization units.

Table 1. Groundwater Recharge Dams in the Sultanate of Oman [8]

Dam	a	b	c	d	e	f	g	h	i	j
W. Al-Khod*	1985	110	1674	14.0	5100	8.0	5.0	54.0	2.6	11.55
W. Khilts/ Salahi*	1986	126	554	8.8	9062	4.5	3.0	27.0	2.5	0.5
W. Quiryat*	1986	131	427	8.5	1630	5.3	3.0	32.0	0.18	0.125
W. Al-Jizzi*	1989	128	812	6.6	1234	20.4	6.0	129	1.25	5.4
Tanuf#	1989	160	157	5.0	135	14.8	3.0	36.0	0.12	0.7
W. Al-Ghul#	1989	164	173	4.8	415	7.6	4.0	45.0	0.16	0.45
W. Kabeer*	1991	148	753	8.3	2635	5.9	6.0	59.0	0.7	0.5
Al-Maawil*	1991	120	566	7.5	7500	8.3	6.0	38.0	2.5	10
W. Al-Fieleij*	1991	141	680	2.2	530	7.0	4.0	29.5	0.19	0.7
W. Sahnut*	1991	150	258	6.5	2355	22.8	-	-	1.1	6.4

Key:

a. Year of construction of dam.
b. Average yearly rainfall (mm).
c. Dam water surface area (km^2).
d. Length of dam (m).
f. Height of dam (m).
g. Top width of dam (m).
h. Maximum bottom width of dam (m).
i. Dam lake area (km^2).
j. Dam year's storage capacity. (million $m^{3)}$
* Earth dam.
\# Rock Filll dam.
\- Unavailable

Institutional building has been one of the concerns of the government to deal with growing water problems. Before 1975 a number of government ministries and agencies were supervising water resources management and use based on location and type of use. The MOAF supervised irrigation water, the Ministry of Electricity and Water dealt with potable water, the Minister of State for Dhofar worked with water projects in the Southern Region, and Musandam Development Committee worked with water projects in Musandam. In September 1975 the Water Resource Council was established under the chairmanship of His Majesty, the Sultan of Oman, to decide, as a central body, on policies and programs in the water sector (to be implemented by concerned ministries and government agencies). The council's main function was to coordinate exploitation of water resources to ensure a balance between supply. The Public Authority for Water Resources was established in December 1979 under the chairmanship of the Minister of Electricity and Water, to assist the Council for Water Resources in carrying out its duties. The Ministry of Environment and Water Resources was established in 1985 to reorganize water management activities. The Department of Water Resources of the MOAF was transferred to the newly formed ministry. The alarming decrease in water availability for different uses and deterioration of water quality led to the Royal Decree of November, 1988 which considered **water as a national wealth**. This decree was considered necessary in the face of the continuous depletion of ground water aquifers and decrease in water quality. Batinah coastal area was the most affected zone, because of seepage of sea water into ground water aquifers as a result of ground water depletion [9]. In 1989 the National Committee for Guidance of Water Use on the Batinah Coast was established by a Royal Decree. Before 1989 was over, the MWR was established to shoulder all the responsibilities of the previous councils and agencies dealing with water resources management.

A number of projects and programs were initiated by the MWR. These projects have three main objectives: 1) to assess available water, 2) to increase water supply and 3) to decrease water demand through conservation. To achieve the first objective, water resources assessment, number of leading international consulting firms were invited to do water inventory and management studies [10, 11]. Expatriate water experts were employed by different water authorities to help in studying water resources and develop alternative policies for proper water management. Starting 1990 a detailed survey of wells throughout the Sultanate was carried out by the MWR. More than 167,000 wells were registered and technical data have been collected on their depth, groundwater table, aquifer characteristics, quantity and quality of water extracted and other relevant information [12].

The strategies to increase supply of water in the Sultanate concentrate on three areas, water harvesting, desalinization and waste water reclamation and reuse. Water harvesting potentials were recognized from the fact that intense 'wadi' flows, following cyclonic storms, can be restrained to increase the seepage of water into the underlying aquifers. The annual amount of surface water lost to the sea is about 120 MCM [13]. Water desalination started in the Sultanate in 1976, when a unit was built to meet rapid increase in demand for domestic water in the capital area. Presently, about 80 percent of fresh water requirements of the capital area is supplied from desalination units. Smaller capacity desalination units have also been built in other parts of the country. The third source of water now is from waste water reclamation and reuse. Wastewater treatment works are mainly in the capital area with a total capacity of 28,400 cubic metres per day. Table 2 gives details regarding the existing wastewater treatment plants (WWTP's) in the Sultanate as well as the volume of the treated wastewater used for irrigation purposes. The table also illustrates the locations of the WWTP's, their design capacity, the actual flow rates in 1993, and the average effluent quality for Biological Oxygen Demand (BOD_5^{20}), suspended solids (SS), and ammonia-nitrogen (NH_3-N) [14, 15, 16]. Reuse of this water is restricted to the irrigation of landscape areas and parks, recreational activities and in fountains in the capital area. Sludges within each plant are subjected to digestion and dewatering. The stabilized sludges are used as fertilizers or soil conditioners.

Table 2. Wastewater Treatment Plants (WTP) in Muscat Area [17]

WTP Name	Design flow, m^3/d	Actual flow, m^3/d	Standard$^+$, mg/L BOD	SS	$NH_3(N)$	Reuse* (m^3/d)
Al Amerat	600	600	20	30	-	-
Al-Ansab	1200	5400	10	10	1	2000-5000
Al-Aynt	60	100	10	10	1	-
Al-Khod	1200	700	20	30	-	-
Bowshar	400	400	10	10	1	-
Darsait	10800	11500	10	10	1	8000-11500
Jibroo	70	100	10	10	1	50
Mabella	1920	700	10	10	1	-
Al-Qurum	1350	800	10	10	1	800
Total	28400	20300				10850-17350

Key:

+ Design standards for BOD_5^{20}, suspended solids (SS), and ammoniacal nitrogen. (All measurements are in mg/L).

* Reclaimed wastewater used for irrigation processes.

The strategies to conserve water use are based on two approaches [18]. The first is to decrease water loss due to water delivery system, and the second is to restrict well-drilling. As mentioned earlier, the two major irrigation systems in Oman are the Falaj and well-irrigation systems. The Falaj irrigation system is an old practice dating over eight centuries. Relatively, the Falaj community spend very little on water distribution system to/or within the fields. Based on research work on modernizing Falaj irrigation systems, the government embarked on an extensive construction program to maintain and repair Falaj systems [19]. The major objectives of the program are to preserve this efficient system of underground water tapping and decrease water losses through seepage or cracks in the delivery channels. The government through a subsidy program promoted the use of modern irrigation systems to save water losses from pump irrigation using traditional open channels to convey water to the fields, and basin irrigation of crops. Modern irrigation is a description of a system based on sprinkler, drip or bubble irrigation. Farmers willing to adopt modern irrigation systems are subsidized between 25 to 75 percent of the costs of the systems, depending on their farm size. For farms of 10 feddans (4.2 ha.) or less the subsidy is 75 percent of system cost. For farms between 10 and 50 feddans (4.2 to 21 ha.) the subsidy is 50 percent. The subsidy is 25 percent of the cost for farms over an area of 50 feddans. Estimates of water savings, using modern irrigation systems, as compared to traditional irrigation go up to 50 percent [20]. The policy to restrict well-drilling was initiated in early 1990's. Restrictions are now imposed on drilling of new wells unless a stringent set of rules are met. Increasing the depth of any existing well is not allowed unless the MWR issues a permit.

However, permission is only granted if the well is used for domestic consumption. Also, a restriction on drilling wells within a diameter of 3.5 km from a Falaj is prohibited [21].

The MWR is installing flow measuring devices on wells to determine quantities of water pumped. At a later stage there may be a limitation on the quantity of water that is to be pumped based on recommendations of the MOAF. The Ministry will base its recommendations on crops to be grown and their water needs. As an integrated approach for water conservation and management, the MWR is engaged on a public awareness campaign. Mass media, billboards, posters, lectures, seminars, and school syllabi are all used to convey water conservation messages. Religious beliefs, moral and environmental codes and themes were used in these messages.

4. Projections for Water Demand

The current imbalance between water supply and demand is clearly manifested in water shortages, groundwater depletion, deterioration of water quality and intrusion of sea water. Estimates of water demand by the year 2010 expect supply and demand for agricultural use will balance, but that of potable water uses will increase by more than three fold (See Table 3).

Table 3. Estimated Present and Projected Demand for Water in Oman [22]

Region	Agriculture (MCM/year)		Potable use (MCM/year)	
	1990	2010	1990	2010
Batinah and Capital Area	697	To	55.8	167.9
A'Dhahira	116		3.7	12.0
A'Dakhaliya	198	match	3.2	14.6
A'Sharqiya	224		4.3	29.0
Musandam	23	supply)	0.6	5.8
Dhofar	38		10.4	34.6
Total	1,278		78.0	263.9

The demand on limited water resources for agriculture, domestic, commercial and industrial uses is expected to increase continuously. A number of factors will increase water demand augmenting the problems of scarcity of water resources in the Sultanate. Agricultural needs for irrigation water are projected to increase. This will be a consequence of increased demand for agricultural products for human consumption and livestock feed. Factors leading to inc eased demand for agricultural products include an increase in Oman population, a greater per capita use of crops grown for food and livestock. The 1993 census results indicated that the total population of Oman was 2,017,591 with 74 percent being Omani nationals and 26 percent being non-Omanis. Table 4 shows the population and number of families as per region. A general observation suggest that with growing per capita income and more chances for better education levels, consumption of food crops will continuously increase. Expansion of agricultural production to new frontiers, like "Al- Najd" area, will also lead to an increase in water demand for irrigation.

Table 4. Population Estimates [23]

Region	Number of families	Total number of people
Muscat	92298	622506
Batinah	68373	538763
Musandam	4027	27669
Dhahira	21170	169710
Dakhaliya	27311	220403
Sharqiya	35943	247551
Central	2384	16101
Dhofar	22851	174888
Total	274357	2017591

5. Nature of Water Problems

Shortage of water, falling water table and increasing salinity are problems that lead to increased costs of irrigated crops because of the energy needed for lifting and distributing water. Heavy pumping of fresh ground water may allow intrusion of salt water from sea, in coastal areas, resulting in saltwater contamination of fresh water aquifers. Fresh water being less dense, overlays salt water in aquifers. The condition can be aggravated when the soil is already salt affected. Along the Batinah coast, the phenomena is highly manifested. In certain areas encroaching salt water made well water unsuitable for domestic use, and ultimately agricultural use. Monitoring of 270 wells on the Batinah coast indicated that the salinity increased by 38 percent [24]. Use of fertilizers, pesticides and other chemical substances can cause ground water contamination. Use of chemical fertilizers grew from almost nil before 1970's to an average of 15 thousands tons by 1990 [25]. Contamination from certain chemicals, like nitrate (which may cause methemoglobinemia in infants), can be a serious human health hazard. The use of pesticides and insecticides has increased over the last two decades. Government subsidies on fertilizers, pesticides and insecticides amounts to 50 percent of their market price [26].

6. Some Alternative Policies To Deal With Water Problems

Water availability could be increased through different technical options. The major concern is how can the society use and allocate water resources among its members. The uncontrolled use of a resource, in the case of a common property resource, leads to the social trap of each user maximizing his own use without recognizing society needs as a whole. Incentives for water conservation in agriculture, the major user, are few. It may even be claimed that water pricing and opportunity costs of water are not enough incentives to encourage users to invest in technology or gain management expertise needed to conserve water. The low cost of water in agriculture coupled with the lack of opportunity cost, and traditional market failures are apparent disincentives to water conservation.

There are certain facts that can be taken as background for any water policy. First is the fact that over 90 percent of fresh water use in the Sultanate is in agriculture, and that the major bulk of water use is during summer period (May to August) amounting to over 90 percent. The second fact is that water extraction systems are mainly of two types; Afalaj system and pumps. The third fact is that overdrafting already occurred in major water use areas (Batinah). Based on these facts the current government policies of resource augmentation, conservation and monitoring are encouraged to continue as long as they prove to be economically sound.

A major concern is how to raise particular community's involvement. The drive should not only be community participation, but stronger emphasis on their management of water resources. The Falaj system of irrigation is based on community management. There is no community management when pump irrigation system is practiced. Water overdrafting is actually happening in areas where pumps are extensively employed. The suggested role of the government is to provide community enabling environment, which would provide information, know-how, set legal framework, enable users to organize, and oversee the major improvement works which are subsidized by the government.

Previous studies done on water resource assessment and management were carried out by expatriate consulting firms. Oman needs to build a national cadre of water resource specialists to conduct research, manage and monitor activities on continuous long term basis. Minimizing dependence on expatriate labour coming from water rich areas, sometimes with no background in farming, should be advocated. Even passing extension messages to this labour force is difficult, due to language barriers and/or low levels of education.

A strategy of food self-sufficiency needs not be advocated if it means putting more demand on the already stressed water resources. Food security strategy do not put increasing demand on water and can provide the populous with adequate food supplies from domestic sources or from world market-using oil revenue. More dependency on imported food supplies may be needed, at least as a short or a medium term policy. This is to alleviate the pressure on already overdrafted water resources to regain aquifers water levels that can improve quality and counteract sea water intrusion. This policy can be supported by available oil revenue hoping that in future technological developments can provide cheaper methods of desalination of sea water, improve plants tolerance to salts and develop more efficient methods of irrigation. Available desalination techniques cost 30 to 40 times extracting water from wells [27].

The approach endorsed is a holistic and integrated one that recognizes no single solution to solve the problem. Such an approach recognizes both the supply and demand side of water resources management. Water availability could be relatively increased through different technical options, some of which are being practiced or there is planning to practice them in the Sultanate as indicated in section 3 of this paper. Water use can be altered through better water conservation, changing cropping patterns, and shifting use of water from low to high economic value uses. Kenneth Boulding wrote in 1980 that three major mechanisms organize u ilization of natural resources: price, policemen, and preachments [28]. Preachments and extending the message to all concerned about water scarcity and its conservation is practiced in the Sultanate. Religious and moral and nationalistic messages need to be stressed more. Emphasis on community involvement and responsibility in water management needs to be increased. Mass media, schools, mosques, and all public forums can be used as means to deliver messages in different forms and kinds.

Administrative procedures to control water use requires political will and clear vision, which is not lacking in Oman. Restrictions of water use to irrigate low value crops, banning of irrigation of seasonal crops during summer, and limit growing particular crops to certain localities are possible actions that can be taken. These policies can be implemented in the context of regarding user's acquired rights and their dependence on agriculture for their livelihood. Government incentives could be used as a support to administrative actions to decrease growing of tree crops or the number of trees. Horizontal expansion of crop production on soils, of marginal productivity, should be discouraged. Control on quantity of water extracted can be a possibility for restricting water use, especially with the MWR installing measuring devices to estimate well yields. Compensating payments for follow practice can also be used as a mean to conserve water.

Water is almost universally treated as a free resource and no charge is imposed for its withdrawal from its natural supplies. As an additional pillar in water use management, policy pricing of water is suggested. The use of price signals as a mean for water allocation can be tried. Water is considered as a public good, and as such pricing of water needs certain considerations. In the case of the Sultanate, a system of water tariffs and pricing could be effective way to decrease and economize water use in agriculture. The suggestion is to practice a pricing system in well irrigation areas only on the assumption that the natural flow of Falaj make it self regulating. In order to device a pricing system information needs to be collected on an estimated demand curve for water use in different regions and localities. From the demand curve, estimates of marginal value of resource use can be made. Cost of least expensive alternative for providing water can be used as a proxy for the value users willing to pay. An analytical model need to be established before the system can actually be implemented.

A minimum quantity of water can be given free of charge before a progressive price is charged against its use. The money generated could be used for increasing the supply of water or developing more efficient water use and distribution. For these suggested policy alternatives to work certain conditions need to be satisfied. Adequate information about water supply and potential uses need to be established. Clearly defined policies, methods of implementation and control procedures need to be set. It is better to take drastic measures to use the scarce water resources now before they can be damaged irreversibly.

7. Conclusions and Recommendations

Oman is an arid country highly dependent on rainfall for its fresh water supply and groundwater recharge. Rapidly increasing population, improved living standards and use of modern technological machinery in agriculture contributed towards straining and overdrafting of the country's water resources in certain areas. Sea water intrusion, increased salinity, depletion of aquifers are all being experienced. The government is keen on improving the deteriorating water conditions. Institutional reforms are continuously being implemented for the last two decades. The MWR acts as the government body responsible for control of water resources. The government strategies to increase water supply concentrate on water harvesting, desalinization and wastewater reclamation and reuse. On the demand side, the policies advocate water conservation through use of modern irrigation systems and wastewater reuse and community education.

The paper stresses the importance of an integrated water policy, that respond to changing conditions of supply and demand. The suggested strategy not only calls for local community participation in water management programs, but active involvement in devising and implementing these policies. Building a national Omani cadre of water research and management experts is seen as an important facet in the strategy. A food security strategy that do not put an increasing demand on the already strained water resources is seen as a better alternative than striving for food self-sufficiency. The domestic food production can be augmented with imports, using oil revenues. Administrative procedures to control water use, restrict it to high value crops and decrease areas of summer crops is recommended. Use of marginal land needs to be restricted to conserve available water for more productive land.

A policy of putting a price on irrigation water is suggested. Water, extracted over a certain amount, can be charged a progressive rate. The price system is suggested for pump extracted water. The price system makes use of MWR decision to install flow measuring devices on wells. The changing conditions of water supply and demand call for continuous research and monitoring of resources. The information available about water resources, their use and problems can be made accessible by researchers, policy makers and the public at large for a better understanding and devising alternative management policies.

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BAPCO'S Development for The Reduction Ground Water Usage Within the Refinery

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BAPCO'S DEVELOPMENT FOR THE REDUCTION OF GROUND WATER USAGE WITHIN THE REFINERY

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ABSTRACT
The Bahrain Refinery started production in 1936. The water source for the refinery was from Zone 'B' Wells at Budaiya. With installation of additional process units the water demand increased and this led into drilling of more Zone 'B' & 'C' wells at the refinery to accommodate the extra demand. Furthermore, concern foe ground water extraction resulted in the installation of sea water Desalination Plants and in 1979 the Budaiya wells were secured. As a continuation of Bapco's commitment towards the conservation of the Zone 'B' water, an 11 Million Dollar project has been approved to supply the Refinery domestic water by sea water desalination plant and virtually eliminate extraction from the Zone 'B' aquifer to help reduce the current State-wide over-abstraction.

INTRODUCTION
Bahrain Refinery produced its boiler feed water with the installation of the first boiler in 1936.

Water supply for this purpose was supplied from a water treating plant that was built in the refinery. The normal water supply to this plant was from 2 Zone "B" Wells at Budaiya which was treated and distilled and used as boiler feed water. This aquifer is an extension of gelogical structures existing under Saudi Arabia.

Aquifer "B" is the only aquifer from which water is abstructed for domestic use at present.

The Refinery has been expanding since 1936. In 1973 a major unit for low sulphur fuel oil was commissioned. This unit is a major boiler feed water consumer. To meet the increasing demand of the boiler feed water Bapco built two 450,000 USGPD sea water desalination plants instead of increasing the capacity of the water treating plant as a policy to reduce the abstration of Zone 'B' water. The quality of underground water on the island has been deteriorating due to sea water intrusion caused by the continuous drop in hydraulic head of the equifer.

Bapco built its third sea water desalination plant in 1979 with capacity of 900,000 USGPD as part of the project to Centralise Power and Steam Generation facilities (CPSG) and the water treating plant was secured and as such the Budaiya Wells were shutdown.

GROUND WATER ABSTRACTION (Refinery Wells)

As far as the records show, the ground water abstraction in 1986 was:

Zone 'B' = 500 GPM
Zone 'C' = 1500 GPM

Whereas nowadays it is:

Zone 'B' = 300 GPM
Zone 'C' = 1300 GPM

The continued over abstraction had adverse effect on ground water quality especially in Zone 'B' aquifer. Well No. 11 was abandoned in 1968 when it interacted with well No. 10. At this time its salinity was 13000 PPM. Well No. 13 was always the most saline of the Zone 'B' Wells and it was closed in 1971 due to high salinity of 17000 PPM.

In March 1961 a study was carried out and it was stated that hydrology studies indicated that salt water infiltration into Zone 'B' was from South-East and those wells to the South-East would be the most affected initially. However, because the wells interacted, the overall quality of the Zone 'B' water continue to deteriorate as would the individual wells. The future salinity of Zone 'B' water would be a function of the rate of off-take from the refinery wells and the rate of decline of the hydraulic gradient of the aquifer. Trending of salinity in wells Nos. 10, 11, 12, 13, 42 & 44 from 1946 to 1972 gave an average increase of 200 PPM per year. Taking a base value of 7000 PPM as the average Zone 'B' salinity in 1960 the projected salinity in 1995 would be 14000 PPM. This value is similar to present trended values. (See Fig. 1,2, 3 & 4).

EXISTING ZONE 'B' WELLS IN REFINERY

Recently there has been a noticeable deterioration in the quality of the Zone 'B' water wells No. 42 and No. 44. This is of crucial importance since these wells supply the potable water to the refinery. The Zone 'B' water is blended with flush evaporated distillate (FED) at an approximate ratio of 5/95 Zone 'B' to FED to make drinking water. Their present salinities are in the 11000-12000 PPM range but more importantly is the steepness of the recent changes. The World Health Organisation (WHO) guidelines for potable water specifies that the highest desirable chloride is 330 PPM and the maximum permissible level is 988 PPM. As the well water in Wells Nos. 42 & 44 became more saline then the use of this water in blending for drinking water becomes less desirable. If these wells continue to deteriorate at their present trend they will be following the same trend as Well Nos. 10 & 12 prior to their shutdown.

MONITORING

At present the wells are being monitored on weekly basis for the chloride trend and on monthly basis for the chloride, conductivity and hardness.

PRESENT DOMESTIC AND DRINKING WATER SYSTEMS
Two systems supply water for personnel use:

a) Domestic (Zone 'B')
This is supplied from water wells Nos. 42 & 44 and used in:

Toilets, wash basins, Urinals, Showers etc.

The system pumps are located at the North end of the refinery and abstracting water from Zone 'B' aquifer.

The water is hard, saline and corrosive and not suitable for potable use.

Existing Domestic Water Consumption
The consumption on domestic water is fairly constant throughout the day at about 432,000 USG/day (300 GPM).

b) DRINKING WATER
Is a mixture of 5/95 Zone 'B' water to FED.

This is used in:

- Drinking fountains, kitchen, Quality Controlled Laboratory, Clinic use.

Over the years various problems have been developing with both water systems. These have culminated in complaints about the water quality and quantity.

Bapco commissioned a Consultant to investigate both water systems and to recommend solutions to these problems for both the short and long-term. In addition Bapco's long-term policy is to stop the use of well water completely and to have a combined distribution system to serve both domestic and drinking water demands. Hence, this will virtually eliminate abstraction from Zone 'B' aquifer to help to reduce the current state-wide over abstraction.

As a result, a project at a cost of $11 Million Dollar has been approved to serve both the Drinking and Domestic Water Systems.

FUTURE WATER DEMANDS
- Future Domestic Water Demands

A steadily increasing consumption of Domestic water has been indicated throughout the year as indicated in the research carried out during 1989.

It has been agreed that a suitable factor shall be applied to Domestic water flows to allow for an expected immediate increase in consumption once distilled water becomes more readily available within the refinery. This factor is very difficult to estimate. However, it is suggested that a factor of 2 is applied to the Domestic water element of consumption is sufficiently conservative to allow for this increase.

Thus future per capita Domestic water consumption = 0.0069 X 2 = 0.0138 GPM/head.

FUTURE WATER SUPPLY SYSTEMS

- Design Concept

Due to the following:

- The poor condition of the raw domestic water system.
- The lack of spare FED from existing plant.
- The existing limited capacity of the drinking water system.

It has been decided to install a new desalination plant and a new combined, drinking and domestic water system, supplying potable water to all domestic outlets within the Refinery. Consumer metering points will allow effective monitoring and policing of usage. Wherever possible, the supply piping would be protected from solar temperature gain, by burying or by insulating the piping.

The design relies on chemical dosing only to produce the desired drinking water quality, and Zone 'B' extraction will be discontinued. This complies with the national incentive to reduce or eliminate further extraction from Zone 'B' aquifer.

A number of sea water desalination plants were looked at such as:

- Multi stage flash (MSF).
- Multi effect stock (MES).
- Reheat thermo compression (RTC).
- Reverse Osmosis Process (RO).
- Vacuum Vapour Compression (VVC).

Of all these process, reheat - thermo compression has proved, on budget inquiries, to be the most cost effective in all respect.

The unit is capable of producing 300 GPM of high grade quality water. The basis for this selection is:

- Produce approximately 180 GPM; average consumption, for domestic water purposes.
- Supplement the existing desalination plants capacity with 120 GPM of high grade quality water.
- Meet peak domestic water consumption of 300 GPM.

PRODUCT QUALITY
The design quality basis used were the World Health Organisation 1984 Guidelines.

The water is at a high temperature (120 - 130^{O}F), and because of its lack of minerals is not palatable, and is quite aggressive to metals and concrete. Treatment and filtration is therefore essential.

Consumption/Demand
The design average flow rate of 180 GPM is estimated on the following basis:

Office and plant personnel population
Plus construction contractors (peak shutdown period)
to a total of approximately 3000 persons.

Average per capita consumption - 85 Gal/head/day

Average consumer demand $\frac{85 \times 3000}{24 \times 60}$ = 177 GPM (Approximately 180 GMP)

The pumping and distribution system is designed for a peak flow rate of 180 x 1.7 = 300 GPM.

Note: This peak factor of 1.7 is an acceptable international factor.

Distribution System
The Domestic water will be distributed via a ring main system with metered off-takes, these consumer meters will be robust type meters. A survey during the detail design phase have established the grouping of consumer points, and hence the numbers and locations of off-takes, and metering points.

The result will balance investment cost and ease of operational management against the degree of monitoring and control considered appropriate. An initial review provided a cost estimate basis of 30 separate metered off-takes.

PIPE MATERIAL SELECTION
Pumping and Distribution Mains (3" and above)
The following alternatives were considered:

- Asbestos cement.

- Glass reinforced plastic pipework.

- Polyvinyl chloride pipework.

- Galavanized steel.

- Ductile iron (cement lined).

After reviewing the above materials, ductile iron was chosen to be the most suitable material for this service since chemically stabilised FED water will be used.

Elevated Water Tower

The selected elevated water tower performs three functions:

- Balancing supply pump flows against system demand.
- Providing a small quantity of elevated storage in the case of power failure or other system failure.
- Controlling supply pressure. In this instance, the head on the distribution system will always be between 20 and 28 meters (28 - 40 PSIG), with no possibility of pressure surges, which could cause multiple pipe and fitting bursts.

Tower volume at top water level (TWL) = 65,000 USGal
Tower volume at bottom water level (BWL) = 20,000 USGal

INVENTORY STORAGE

During the annual 14 days shutdown of the new desalination plant when only 20 USGPM is available from incremental existing desalination plants capacity, the average demand of 180 USGPM was used to size inventory storage:

$$\frac{(180 - 20) \times 60 \times 24 \times 14}{42} = 76,000 \text{ BBLS)}$$

(This does not consider the peak demand of 300 USGPM).

A design capacity of 80,000 BBLS was chosen, to duplicate the two existing 80,000 BBLS flush evaporated distillate tanks (FED tank).

PROJECT IMPLEMENTATION

The new reheat thermo compression unit will be commissioned before the end of 1994.

The inventory storage tank (80,000 BBL) is now in service and is serving the refinery as a FED in parallel with the other tanks.

It is anticipated that the total project completion date would be in the first half of 1995 serving the combined domestic and drinking water system and after which the abstraction of Zone 'B' water will be stopped.

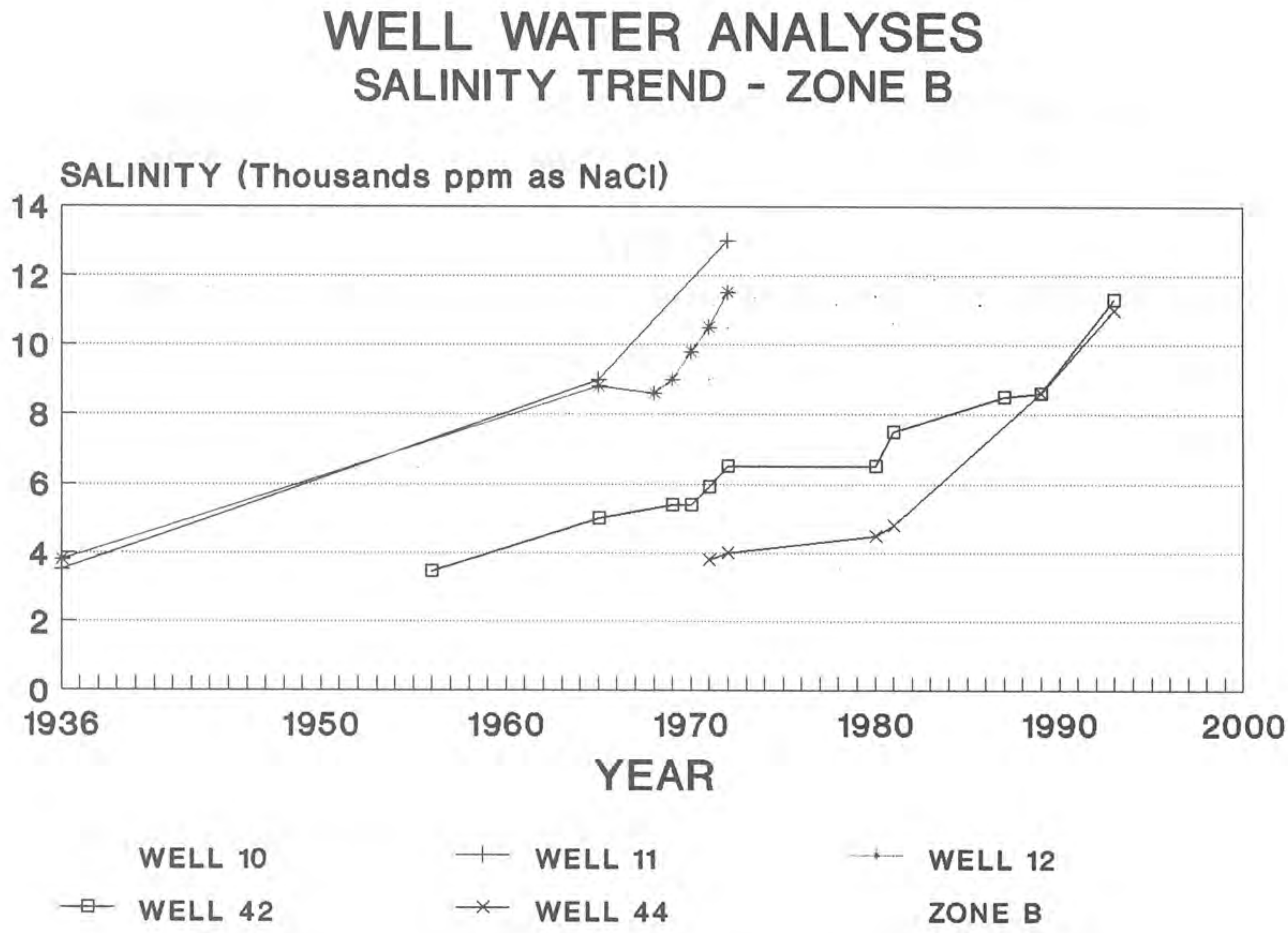
WELL WATER ANALYSES
SALINITY TREND - ZONE B
SALINITY (Thousands ppm as NaCl)
14
12
10
8
6
4
2
0
1936
1950
1960
1970
1980
1990
2000
YEAR
WELL 10
WELL 11
WELL 12
WELL 42
WELL 44
ZONE B

FIG 1

WELL WATER ANALYSES
SALINITY TREND - ZONE B (CONTINUED)

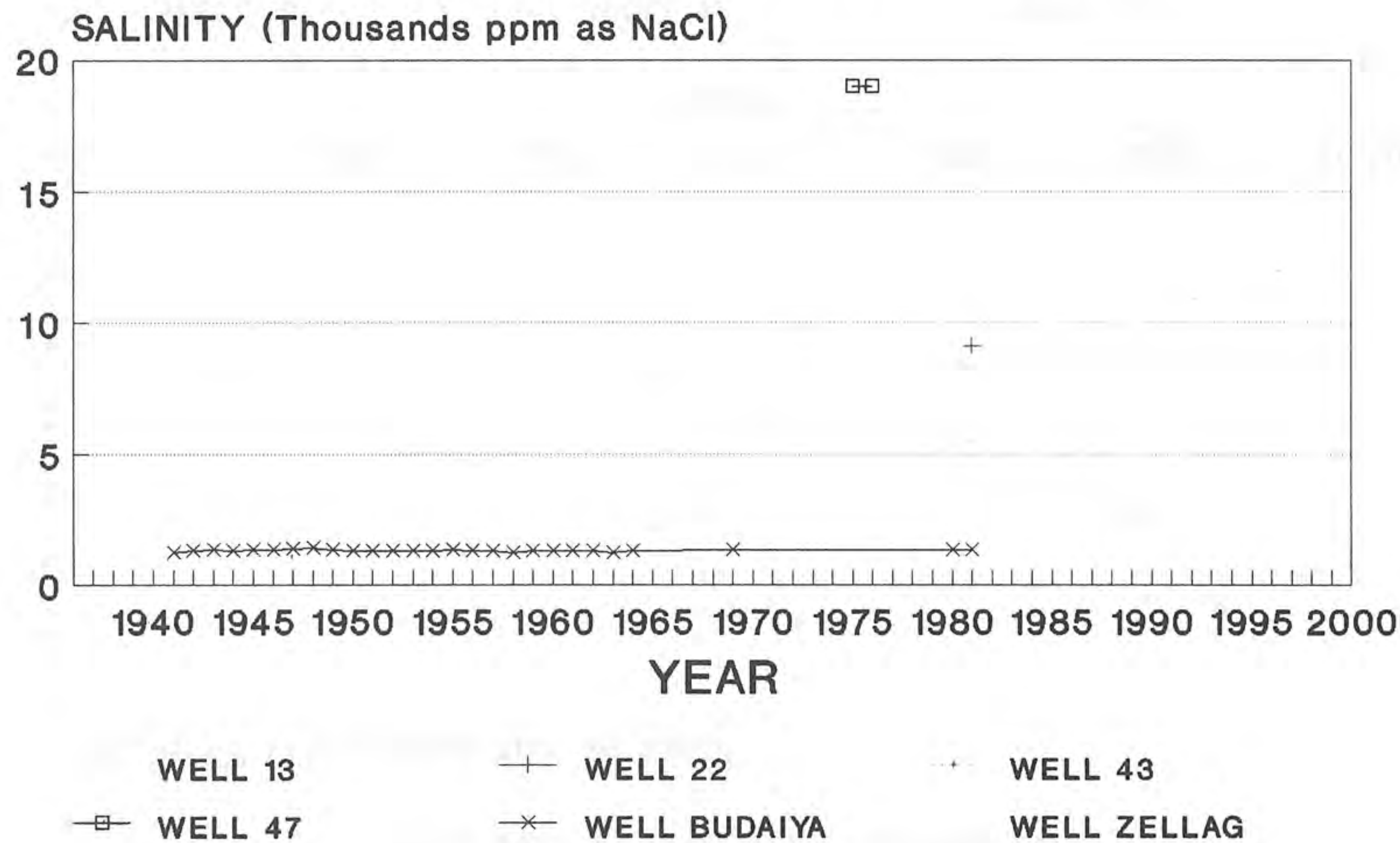

FIG 2

WELL WATER ANALYSES
SALINITY TREND - ZONE C

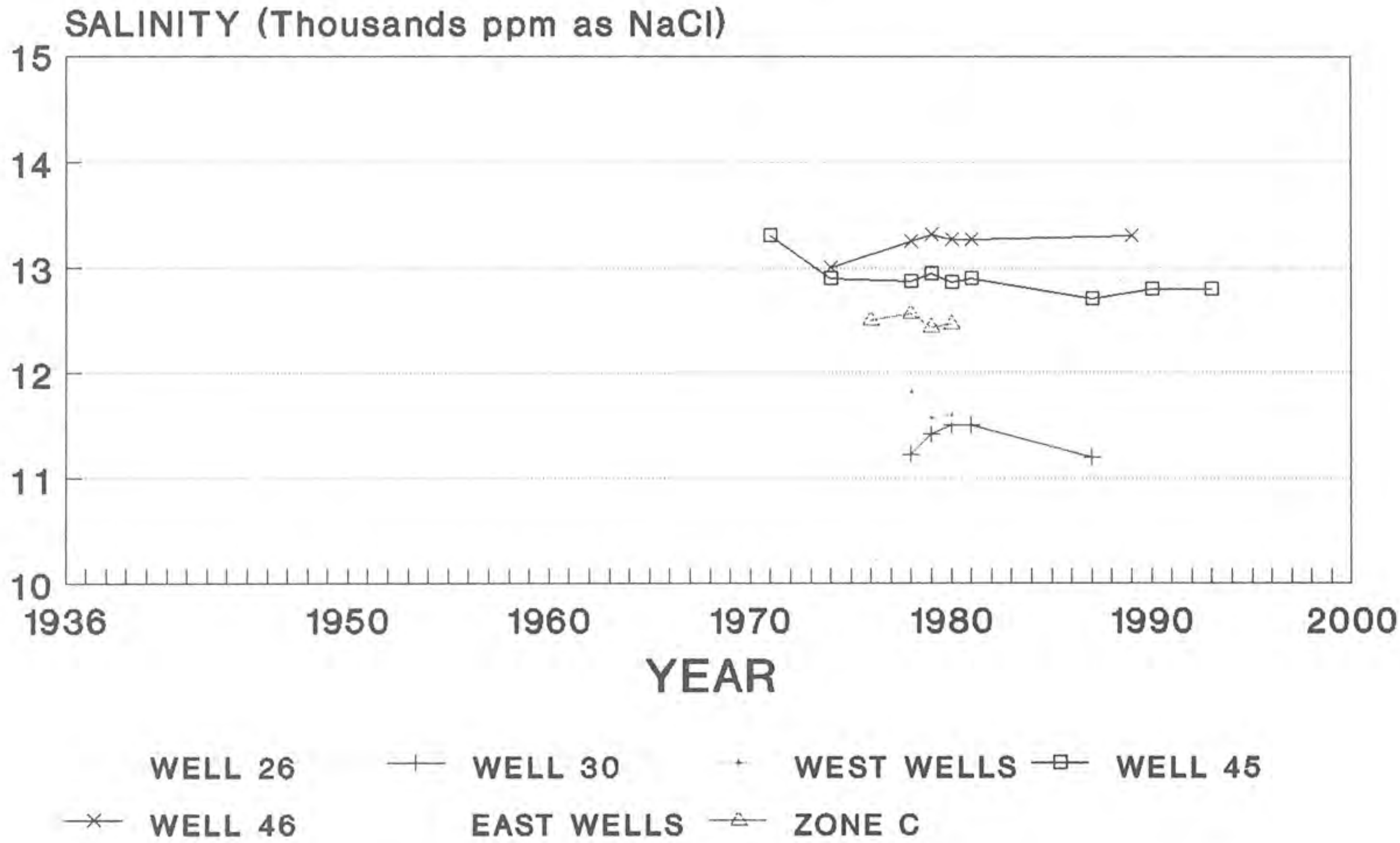

FIG 3

WELL WATER ANALYSES
SALINITY TREND - ZONE B

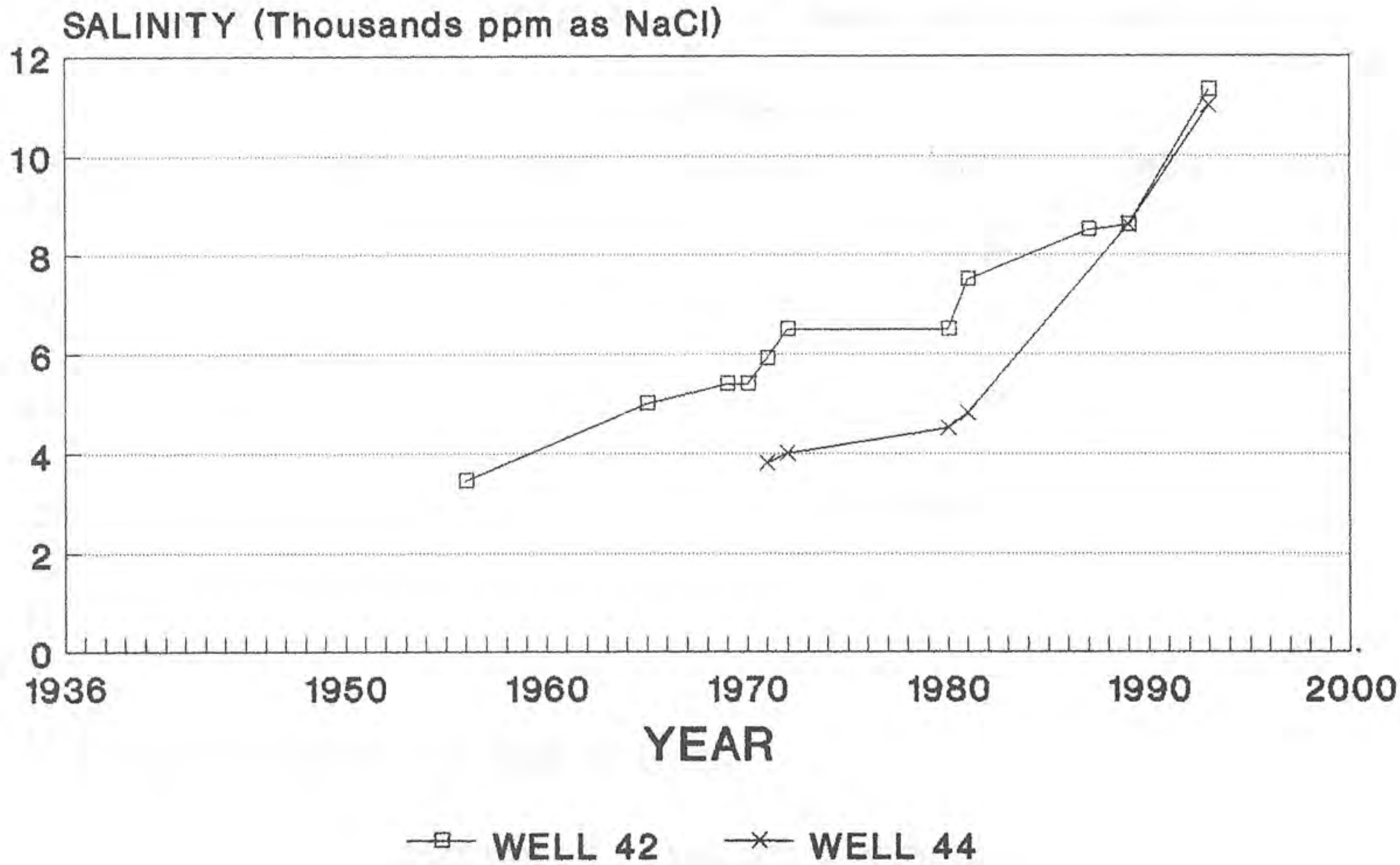

FIG 4

Session - 3
Treatment and Scale Prevention

7 Years Experience of Operating Multi Stage Flash Units at Sitra Power & Water Station

Moh'd Redha Hussain and Badria Al Marzooq

7 YEARS EXPERIENCE OF OPERATING LARGE MULTI STAGE FLASH UNITS AT SITRA POWER & WATER STATION

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ABSTRACT

Sitra Power & Water Station produces electrical power and over 60% of Bahrain's total supply of desalinated water. Of this water production, approximately 75% is supplied from three large distillers on Phase III of the Station's plant.

This paper addresses practical field experiences and the operating problems encountered during the seven years of service of these large M.S.F. units.

Operational syptoms of demister clogging and scale build up in the hot flash chamber stages are reviewed, and the various operating variables are discussed based on extensive operating experience with this group of evaporators.

In particular details are given of various chemical controls and optimisation trials carried out to reduce production costs. Details of scale control additives dose levels are addressed for different top brine temperatures where dose rates were decreased from 6 - 2.5 ppm after the plant warranty period.

INTRODUCTION

Description Of Sitra P/W Station.

Sitra Power and Water Station produces electrical power and supplies more than 60% of Bahrain's total supply of desalinated water. The station is principally run for base load duty and continuous operation at sustained high load factor.

Commissioned in 1975, to meet the Country's growing demand for electrical power and good quality drinking water, the plant originally consisted of four sets of Boilers and Steam Turbo generators, and two multi-stage flash (Units 1A &1B) sea water desalination units with a capacity to produce 100 MW of electrical power and 5 MGPD of desalinated water, at that time the only source of desalinated water supply to the country.

Subsequently in1984/85, and as part of the desalination plant extension plan, three MSF units (Units 2,3 & 4), each with a capacity of 5 MGPD when operated at a Top Brine Temperature (TBT) of 90^{o}C , were commissioned. Each of these units is designed to produce about 6.8 MGPD if operated at TBT of 110^{o}C.

Also in1984, Unit 5 was commissioned that consists of a 25 MW Gas Turbine with a waste Heat Recovery Boiler, an Auxiliary Boiler and a 5 MGPD capacity (MSF) unit. Unit 5 is a self contained unit with its own gas supply system, sea water intake and other auxiliary plant.

Thus the capacity of the installed plant at Sitra Power & Water Station is now 125 MW of electrical power and 25 MGPD of water.

Description Of Large MSF Units (U 2,3 & 4).

The sea water desalination plant extension units 2, 3 and 4 consists of three identical multistage flash evaporators with their associated auxiliary and ancillary systems. An antiscale treatment system is used for treating the brine recirculation water.

The multi-stage flash evaporator consists of 21 stages, 18 of these stages make up the heat recovery section and the remaining 3 being the heat reject section. Table No.1 shows units 2, 3 & 4 materials of construction.

PRACTICAL FIELD EXPERIENCE

In this section ve describe and discuss actions carried out to address the following major issues:

1- Scale phenomena in the flash chambers
2- Demister fouling
3- Stripping pipe salt build-up.

Scale Phenomena In The Flash Chambers.

From the outset of operation, scale was found to cover the hot wall of high temperature stages. Scale build-up was observed after two weeks of operation, but found to be non adherent and exfoliated regularly. Flakes of the scale, however, tended to build-up on brine recircualtion filters leading to high differential pressures.

Table No.2 (Analysis of the Scale) reveals that calcium sulphate is the predominant constituent in stage 1 of inlet brine pipe, magnesium hydroxide dominates on the hot wall of stage 1 - 8, and calcium carbonate in stages 9 21 as the temperature decreases.

Demister Fouling

During annual inspection of these distillers it was observed that the demisters were heavily blocked with hard scale in the hottest stages (the first eight stages) while it reduces from stage 9 onwards.

The demisters of the stages 1-8 were acid cleaned on a yearly basis and the stages 9 - 21 demisters were cleaned as their condition dictated. Refer to photo No.1.

Presently the only available indication of demister fouling is when distillate conductivity shoots up, because there is no on line monitoring instruments of demister fouling.

Remedial Action

So far various efforts have been made to understand the above phenomena and to find a method to reduce demister fouling. It is well known that the demister fouling rate is affected by the following :

1- Flash range
2- Brine concentrations
3- Brine level
4- Stage vacuum
5- Operating temperatures in the higher ranges.

First trial has been made to :

a. Limit the flash range to only 60°C instead of 68.2°C.

b. Maintain concentration ratio of Brine Blow Down within the limits of 1.56.

c. Lowering the brine level by about 100 mm than normal.

d. Adjusting the vacuum is rather difficult in some cases for the reason that there are no individual venting valves, (therefore vacuum could not be adjusted to bring down rate of demister fouling).

Increase of Antifoam Dosing

Antifoam dosing on one of the distillers has been doubled for one year, from 0.1 to 0.2 ppm. No change was found in the conductivity trend as conductivity has gradually increased as demister fouling took place. Fig. No.1 shows conductivity vs times at different top brine temperature.

Stripping Steam Pipe Salt Deposits Build-Up

Since the early stage of operation salt deposits build-up on the stripping steam pipe in the deaerator has been in evidence. This is due to direct contact of sea water and the LP steam having a temperature of 170^{o}C. Analysis of the deposit was as follow :

CaSo4	80%
CaCo3	10%
Mg (OH)2	2%

The stripping steam pipe was modified by the contractor to eliminate the deposit. The trial was to increase the steam velocity to effect self cleaning, but the deposit persisted even though the total quantity was less than before. The new smaller steam outlet was now almost totally blocked. Refer to photo No.2.

CHEMICAL CONTROL EXPERIENCE AND OPTIMIZATION.

Sitra Power & Water Station has gained considerable experience in operating MSF plant and their water treatment since the commissioning of Phase I distillers in the mid 1970s. This experience has been invaluable when selecting and optimising chemical additive regimes for the Phase III plant.

The various chemicals added to sea water, brine recycle and product water are shown in Table No.3.

Scale Inhibitor :

Proprietory chemicals have been selected based on prequalification procedures and a test protocol. These are continuously under review based on their cost effectiveness and improvements offered by various manufacturers. On Phase III plant Belgard and Calgon are in use as scale inhibitors at the time this paper was published. Their use has been developed since the use of Belgard EVN on Phase I distillers in 1980. This has continued on Phase III plant.

During the reliability run polypohosphate at 5.0 ppm at 90°C TBT was used and Belgard EV used above 90°C TBT, this continued to be used until the introduction of Belgard EV 2000 and Calgon EL 4420 in 1988. The dosage rate of Belgard EV as per the manufacturer's recommendation was 6.00 ppm at 110°C TBT.

Since 1988, further trials and optimisation studies were carried out. These are detailed in the following sections.

Belgard EV 2000 Trial & Optimisation (1 Mar 1988 - 9 May 1988)

Optimisation was carried out with the main objective of the trial to :

1- Prove that Belgard EV 2000 was suitable for high temp at 110°C TBT.

2- Reduce the dosing rate from 6.0 - 2.5 ppm Table No.4 show the average parameters results during the trial.

   The test was conducted for 56 days at 110°C at a dose rate of 2.5 ppm together with sponge ball cleaning. The trial results indicate that the chemical can be used successfully at high temp with a dose rate of 2.5 ppm at 110°C.

   The reduction of Belgard EV 2000 dosing from 6.0 ppm to 2.5 ppm was a good step operationally and economically.

   Fig No. 2 shows overall chemical cost before and after optimization.

Calgon EL 4420 Trial & Optimisation (1 Aug 89 - 20 Nov 89)

Following the station financial policy to practically reduce costs, Calgon EL 4420 was tested and optimized. The test was carried for 102 days.

Test results are shown on Table No.5. The distiller operated at max. TBT 110°C with 2.5 ppm Calgon EL 4420 in combination with sponge ball cleaning of 16 cycle/day. The test proved successful.

Antiscalant Test Protocol

It can be generally said that amongst the G.C.C. States, we here in Bahrain are considered the leading state, in formalising the test of antiscalant in our MSF desalination plant, through a stringent prequalification procedure. Protocol, entitled Scale Inhibitor Qualification License Programme, was developed in 1980s and as we gained more experience over the years, its content was accordingly modified, it secures our interest as user and attracts potential reliable manufacturer of these chemicals simultaneously.

Chemical Dosing Control & Plant Performance Monitoring.

Additive chemical dosing control is performed by the chemical engineering section as per manufacturer's recommendation. We have two schemes of dosing with sponge ball cleaning and without sponge ball cleaning (Table No.6 shows dosing rate versus TBT). It is worth mentioning that low density sponge ball cleaning was introduced into our MSF after we noticed, during annual inspection, that the upper portion of the brine heater and heat recovery section was getting more soft sludge than the lower portion.

Plant Performance and Fouling Factor.

A daily routine programmable calculation is being carried out to measure the performance ratio, fouling factor, heat transfer coefficient.

Distiller design parameters at clean and fouled condition is shown in Table No.7.

Our experience indicate that the operating period of the evaporator between two successful acid cleaning is 3 years with on load tabe cleaning system (Fig No.3 shows the trend fouling between two acid cleaning).

TABLE NO. 1

Unit 2, 3 & 4 Materials Of Construction.

ITEM	MATERIAL
Tubes	
stages 1 - 4 and	CuNi 66/30 2 Fe 2 Mn
stages 19 - 21	Bs 2871 CN 108
stages 5 - 18	CuNi 90/10
	BS 2871 CN 102
Tube sheets	Naval Brass ASTM B171 Alloy C46400
Tube support plate	CuNi 90/10 ASTM B402 Alloy C70600
Shell	Fe 510.1 kw UNI 5869 cladding
	CuNi 90/10 ASTM B402 Alloy C70600
Distillarte trays	CuNi 90/10 ASTM B402 Alloy C70600
Distillate channel	CuNi 90/10 ASTM B402 Alloy C70600
Demisters	Monel 400 ASTM B127 UNS N04400
Water boxes	Fe 510.1 kw UNI 5869 cladding
	CuNi 90/10 ASTM B402 C70600
Vent connections	CuNi 90/10 ASTM B466 Alloy C70600

TABLE NO. 2

Analysis Of Scale At Various Locations.

LOCATION	% PORTABLE COMPOSION				
	$Mg(OH_2)$	$CaCo_3$	$CaSo_4$	Na Cl	Moisture *
1st Stage Hot Wall	90.66	0.20	1.08		4.10
1st Stage Cold Wall	87.70	6.64		1.39	4.0
1st Stage Brine Inlet Pipe Wall	5.54	0.64	73.71		19.0
Demister Pad on Stage 4	84.69	4.65	2.90	2.52	2.09
Brine Heater Inside Wall	6.12	53.88	3.56	4.04	22.53 4.33
Deaerators Stripping Steam Pipe	2.04	10.03	80.33		

* Plus other matters

TABLE NO. 3

Chemicals Dosed To MSF Streams

NAME OF CHEMICAL	PURPOSE	POINT OF INJECTION	DOSE RATE PPM
Sodium Hypoclorite	To keep the system free from bacteria and algea (produced by Electro chlorinators)	Inlet sea water	1.00
Sodium Sulphite	Oxygen scavenger	Brine Recycle	1.0 - 2.0
Antifoam	Prevent foaming	Make-up	0.1 - 0.2
Caustic Soda	Ph adjusment to around 8.0 to protect the underground cement line	Product water	
Carbonate	Ph adjustment to around 9.5 to protect the distillate transfer lines	Product water	

TABLE NO.4

RESULTS

The table below shows the average parameter results during the trial of EV 2000

PARAMETER	UNIT	D.VALUE	TRIAL VALUE
Sea water temp	^oC	35.0	33.7
Bottom brine temp.	^oC	41.8	42.3
Brine inlet temp.	^oC	102.8	102.0
Brine outlet temp.	^oC	110.0	109.7
Condensate temp.	^oC	116.0	112.2
Condensate flow	tons/hr	150.4	158.5
LP Steam pressure	bar abs.	2.4	2.38
LP Steam temperature	^oC	146.0	172.5
Shell pressure	bar abs.	2.0	1.4
Total product flow	tons/hr	1305.6	1335.3
Brine recycle flow	tons/hr	12030.0	11970.0
Make up flow	tons/hr	3690.0	3720.0
Blowdown flow	tons/hr	2380.0	2145.0
Antiscalant dosing	PPm	6.1	2.5
Performance ratio	kg/2326KJ	8.89	8.36
Heat transfer coeff	$w/m^2\ ^oC$	4511.0	4081.5
Fouling factor	$m^2\ ^oC/W$	2.0×10^{-4}	1.9×10^{-5}
T.T.D.	^oC	6.0	2.44
Flash range	^oC	68.2	67.5
Concentration ratio (Brine Blow Down)		1.56	1.52

TABLE NO.5

TEST RESULTS.

The Table below lists the average parameter during EL 4420 tests.

PARAMETER	UNIT	DESIGN VALUE	TEST VALUE
Sea water temperature	^oC	35.0	32.5
Bottom brine temperature	^oC	41.8	41.7
Brine inlet temperature	^oC	102.8	100.7
Brine outlet temperature	^oC	110.0	109.0
Condensate temperature	^oC	116.0	113.3
Condensate flow	Tons/hr	150.4	148.7
LP Steam pressure	Bar	2.40	2.49
LP Steam temperature	^oC	146	173.9
Shell pressure	Bar	2.00	1.6
Total product flow	Tons/hr	1305.6	1286.5
Brine recycle flow	Tons/hr	12,030	11,895
Make up flow	Tons/hr	3690	3762
Brine blow down flow	Tons/hr	2380	2280
Antiscalant dosage	PPm	6.1	2.5
Performance ratio	kg/2326KJ	8.89	8.50
Heat transfer coefficient	$W/m^{2-o}C$	4,511	2934.3
Fouling factor	$M^{2-o}C/W$	2×10^{-4}	1.19×10^{-4}
T.T.D.	^oC	6.0	4.3
Flash range	^oC	68.2	67.3
Concentration ratio (Brine Blow Down)		1.56 (max)	1.53

TABLE NO.6

Antiscalant PPM Dose Rate At Different Top Brine Temperature.

TBT (^oC)	90	95	100	105	110
With sponge balls (ppm)	1.00	1.25	1.7	2.0	2.5
Without sponge balls (ppm)	1.40	2.5	3.0	4.0	6.0

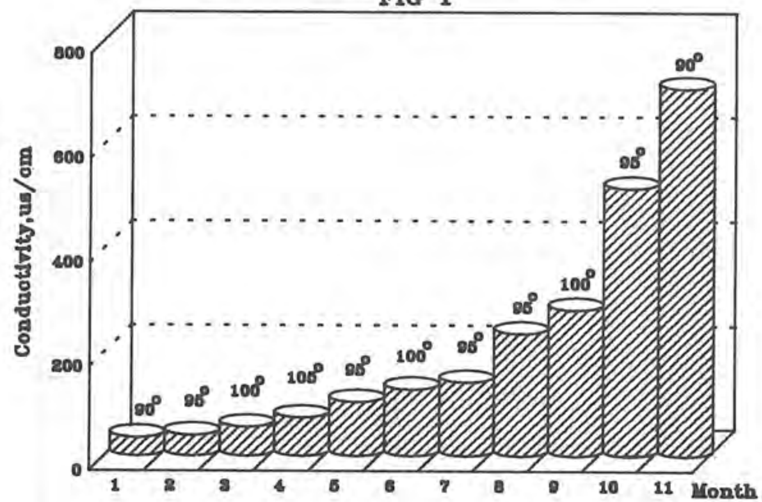
TABLE NO7

Design Clean & Fouled Condition Parameters

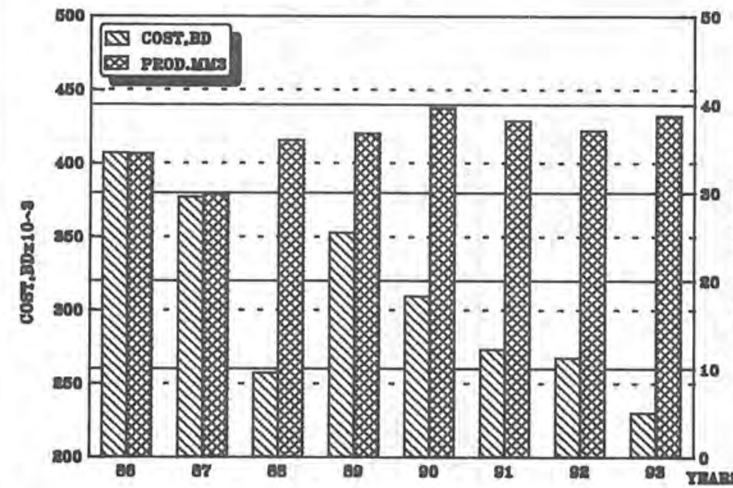
	PR	F.FR	UBH
Clean condition	8.89 KG/2326KJ	0.00001 $M^{2o}C/W$	4511 $W/M^{2o}C$
Fouled condition	8.89 KG/2326KJ	0.0002 $M^{2o}C/W$	2370 $W/M^{2o}C$

SITRA POWER &WATER STATION

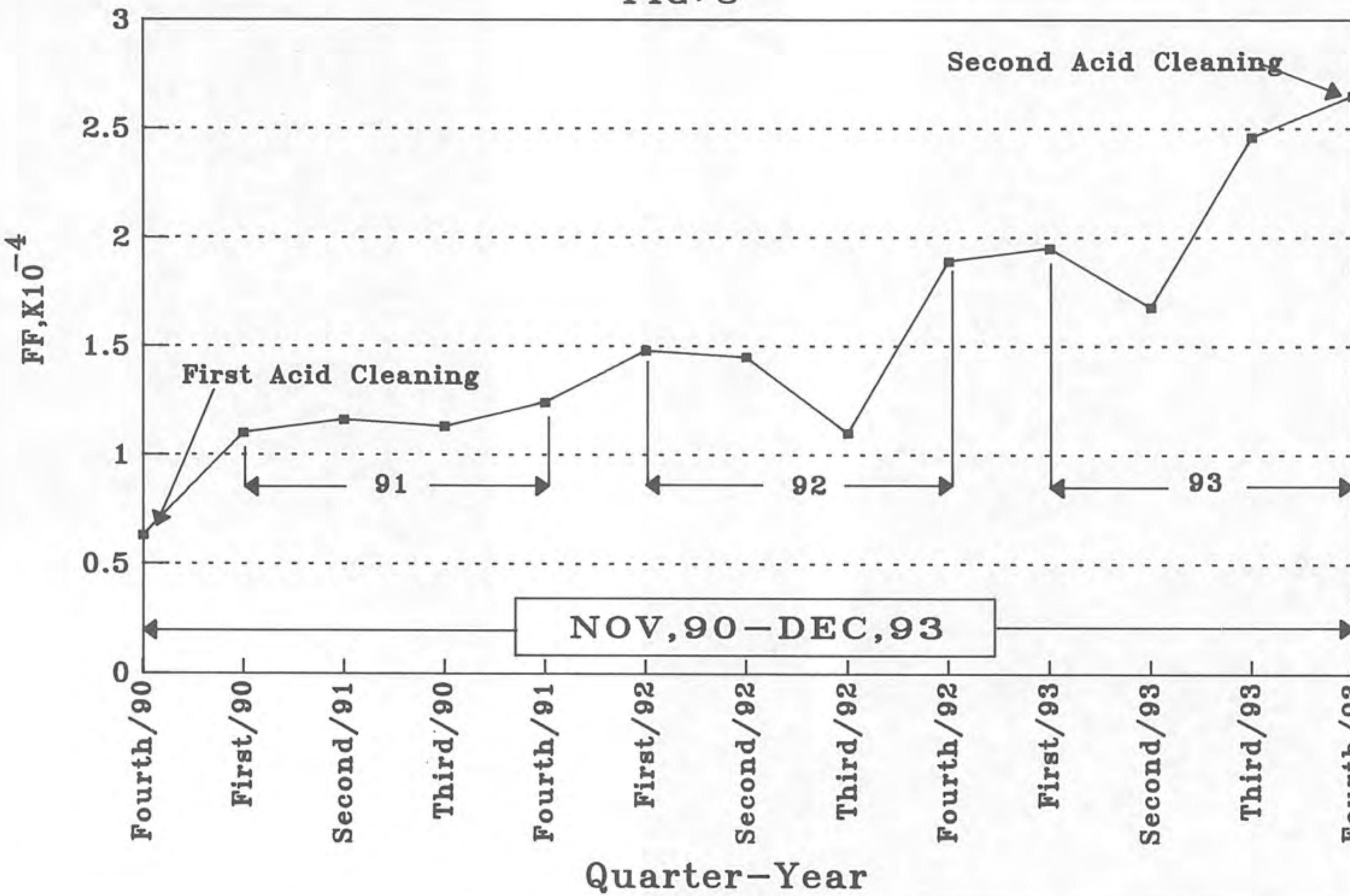
DISTILLER No.2 PRODUCT CONDUCTIVITY vs TIME
PERIOD:92–93
At Different Top Brine Temperatures
FIG 1

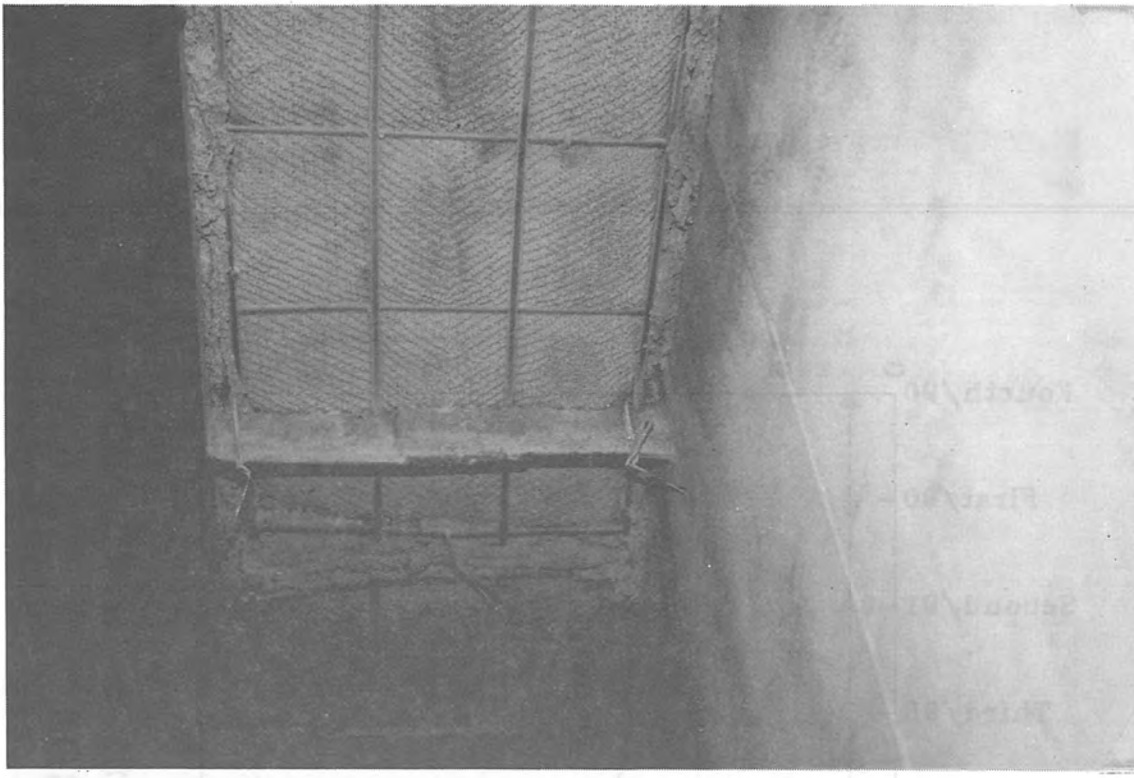


CHEMICAL COST & GROSS WATER PRODUCTION
FROM :1986–1993
FIG 2



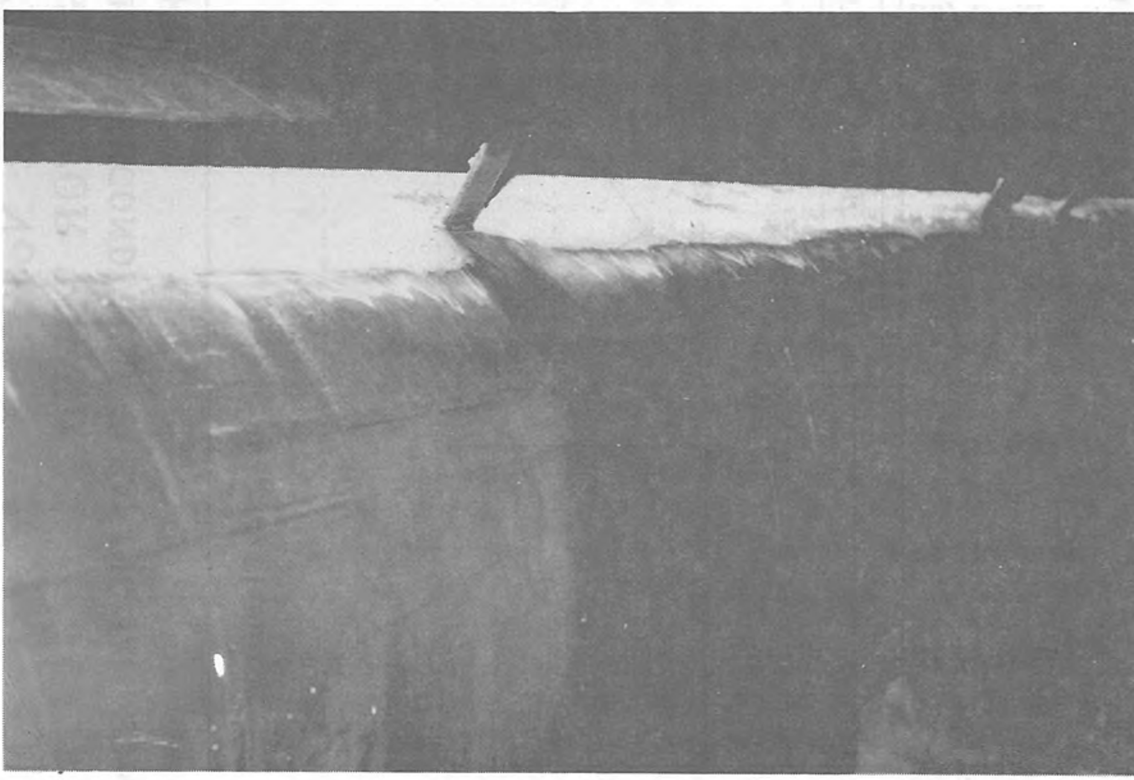
SITRA POWER & WATER STATION
DISTILLER No.2
FOULING FACTOR TREND
FIRST ACID CLEANING: NOV 13th,90-SECOND ACID CLEANING:JAN 23rd,94
FIG. 3





STAGE 1 - DEMISTERS COVERED WITH HARD SCALE

PHOTO No. 1



SCALE ACCUMULATED ON THE L.P. STEAM DISTRIBUTION PIPE LINE

PHOTO No. 2

Water Softening and Using Sand Palette Reactors in Saudi Arabia

Adnan J. Al-Saati and Abdulrahman I. Alabdula'aly

WATER SOFTENING USING SAND PELLET REACTORS IN SAUDI ARABIA

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ABSTRACT - Water softening using pellet reactors or fluidized bed crystallizers (FBCs) is advocated in recent years as a viable alternative to the conventional lime-soda softening process. Pellet reactors produce relatively small quantity of dry and hard pellets (sand grains) that can easily be disposed or reused. Among the plants commissioned in recent years in Saudi Arabia, Unayzah RO water treatment plant (51,000 m^3/d) was designed using pellet softening process. This paper provides information on the pellet reactors process with reference to its performance at Unayzah RO water treatment plant and compares it with the lime-soda softening process adopted at two selected water treatment plants (Al-Wasia and Salbukh) in the Central Region of Saudi Arabia. The actual operational experience of pellet reactors at Unayzah RO plant suggests that further assessment of the process is needed in order to determine its potentials as an attractive alternative to the lime-soda softening process especially as a pre-treatment process in RO water treatment plants.

KEYWORDS - Water softening, pellet reactors, fluidized bed crystallizers, water treatment.

INTRODUCTION

Groundwater is the only source of drinking water in all cities and towns of the Central Region of Saudi Arabia with the exception of Riyadh which meets the major part of its water requirements using desalinated sea water from Al-Jubail plant at the Arabian Gulf [1]. The groundwater supplies from the major aquifers in the region are generally characterized as brackish water. For example, the water from the deep wells that tap the Minjur aquifer around the city of Riyadh contains high total dissolved solids (1,200-1,800 mg/L), total hardness (500-900 mg/L as $CaCO_3$), and sulfate (400-700 mg/L) and the water temperature may exceeds 60°C [1].

Due to the high salinity of the water and in order to meet the drinking water quality requirements, at least a desalting process is required in the design of many treatment plant. Reverse Osmosis (RO) is currently the most widely used membrane process for desalting brackish water supplies in the Central Region. However, the RO process could not function optimally without proper pre-treatment of the raw water to control membrane scale and fouling problems. Evaporative cooling, lime-soda softening and rapid sand filtration are the most widely used pre-treatment processes in many large groundwater RO plants in the region.

Softening is used in water treatment to remove hardness that shortens the life of pipes and heating systems, decreases their efficiency, and increases soap consumption. In RO treatment plants, softening is used as a pre-treatment process to reduce calcium and magnesium hardness and subsequently control membrane scale potentials and increase water recovery. In general, water can be softened by ion exchange processes and by processes based on the dosing of chemicals.

Ion exchange using synthetic resins has become the predominant softening process for industrial purposes. The major disadvantages of using ion exchange in drinking water treatment, however, are the release of relatively large volume of brine, the increase in the sodium content of the water, and the relatively high costs especially for large plants [2].

Softening by chemicals can be divided into precipitation (sludge) processes and crystallization (pellet) processes. The choice between precipitation and crystallization is mainly determined by attributes such as costs, reliability, and operation requirements. Softening by precipitation using lime-soda is the most widely used process in groundwater treatment plants in Saudi Arabia. However, among the plants commissioned in recent years, Unayzah RO water treatment plant was designed using the crystallization process.

OBJECTIVE

The main objective of this paper is to provide information on the crystallization process with the emphasis on its use at Unayzah RO water treatment plant. Comparison of the performance of Unayzah pellet reactors softening with the conventional lime-soda softening used at two selected groundwater RO treatment plants in the Central Region of Saudi Arabia will also be presented.

SOFTENING BY CRYSTALLIZATION

Softening using pellet reactors is relatively a new technology in public water supply. The principle of pellet softening is based on the rapid and complete crystallization of $CaCO_3$ in a fluidized bed of seed grains. The softening process is governed by the fundamental equations concerning chemical equilibrium, crystallization kinetics and mass balance, and the hydraulics of the fluidized bed [3].

Softening crystallizers or fluidized bed reactors typically consist of a cylindrical reactor, partially filled with a suitable seed materials as shown in Figure 1. At the bottom of the reactor, raw water and chemicals are injected through separate nozzles. The nozzle system is designed in such away to ensure even distribution of water and chemicals over the

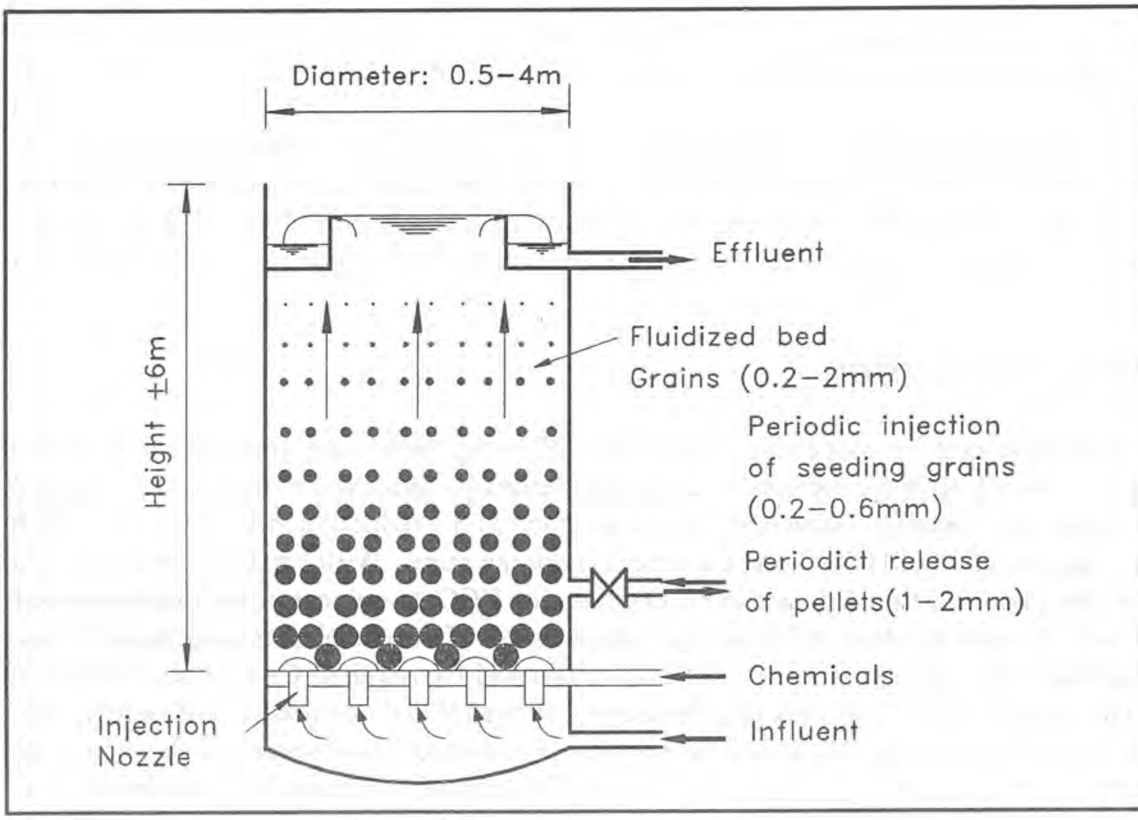

Figure 1. Sand pellet fluidized bed reactor.

base to avoid short circuiting and to obtain plug flow conditions in the reactor. The turbulence caused by the horizontal outflow of water from the nozzles should be high in order to mix the water and chemicals in the presence of seed grains and to prevent scaling of inlet nozzles and reactor wall. The chemical conditions in the reactor are selected in such a way that the solubility product of $CaCO_3$ is exceeded. The fluidized bed provides a large crystallization surface and in a fast reaction the $CaCO_3$ crystallizes on the seed sand. The sand grains thus grow into pellets and the larger and heavier pellets accumulate at the bottom of the reactor where they can be removed periodically and replaced by smaller-diameter seeding sand. Lime, caustic soda, or soda ash is added near the bottom of the reactor to cause the crystallization of $CaCO_3$ onto the fluidized pellets.

PROCESS SELECTION

The choice between a crystallization process and the conventional lime-soda softening process has to be based on a careful appraisal of the two processes for each treatment facility. The crystallization process produces relatively small quantity of dry and hard pellets that can easily be disposed of or even reused for several purposes such as acid wastewater neutralization. Other reported advantages of pellet reactors include lower capital cost, smaller space requirements, relative operating simplicity and low maintenance [4].

The crystallization process has some limitations. Reported experience shows that the application of chemicals especially lime leads to high suspended solids carryover to the filters and subsequently shorter filter runs [4,5]. The process requires an influent water that is not too high in turbidity, phosphate, and iron (III). In the process, only calcium hardness can be removed, whereas the precipitation process allows also for removal of magnesium hardness. However, for magnesium removal the additional chemical demand is relatively high. Therefore, softening is seldom aimed for magnesium removal under normal water quality conditions.

CHEMICALS SELECTION

Softening can be induced by any of the following chemicals: lime, caustic soda and soda ash. With $Ca(OH)_2$, the softening reaction leads to a reduction in the HCO_3^- -content of the water (in mmole/l), which is twice as much as the reduction in Ca-hardness. With NaOH, the reduction of HCO_3^- and Ca-hardness are the same. With Na_2CO_3, the softening reaction does not lead to a decrease in HCO_3^-. As the HCO_3^- -content of the drinking water should not become too low, it follows that the selection of the chemical is determined by the ratio between the HCO_3^- -content of the raw water and the required Ca-hardness reduction [5]. This implies that lime can only be used when the HCO_3^- -content is sufficiently high. Caustic soda can be used if the HCO_3^- -content is moderate, whereas soda ash can also be used with a very low HCO_3^- -content. It was reported that a minimum of 2 mmole/l HCO_3^- -content of the drinking water has been adopted by the Dutch waterworks in view of the pH stability in the distribution network and corrosion effects [5].

The selection of the proper chemical(s) to be used is determined by the raw water quality, the ease of operation and the cost of chemicals. With respect to ease of operation, preference should be given to caustic soda because it is commercially available at a 50% liquid that can be handled easily, whereas, lime requires tedious operational efforts in handling, feeding and scale prevention. The cost of chemical and dosing equipment depends on the size of the plant. The use of caustic soda is cheaper for small plants (capacity up to 1000 m^3/h), whereas for large plants, application of lime is more cost effective [3].

UNAYZAH RO PLANT

Unayzah RO Plant was designed for a maximum production capacity of 51,000 m^3/d. Figure 2 shows the process flow diagram of the plant. Raw water from ten deep wells (8 running and 2 standby) is pumped to the plant through two feeding lines (Line 1&2) to six pellet softening reactors. The softened water is chemically conditioned using H_2SO_4 before pumping to eight gravity sand filters (6 running and 2 standby). The softened and filtered water is then pumped to a separate building which houses the RO desalting plant. The RO process was designed for a maximum recovery of about 90% and the generated brine is discharged into four evaporation ponds. In order to minimize the size of the evaporation ponds and to maximize the utility of the water, a brine recovery unit (a vapor compression distillation unit) was also constructed in order to increase the total recovery from the RO process to about 99%. The plant was designed with a raw water blend lines (25% of feed water) which are directed to four gravity sand filters (2 running and 2 standby) and ultimately

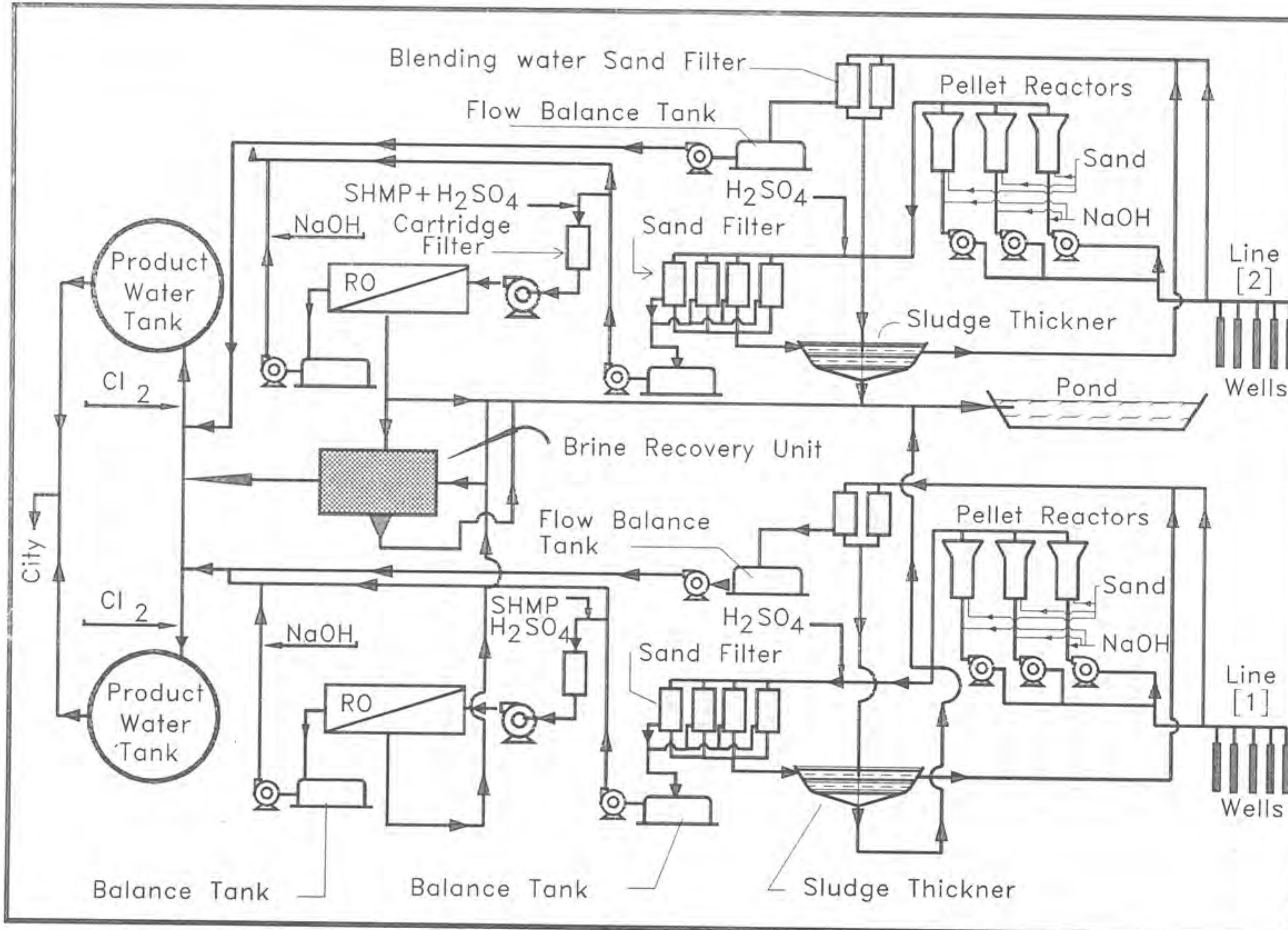

Figure 2. Process Schematic of Unayzah RO Plant.

mixed with the desalted water in order to meet the water quality requirements. The configuration of the unit processes used in the plant guarantees a production of 85% of the design treatment capacity during failure periods of any one unit. In addition, several by-pass facilities per treatment unit or process is provided to ensure the highest operational flexibility [6].

PELLET REACTORS

Limitation of groundwater supply and the economics of water desalting require achieving a high water recovery from the RO process. The pellet reactor process was selected as the most economical and appropriate softening process for the removal of calcium in order to increase RO water recovery to about 90% under the prevailing raw water quality conditions in the Central Region. The softening process adopted at Unayzah plant is based on the $CaCO_3$ crystallization on a seeding material with caustic soda and soda ash solutions injected with the raw water to the conical bottom of the reactors. The high rate of crystallization allows a very short retention time, so that the volume of the reactor can be small. Unayzah pellet reactors were designed for surface loading of 100 m/h and with a diameter of 1.85 m and a height of 6 m. During operation the pellets in the reactor increase in size from the original sand diameter of 0.4 mm to an average of 1.0 mm. The pellets are discharged intermittently from the reactors during operation and replaced by fresh seed grains at a current rate of 40 Kg/d per reactor.

The pellet softening at Unayzah has a very high process efficiency at retention times of 2.4 minutes compared to 1.2 and 2.28 hrs in the turbo-circulator softening systems at Riyadh's Salbukh and Al-Wasia water treatment plants respectively. Table 1 compares the three plants with respect to: number of units used, volume and hydraulic design capacity of each unit, retention times, and the dosing rates of used softening chemicals. Table 2 provides information on the raw water and softened and filtered water with respect to changes in pH, calcium hardness, magnesium hardness, total hardness, and bicarbonate alkalinity at the three plants. It is important to recognize that the performance of the softening process at each plant is dictated by its treatment objectives.

Al-Wasia treatment plant was designed as only a softening plant with its product to be blended with desalinated sea water from Al-Jubail plant before being fed to the water distribution network of the city of Riyadh. An average total hardness reduction of about 13.5% was only required during 1991-1993 and was achieved using both lime and soda ash.

Salbukh treatment plant is a typical of six RO water treatment plants forming the grounwater desalination complex that supplies Riyadh with potable water. The remaining plants are Buwaib, Manfouha I, Manfouha II, Malez, and Shemessy [7]. An average total hardness reduction of about 49% was achieved at Buwaib during 1992-93 using higher dosing rates of both lime and soda ash.

The total hardness reduction at Unayzah plant (Line 1) was about 34% during June 1993 and was achieved using only caustic soda. Further discussion on the performance of the pellet reactors is provided in the following section.

Table 1.

Characterization of adopted softening systems at selected RO plants in the Central Region of Saudi Arabia.

	Al-Wasia	**Salbukh**	**Unayzah**
Softening Process	Precipitation (turbo flocculator)	Precipitation (turbo flocculator)	Chrystallization (Sand Pellet)
Number of Units	8	6	6
Volume of Unit (m^3)	2,940	600	16.1
Design Capacity (m^3/d)	31,000	14,400	6,480
Residence Time	2.28 hr	1.2 hr	2.4 min
- Lime Dosing	100 mg/L	120-130 mg/L	--------------
- Caustic Soda Dosing	--------------	--------------	100 mg/L*
- Soda Ash Dosing	60-80 mg/L	200-220 mg/L	117 mg/L*
- Sand Dosing	--------------	--------------	40 Kg/day

* : Given design dosing rate. Only Caustic Soda is used in actual operation of the plant.

Table 2.

Performance of the softening process at selected RO plants in the Central Region of Saudi Arabia.

	Al-Wasia *			**Salbukh ***			**Unayzah ****		
	Raw Water	Softened & filtered Water	Reduc-tion, %	Raw Water	Softened & filtered Water	Reduc-tion, %	Raw Water	Softened & filtered water	Reduc-tion, %
pH	8.24	7.76	5.8	8.02	6.65	17.2	7.62	7.29	7.3
Calcium, mg/L	148.0	121.8	17.7	172.2	56.6	67.1	84.0	56.0	33.3
Magnesium, mg/L	42.6	39.3	7.7	60.8	48.7	19.9	29.0	5.0	82.8
Total Hardness as $CaCO_3$, mg/L	547.0	473.0	13.5	671.0	342.5	49.0	240.0	110.0	33.8
Alkalinity, mg/L	144.0	70.8	50.8	178.1	31.7	82.2	120.0	108.0	10.0

* : Average for the year 1413 (1992-1993) [After El- Rehaili et al., 1993].

**: Collected samples from Line -1 on 17/06/1993.

PROCESS PERFORMANCE

The design of Unayzah water treatment plant is based on raw water quality which is considerably worse than the actual quality. Softening chemicals (caustic soda and soda ash) and their dosing rates (as given in Table 1) that were planned to be used, had to be changed. Only caustic soda was initially used and then replaced by lime. However, due to the operational problems associated with the use of lime including the increased carryover from the reactors to the sand filters, the dosing of only caustic soda was resumed since 1993. Figures 3a and 3b show the performance of Lines 1&2 pellet reactors in terms of total hardness removal for the period from March 1991 to June 1993, respectively. During the period of lime dosing the total hardness reduction was maintained at an average value of 35% in both lines (3 reactors per line). With the use of caustic soda, the total hardness reduction in Lines 1&2 was about 14% and 43% respectively. This difference is attributed to operational schedule of the six pellet reactors and their chemical dosing rates. The three reactors of Line 2 are more frequently used to remove the calcium hardness of the raw water before feeding it to the RO process, whereas, the other three reactors of Line 1 are less frequently used to achieve this objective. This situation was created by the fact that since the total dissolved solids (TDS) of the raw water is about 675 mg/L, it is only required to reduce the overall TDS of the softened water to nearly 600 mg/L and to produce only little RO product (22%) for blending with softened and filtered water (78%) and with filtered raw water, to lower the overall product water salinity to around 500 mg/L [8].

COST OF PELLET SOFTENING

Water softening using pellet reactors was selected as the most economical and appropriate process for the removal of calcium at Unayzah RO plant. It was estimated based on the mean values of the lower five bidders for this plant that both capital and O&M costs will be lower for the pellet reactors process than the lime-soda softening process. Aside from the chemical costs, the costs for the crystallization process was estimated at some SR 0.2-0.3 per m^3, whereas the costs for the lime-soda softening process are estimated at some SR 0.3-0.4 per m^3 [9].

SUMMARY AND CONCLUSIONS

Pellet reactor softening is advocated as an example of a clean technology. The principle of pellet softening is based on the rapid and complete crystallization of $CaCO_3$ in a fluidized bed of seed grains. The process produces no sludge but relatively small quantity of dry and hard pellet that can easily be disposed of or reused in some industrial applications.

The pellet softening process at Unayzah RO treatment plant is the first system to be implemented for water treatment application in Saudi Arabia. The process was selected as the most economical and appropriate softening process for the removal of calcium hardness. Performance evaluation of the process indicates that it has been successfully used in Unayzah RO water treatment plant. However, the carryover problem from the pellet reactors and its impact on shortening filter runs requires additional evaluation. Further research on the performance of the process under various modes of operation, dosing chemicals and different

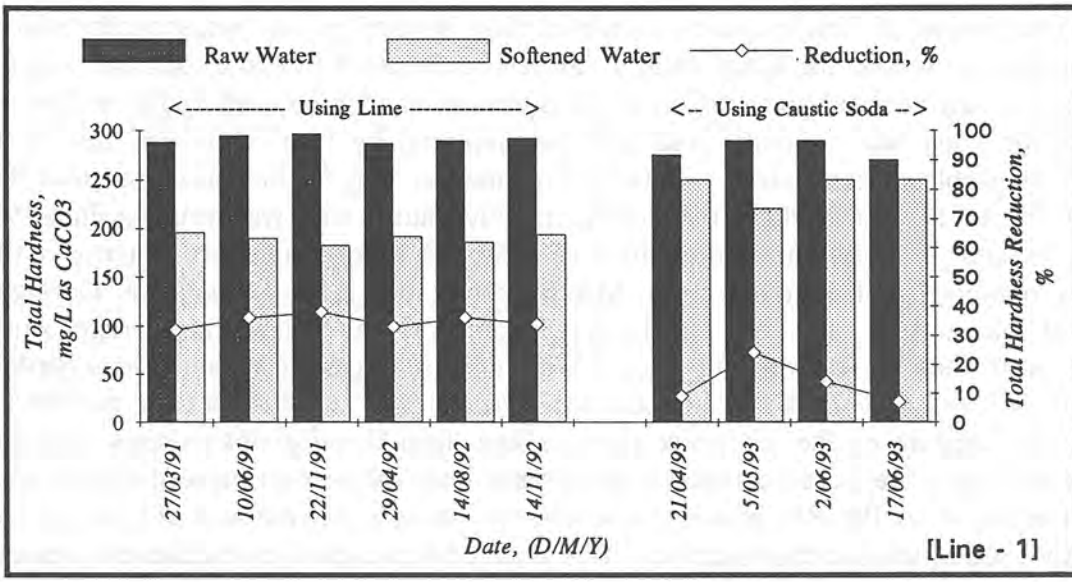

Figure 3a. Performance of sand pellet reactors at Unayzah RO plant (Line-1).

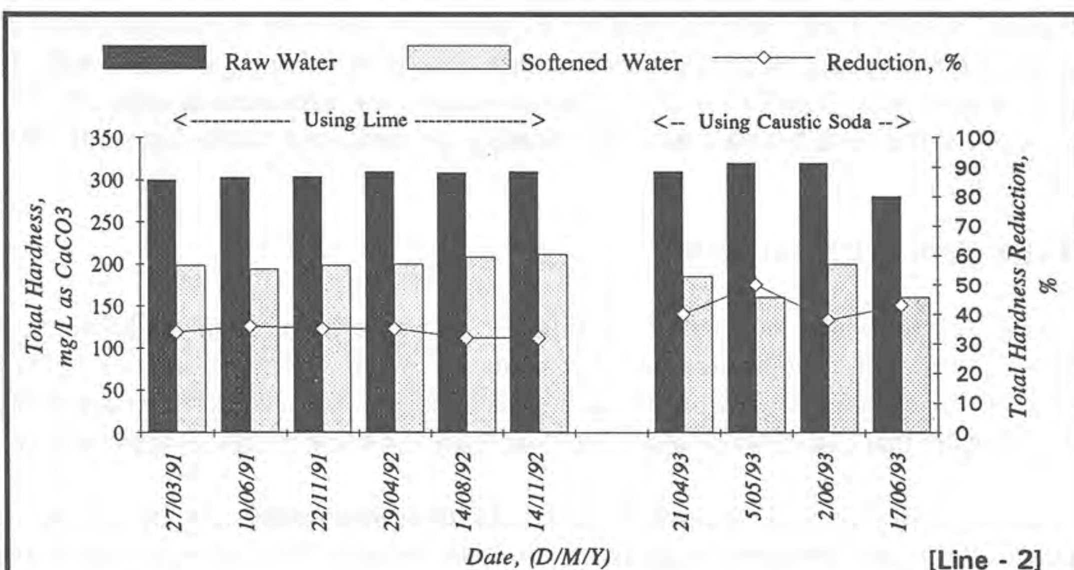

Figure 3b. Performance of sand pellet reactors at Unayzah RO plant (Line-2).

water qualities is needed for the situation of Saudi Arabia. It is expected that the available information on the performance of the pellet reactors at Unayzah RO water treatment plant and future experimental work on crystallization softening will provide needed insights on the potentials of adopting the process as an attractive alternative to the lime-soda softening process in future groundwater treatment plants.

ACKNOWLEDGMENT

The authors would like to thank King Abdulaziz City for Science and Technology for the financial support of this study.

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Laboratory Testing of AntiScalant Threshhold Effectiveness

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Yoshihiro Tanaka, Saad Abdullah Al-Sulami

LABORATORY TESTING OF ANTISCALANT THRESHOLD EFFECTIVENESS

BY

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ABSTRACT

This paper reports laboratory test results carried out to study antiscalant threshold effectiveness. The primary objective of these tests were to establish some reference points to be used as a base for initial evaluation and selection of antiscalants for further testing in a single heat exchanger tube testing. These results are intended to be used for evaluation and selection of antiscalants for MSF Pilot unit.

These bench top tests indicate Magnesium hydroxide precipitation is not time dependent unlike calcium carbonate in the presence of antiscalant in brine solution at pH of 8-9. They also indicate that most antiscalants are effective for up to 20 minutes at 95°C with up to 2 parts per million concentration. While at higher temperature of 110°C this effectiveness dropped down to less than 10 minutes. Furthermore, no major difference was noted on effectiveness at 95°C for a range of antiscalant concentration of 1-2 ppm.

INTRODUCTION

In seawater desalination predominant scales are of calcium combining with either carbonate or sulfate, and magnesium combining with hydroxide. Calcium forms hard scale when it combines with sulfate, but this reaction is of appreciable magnitude only at temperatures of 121°C or more. Calcium sulfate scale can therefore be avoided if operating temperature is maintained below the said value (< 121°C). On the other hand calcium and magnesium form softer scales with carbonate and hydroxide (respectively) at lower temperatures and brine concentration of 65000 ppm (or less). In distillation process where temperatures are maintained below 121°C, $CaCO_3$ and $Mg(OH)_2$ forma-

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tions are controlled by either chelation or depletion. In depletion process carbonate is decomposed down to CO_2 through reaction with acid e.g. H_2SO_4 or HCl. In this process, seawater make up pH drops down to 7.8 or less after CO_2 removal in the decarbonator. Multi Stage Flash (MSF) evaporators which are in operation in Gulf Cooperation Council (GCC) states are being mostly operated with additive treatment at top brine temperatures well below the said value (TBT < 121^0C). Further limiting factors of TBT are the capabilities of available additives [1].

EXPERIMENTAL SETUP AND PROCEDURE

1. Reagents Preparations

(a) *Sodium carbonate solution:*

53 grams of Na_2CO_3 powder was weighed and dissolved in 1L of deionized water to prepare IN solution.

(b) *Stock Solutions of antiscalants:*

0.5 gram of each antiscalants were weighed accurately and dissolved in 500 ml of deionized water in order to give 1000 ppm stock solutions.

(c) *Artificial Brine:*

(i) Chemicals used for preparation & mixing ratio:

- Chemical A: NaCl
- Chemical B: INSTANT OCEAN (a registered commercial name of mixture of salts)
- Mixing ratio should be adjusted based on the following ratio: NaCl/INSTANT OCEAN = 1.375

(ii) In order to prepare artificial brine with a concentration equals 1.4 times that of seawater, approximately 8.8 grams of NaCl and 6.4 grams of INSTANT OCEAN were dissolved in un.iltered seawater. M-Alkalinity of final solution was adjusted to 180 ppm by dissolving more salts or by diluting with seawater.

2. Instruments For Analysis & Measurements

(a) pH meter (Fisher brand Model 825 MP) equipped with glass reference electrode to measure pH of solutions.

(b) Automatic Titrator (Fisher brand Model 465) to measure M-Alkalinity.

(c) Atomic Absorption Spectrometer (Varian Model AA-975) for Ca and Mg analysis.

3. Experimental Apparatus

(a) *Low temperatre experimental apparatus:*

As shown in Figure 1, the apparatus consists of a three neck flask (1L capacity) equipped with a condenser and a thermometer. Heating mantle equipped with a stirrer was used to heat up the brine solution, and a vacuum pump was used in order to create vacuum above the brine surface for flashing to take place.

(b) *High temperature apparatus:*

As shown in Figure 2, the apparatus consists of a reaction vessel that can withstand high pressure and temperatures. The vessel is equipped with a heating coil and a stirrer. Temperature and stirring speed can be controlled from a control unit.

4. Test Conditions

(a) Test temperatures	:	95 & 110^{o}C
(b) Brine M-Alkalinity (M-Alk)	:	180 ppm
(c) Concentration of antiscalant	:	2 ppm (or less)
(d) Retention times	:	5, 10, 15, 20, 30 & 40 minutes

5. Test Procedure

500 ml of artificial brine was measured accurately and spiked with 1 ml of antiscalant stock solution in order to make 2 ppm of antiscalant in the brine final solu-

tion. The brine was charged to the reaction vessel or flask and it was heated to the desired temperature under refluxing. When the solution reached the required temperature, it was kept refluxing for 15 min to reach equilibrium and to observe any possibility of scale formation. Because there was no precipitation in the presence of antiscalant, 10 ml of lN Na_2CO_3 was added to break the supersaturation point and to form the scale. The moment at which Na_2CO_3 was added was considered to be T = 0. Successive samples (25 ml each) were drown from solution at time intervals of 5, 10, 15, 20, 30 and 40 minutes. The samples were filtered through 0.45 micron filter paper and filtrate was analysed for Ca, Mg and M-Alk.

RESULTS AND DISCUSSION

Scale formation behavior was studied in laboratory at temperatures similar to actual TBT's (95 & 110°C). Figure 3 shows results of tests carried out on calcium and magnesium presence in solution as a function of change in solution M-Alkalinity at 110°C. This Figure clearly demonstrates the dependence of Calcium concentration on M-Alkalinity in brine solution unlike Magnesium. These phenomena are further verified by the results of additional testing of magnesium and calcium ratios versus time in a brine solution heated up to 110°C. Figure 4 & 5 show changes in magnesium and calcium concentration ratios denoted by (f) for final over initial (i) as function of time at 110°C in the presence of 2 parts per million of different antiscalants in a brine solution having salts concentration of 1.4 times gulf seawater (GSW TDS <61000 ppm). After having verified the fact that $CaCO_3$ scale formation is time dependent further laboratory tests were carried out. To study this time dependance, at 95 and 110°C in the presence of different antiscalants in brine solution (1.4xGSW). Figures 6 & 7 show laboratory test results for different antiscalants at 2ppm concentration and at 95 & 110°C respectively. In addition, brine samples at the above two temperatures were also tested in the absence of antiscalant for $CaCO_3$ measurement against time. These points are denoted by a solid squares and referred to by the letter (B) for blank in Figures 6 & 7. It can be seen from Figure 6 that at 95°C most antiscalants have almost the same threshold effect for up to 15 minutes. Thereafter, they start to show some variations in their effectiveness between 15-30 minutes at 95°C. However, results at 110°C indicate this variation in effectiveness much faster as can be seen in Figure 7. Figure 7 also shows that loss of effectiveness of almost all antiscalants start after only 10 minutes at 110°C compared to 30 minutes at 95°C which can be seen from Figure 6. Figures 8 to 11 show results of antiscalant concentration optimization laboratory tests on the most outstanding antiscalant of those tested (at 95°C) and reported in Figures 3 to 7.

The first pair (Figures 8 & 9) are on a totally synthetic brine of 1.4xGSW TDS. These two Figures show that effectiveness is almost proportionally related to antiscalant concentration. Yet when brine solution was prepared by salt addition to filtered GSW there was a change in behavior which can be seen in Figures 10 & 11 where there is an indication of an optimum dose rate specially as for longer elapsed time of retentions.

Further experimental laboratory and mini module of single tube heat exchanger testing are still underway. Nevertheless the above results could be used to explain some of the earlier reported observations on actual operating plants. In particular the existence of an optimum dose rate plus observed slugging and scaling in flash chambers and demisters [2 & 3].

CONCLUSION

Laboratory experimental results shown in Figures 3-11 can help in making the following observations and recommendations.

1. Magnesium hydroxide separation is pH dependent and time independent.

2. Calcium carbonate separation is time, temperature and antiscalant concentration dependent.

3. At antiscalant dose rates of up to 2 ppm there are quite few brand name antiscalants which are effective for as long as 20 minutes at 95°C and brine concentration of 1.4 GSW TDS.

4. At antiscalant dose rate of 2 ppm there appears to be only few which could be considered to be effective at 110°C and brine concentration of 1.4 GSW TDS.

5. It seems that to maintain an acceptable threshold effect at 110°C either brine concentration needs to be lowered drastically, antiscalant dose rate optimized, or a second antiscalant injection point near the brine heater needs to be considered so that effectiveness time is further elongated beyond the first 10 minutes.

6. To verify these results further tests are required on mini module, (of single tube heat exchanger) pilot units and most importantly actual trial tests on large plants specially where total residence time is 15 minutes or more (up to 30-40).

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Figure 1 : Low Temperature Test Equipment

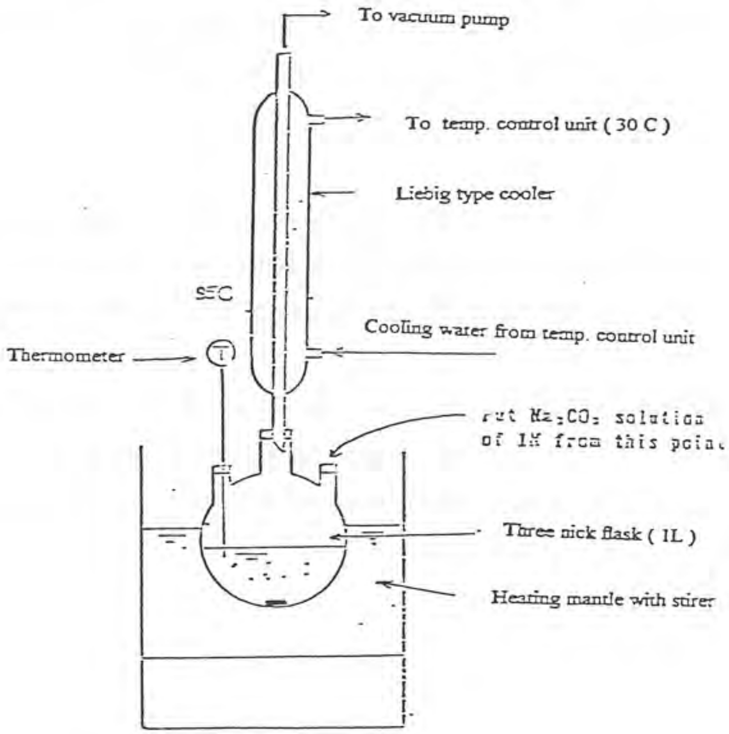
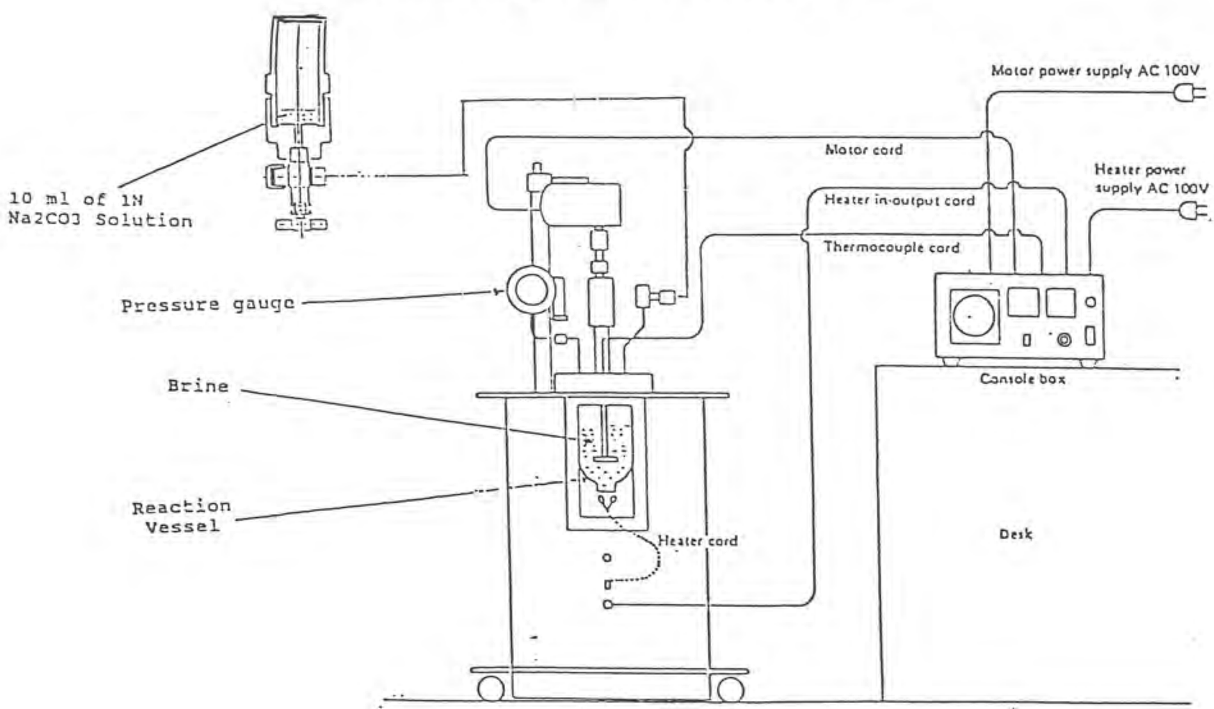


Figure 2 : High Temperature Test Equipment



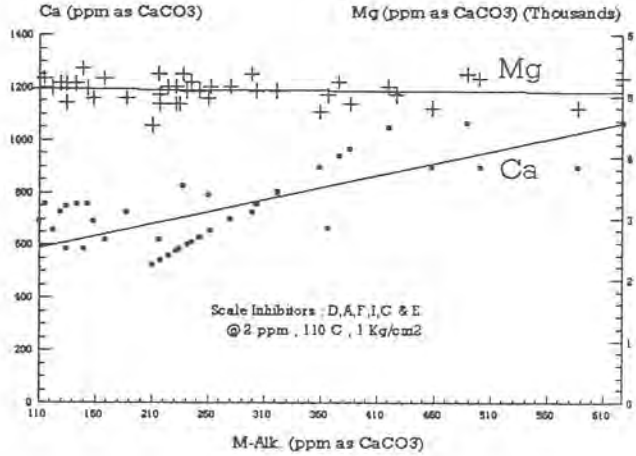


Figure 3 : Relationship Between M-Alkalinity and Ca & Mg Concentrations

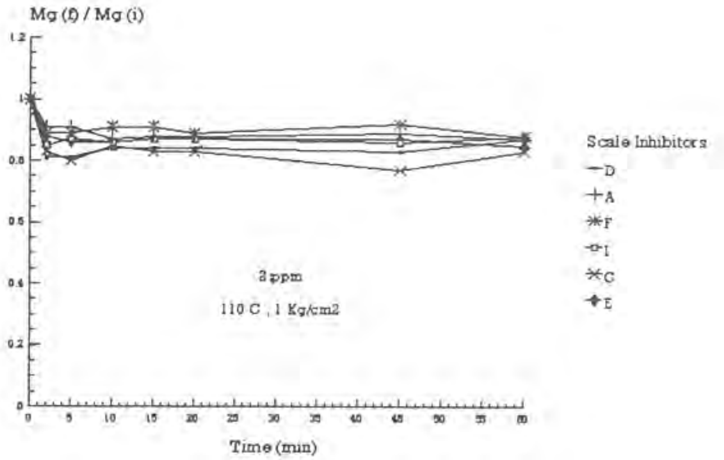


Figure 4 : Changes of Threshold Effect Mg(f)/Mg(i) With Retintion Time

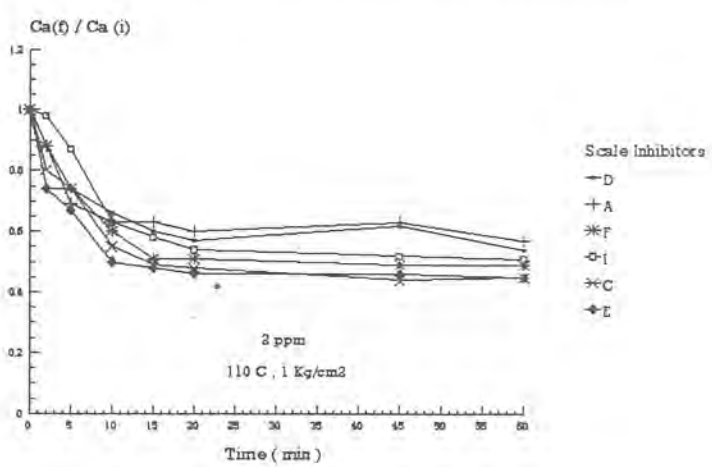


Figure 5 : Changes of Threshold Effect Ca(f)/Ca(i) With Retintion Time

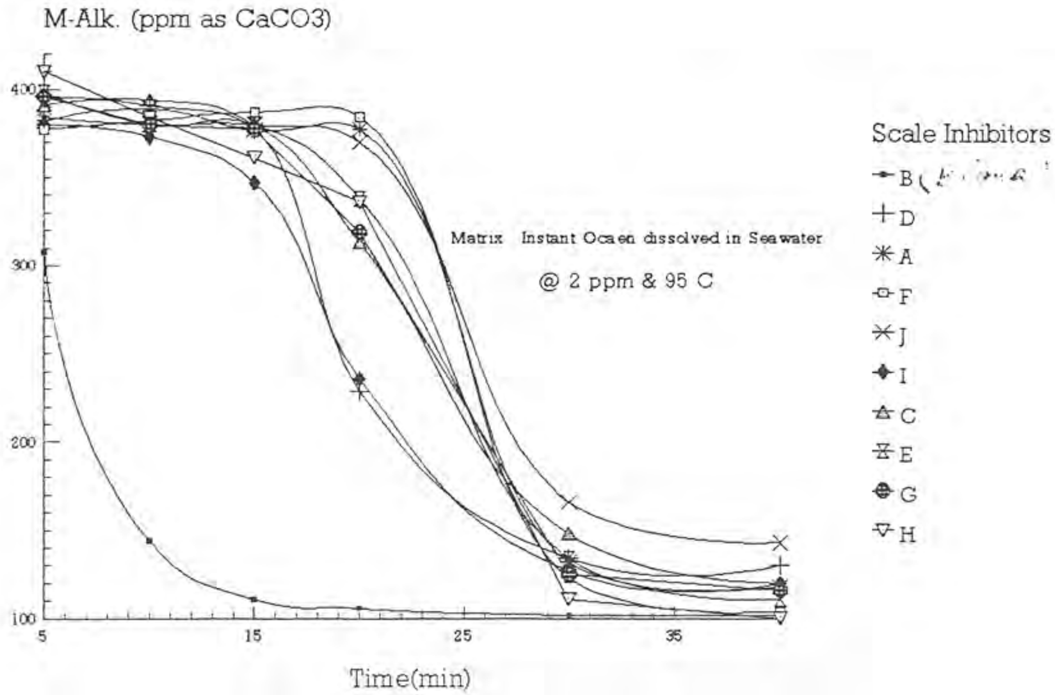


Figure 6 : Threshold Effect of Scale Inhibitors

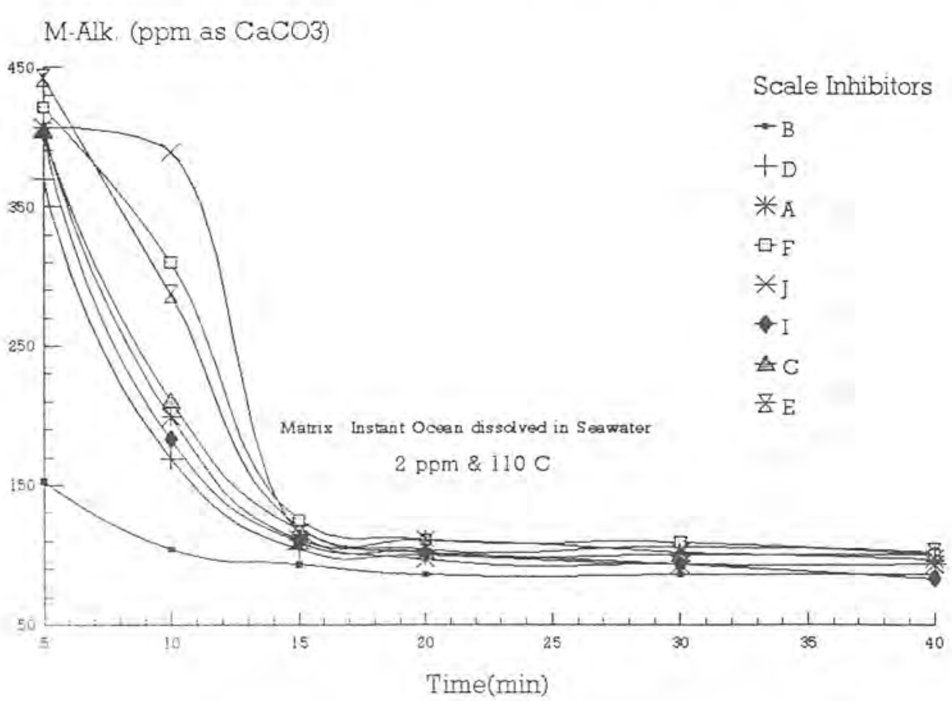


Figure 7 : Threshold Effect of Scale Inhibitors

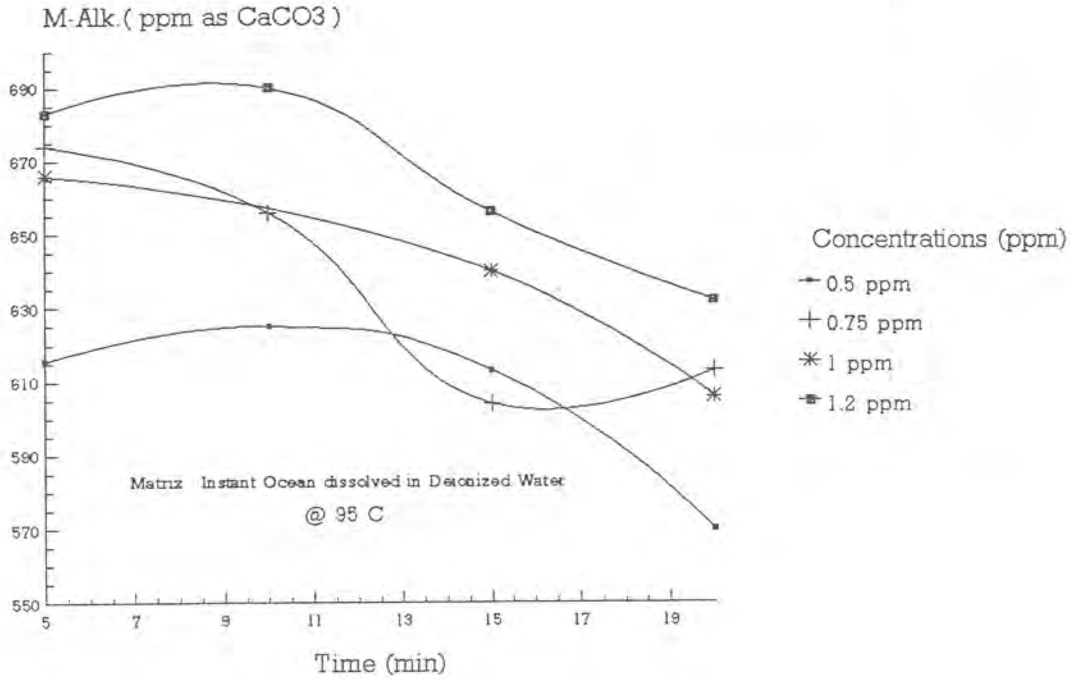


Figure 8 : Optimizing The Concentration of (F)

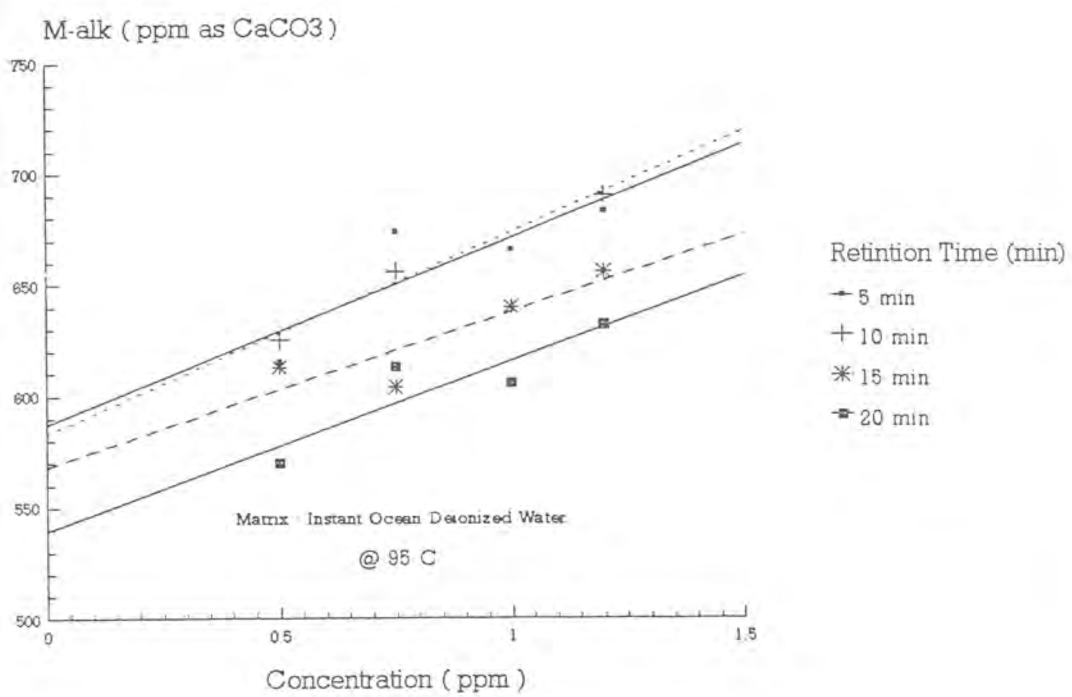


Figure 9 : Optimizing The Concentration of (F)

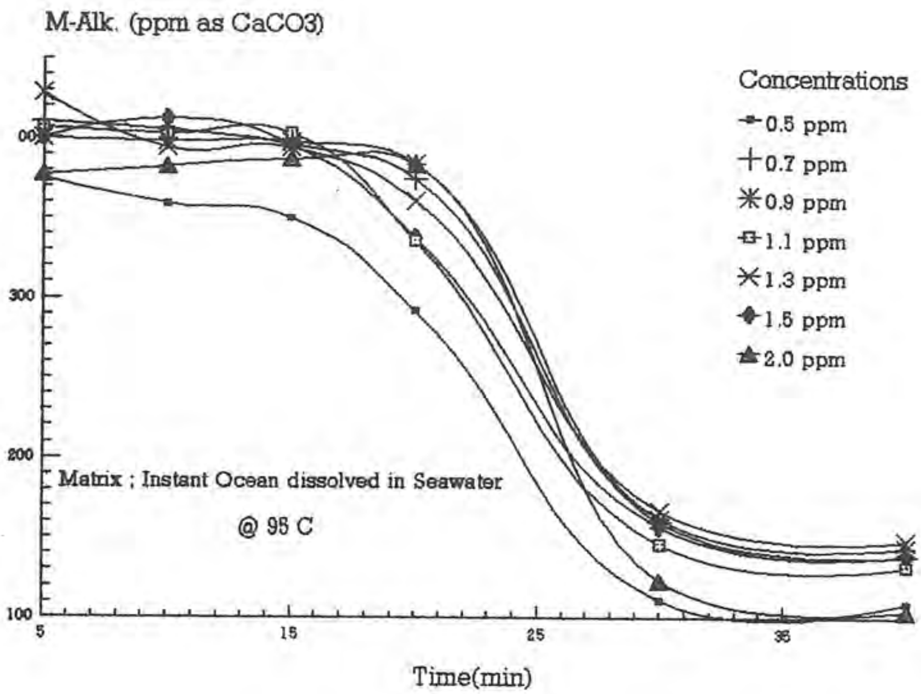


Figure 10 : Optimizing The Concentration of (F)

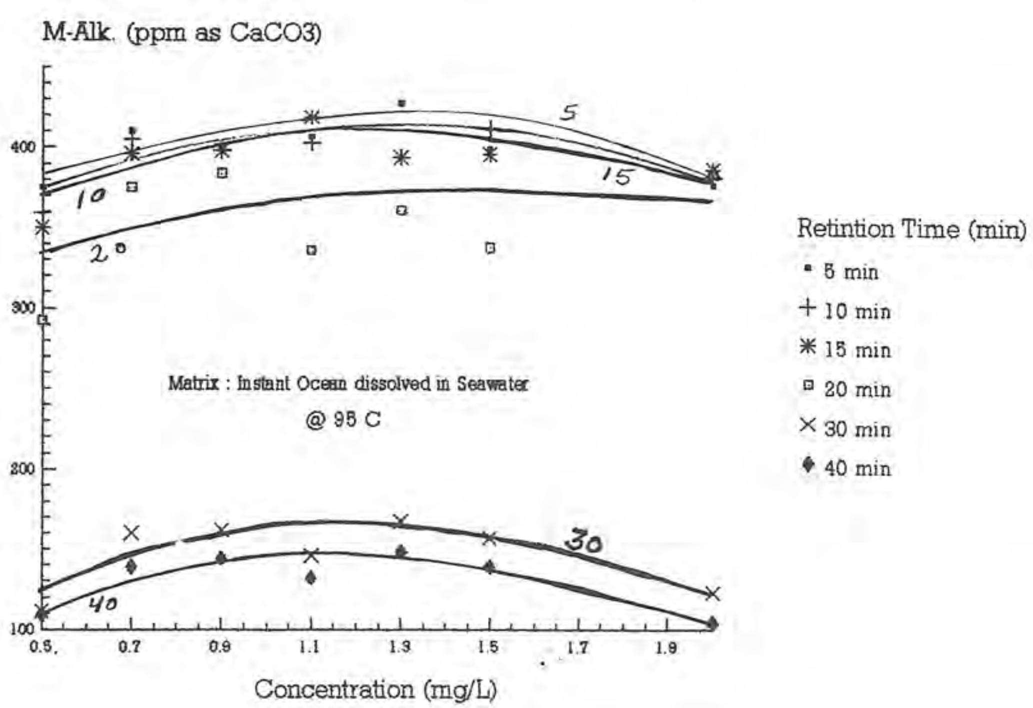


Figure 11 : Optimizing The Concentration of (F)

Identification and Solution to The Unique Problem at SUR SWRO Plant

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IDENTIFICATION AND SOLUTION TO THE UNIQUE PROBLEM AT SUR SWRO PLANT

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ABSTRACT

SUR 1.0 US GPD single stage seawater reverse osmosis (SWRO) desalination plant is the first major single stage SWRO plant built in the Sultanate of Oman to meet part of Sur Town drinking water needs. This plant obtains its raw sea water from beach wells next to the plant.

Since commissioning and commercial startup in early Feb.1993, though the plant had been meeting the normalised qualitative and quantitative output per design, it required frequent reapplication of the protective/ dynamic coating (PT-B) to the RO membranes which was due to presence of oxidant-like substances.

In a classical approach, the Water Department, the O&M Contractor, the RO Desalination contractor, Consultant and Membrane supplier conducted extensive investigation which provided an understanding and solution to the rare type of problem.

This paper describes the investigation efforts and solutions, reviews normalised plant performance and savings in chemicals cost and underlines the achievement made possible due to the team spirit of all concerned parties.

Nomenclature: **PT-A** & **PT-B** are dynamic protective proprietary DuPont chemicals for membrane treatment. **MEW**- Oman Ministry of electricity & water,

Brief Plant description

The beachwell field is adjacent to the plant and comprises of 6 beachwells, each having a depth of 35 meters. Typically, 3 beachwells are required for plant operation while the other three are standby and used during filter backwash operation.

Raw seawater is pumped by the beachwell pumps to a common header and flows into a large covered concrete tank preceding which chlorine injection point is located.. Residence time in the tank is about 45 minutes following which two transfer pumps supply the water to the RO pretreatment system. Facilities to dose coagulant, coagulant aid, and primary acid injection are located on the discharge side of the transfer pumps.

Filtration section comprises of dual media pressure filters; there are two vessels with three chambers each having flow rates of about 8-10 M/H. Media configuration consists of Aquafilt filter media with fine sand and gravel support media. Effluent from the media filters is sent to 5-micron cartridge filters following which dechlorination is done and the water flows to the High pressure pumps. All piping, filters etc. used in the pretreatment are of non corrosive plastic material.

Two Reverse Osmosis blocks, each with their respective high pressure pump and Energy Recovery Turbine are installed. Each train has 118 B-10T DuPont Permeators installed in a single stage arrangement and is provided with appropriate instrumentation, valves, safeties etc. All the high pressure stainless steel work is done using SS 316L.

Post-treatment is done by adding calcium hydroxide and chlorine. Permeator cleaning system, On-Line PT-A / PT-B application facilities as well as automatic PLC control and monitoring facilities are provided (Figure 1).

Problem

After startup and stabilisation in February 1993, qualitative and quantitative performance was satisfactory. But in the following days, permeate conductivity was unstable and would creep up from about 400-500 micromohs (0.5-0.6 % normalised salt passage{SPn}) to about 1200 micromhos (1.2-1.4% SPn) in 10-12 days of operation while product flow and differential pressures remained almost unchanged. This required on-line and some times off-line reapplication of the PT-A / PT-B treatment (which is in-situ protection and salt rejection enhancement chemical treatment recommended by the membrane manufacturer).

As a consequence of the above, chemical cost and downtime was increased in addition to labour. Also, due to this there was considerable anxiety towards the long term effect of this treatment on the membrane which in normal case would be required once in 3-4 months.

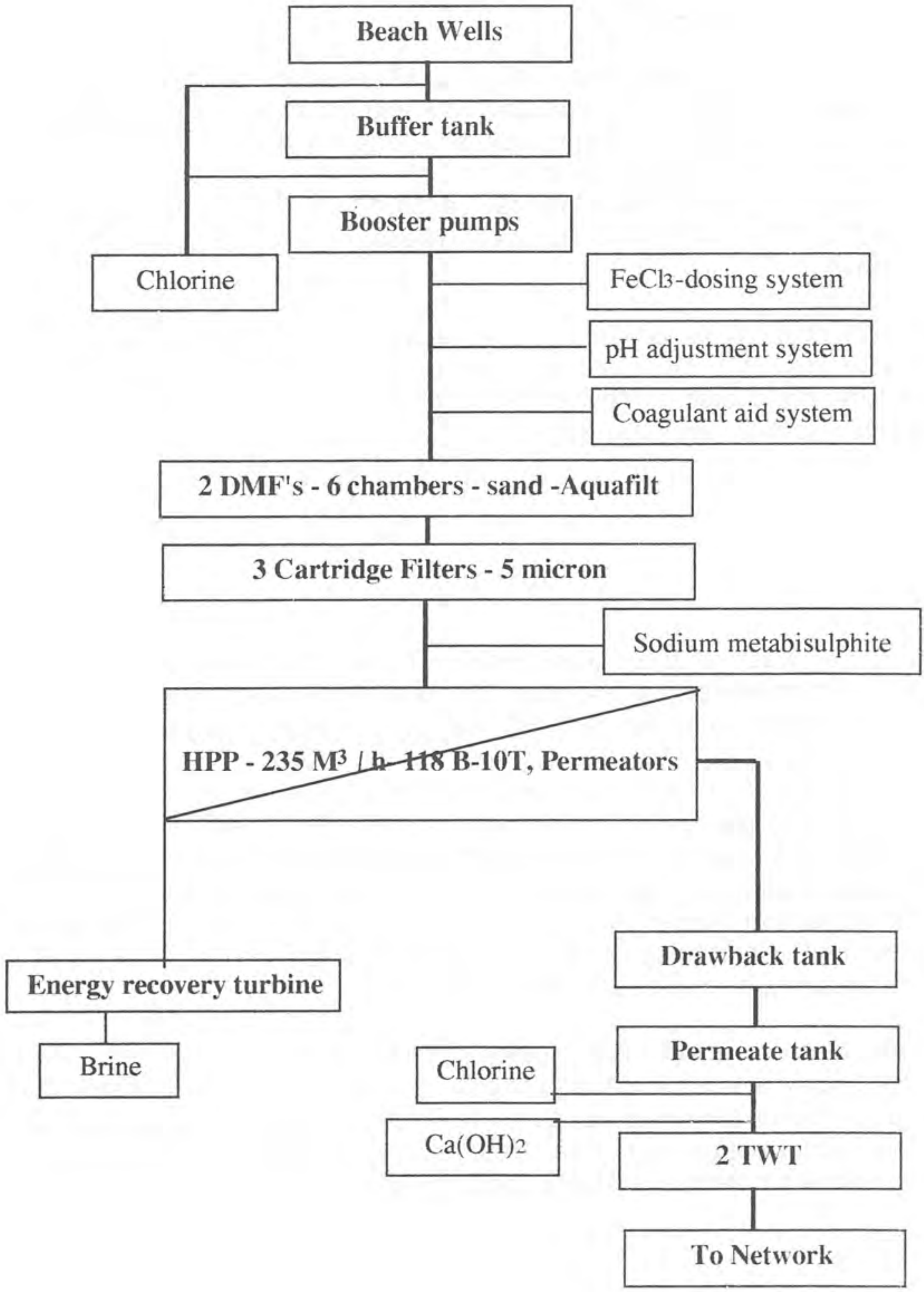


Figure 1-SUR 1.0 MGPD SINGLE STAGE SEAWATER RO PLANT PROCESS FLOW SCHEMATIC

Investigation & Discussion

Based on extensive reviews of operation log, RO data normalisation and due to the fact that there was no change in product flow and Permeators differential pressure, it was apparent that the protective dynamic coating (PT-A / PT-B) was unstable and was getting removed frequently from the membrane surface. This kind of situation had never been seen before. Based on the normal day-to-day analysis conducted at the plant, no apparent reason could be attributed for this phenomena.

Faced with the above problem, the MEW Projects Department, the O&M Contractor, the RO Desalination contractor, Consultant and Membrane manufacturer conducted extensive investigation which provided an understanding and solution to the rare type of problem. The following are the key phases of investigation;

Phase 1

First suspicion was that there was presence of iron in feedwater. Cartridge filters indicated affirmatively and there were also iron residue stain in the raw feedwater tank. Standard iron tests at site of RO feed did not indicate any positive presence.

Several analysis of the well water and feed water (Table 1) were conducted at specialised laboratories, but the results obtained were erratic with poor repeatability. But, due to presence of iron on cartridge filters, investigation was concentrated to pinpoint its source and the beachwell pump risers were suspected. Physical inspection proved that the risers which were made of galvanised steel pipe were indeed corroding. It was decided to replace the risers with FRP risers and subsequently, all well risers were replaced.

Membranes were cleaned, post-treated and the operation re-started. The fouling of the feedwater tank and cartridge filters significantly reduced but still there was the gradual rise in permeate conductivity. The time between reapplication of the membrane protective coatings (Post-treatment) was slightly prolonged to about 15-18 days but the problem of unstable conductivity was very much there.

Phase 2

Naturally, all eyes turned to the RO membrane. DuPont reviewed their manufacturing records and did not find any explanation for this phenomena as all other membranes manufactured in the same batch installed at 7-8 different sites were performing normally. To study this in-depth, DuPont sent 5 other similar model B-10T Permeators membranes which were installed in different locations and control monitoring was done.

Initially, for about two weeks the new membranes from a totally different batch

Table 1 - SUR SWRO PLANT
BEACH WELL WATER ANALYSIS

Date & Time of collection	: 03-05-94-06.00 hrs	**Appearance**	: Clear	**Turbidity-NTU**	: 0.27
Date & Time of receipt of sample	: 04-05-94-11.00 hrs	**Colour**	: Colourless	**Temperature Deg. C**	: 30.9
Date of analysis	: 05-05-94	**Odour**	: Odourless	**pH**	: 7.44
		Taste	: Salty	**Ele. Cond. uS/c.m at 25 Deg. C**	: 55400

ITEM ANALYSED		WELL 1 mg/l	WELL 2 mg/l	WELL 4 mg/l	ITEM ANALYSED		WELL 1 mg/l	WELL 2 mg/l	WELL 4 mg/l
Total Dissolved Solids		37775.5	38584.91	38484.45	Carbonate Alkalinity	as $CaCO_3$	0.0	0.0	0.0
P.Alkalinity	as $CaCO_3$	0.0	0.0	0.0	Bicarbonate Alkalinity	as $CaCO_3$	105.0	109.0	107.5
M.O.Alkalinity	as $CaCO_3$	105.0	109.0	107.5	Dissolved Oxygen	as O_2	5.60	5.11	5.70
Total Hardness	as $CaCO_3$	6730.0	7926.0	6872.0	Free Res. Chlorine	as Cl_2	0.0	0.0	0.0
Calcium Hardness	as $CaCO_3$	1092.0	1110.0	1106.0	Total Res. Chlorine	as Cl_2	0.0	0.0	0.0
Carbonate Hardness	as $CaCO_3$	105.0	109.0	107.5	Chloride	as Cl^-	20815.0	21169.62	20992.23
Non Carbonate Hardness	as $CaCO_3$	6625.0	7817.0	6764.5	Nitrate	as NO_3	0.404	0.580	0.538
Calcium	as Ca	436.8	444.0	442.4	Nitrite	as NO_2	0.0	0.0	0.0
Magnisium	as Mg	1353.12	1635.84	1383.94	Fluoride	as F^-	0.0	0.0	0.0
T. Iron	as Fe	0.284	0.387	0.365	Sulphate	as SO_4	2987.0	3307.0	3288.0
Silica	as SiO_2	4.190	3.77	4.19	Total Phosphate	as PO_4	0.597	0.071	0.426
Sodium	as Na	11560.0	11406.0	11764.0	Ammonia	as NH_4	0.745	0.763	0.442
Pottassium	as K	506.30	508.7	499.4	Carbon Dioxide	as CO_2	5.40	5.69	5.60
Manganese	as Mn	0.038	0.064	0.030	Hydrogen Sulfide	as H_2S	0.0	0.0	0.0
Copper	as Cu	0.127	0.119	0.121	Phenolic Compounds as PHEN-		0.0	0.0	0.0
Zinc	as Zn	0.025	0.027	0.024	OL		0.0	0.0	0.0
Aluminium	as Al	0.0	0.0	0.0	Anionic Detergents				
Lead	as Pb	0.893	0.975	0.944					
Langelier Saturation Index (LSI)		-4.06	-4.36	-4.145					

Reference : Above data taken from Central Laboratory, Gurbah, MEW - SULTANATE OF OMAN resisted the conductivity increase effect which was relatively slow. After about three to four weeks in operation, the rate of conductivity rise became almost similar to the previous membranes.

Based on the above observation, it implied that the prime cause for this rise in conductivity was due to some substance in water which reacted very similar to an oxidising agent and removed the dynamic protection coating (PT-B) layer from the membranes. Many analysis were conducted to determine if any halogens or strong oxidants were present in the RO feed stream but no evidence was detectable.

This important event allayed concern about the suspected problem with the membranes and focus was shifted to investigate further about seawater.

Phase 3

Detailed investigations were started with more precise analytical instruments and methods were adapted to detect trace elements, combined oxidants etc. It was during this phase that the critical breakthrough was attained which helped in resolving the problem and stabilising the operation of the plant.

Several analysis for heavy metals and trace elements were conducted - for *Ni, Zn, Al, Br2, Cl2, I, Cr3, Cr6, Cu, Fe2, Fe3, FeS, Ba, and dissolved oxygen*. In some samples, traces of some heavy metals were detectable. Though some laboratories identified heavy metals in certain samples, it was very difficult to obtain repeatability and consistent values.

Judging from the lab & operation records, it was noted that chlorine residual maintained was in 0.3-0.4 ppm range (Free Cl_2) while the Sodium meta bisulphite (SBS) dosed and consumed was about 6 to 8 ppm. SBS in RO feed was almost in decimal quantities which prompted to conduct various tests of seawater from different process points.

Tests confirmed that the consumption of SBS in raw seawater from the wells was very high while from surface seawater just 50 yards away was as expected. This was done on several samples and it was concluded that the consumption of SBS in Sur beachwell seawater was abnormally high (Table 2).

From the high consumption of SBS, it was quite obvious that some type of oxidant was present in the water that was consuming the SBS. To verify this theory, feedwater pH of RO feed was lowered from 7.0 to 6.2 for about 48 hours to see the membrane behaviour at this pH. Normally, when pH is lowered, it is expected to get slightly higher salt passage (due to higher passage of HCO_3 etc.)initially but generally, this slightly higher value stabilises in about 1-2 hours. At Sur, the upward trend continued at a much exhilarated rate than previous daily increase rate.

Sample Location	Temp. °C	pH	SBS ppm	Residual SMB as NgSO3 (ppm AFTER)			Residual Diss. Oxygen as O2 (ppm) AFTER		
				(2 min)	(5 min)	(15 min)	(2 min)	(5 min)	(15 min)
Sea	25.30	8.10	Nil	00.00	00.00	00.00	08.00		
Water		8.02	10	07.20	07.00	07.25	07.90	07.90	07.80
		7.95	15	11.70	11.60	10.30	07.80	07.80	07.20
		7.70	30	21.10	21.10	19.50	07.40	07.40	07.20
Beach	25.40	7.20	Nil	00.00	00.00	00.00	05.10		
Well		7.04	10	ND	ND	ND	04.40	04.04	04.40
		6.90	15	00.70	00.70	00.60	03.90	04.00	03.40
		6.70	30	03.10	01.90	ND	02.60	02.60	01.30

Table 2- Consumption of SBS in Beachwell & Seawater

This was another strong indication that some oxidant or similar behaving substance was present in water (as oxidants are more aggressive and active at lower pH).

Phase 4

Having identified with certainty that the principle cause was some oxidant like substance (OLS) in the feedwater itself, efforts were focussed to identify the OLS and counter it with the most cost effective solution. Also, by this time, based on actual operation data it was clearly established that the SDI of RO feed was lower than 1.0 as well as biological activity of the water was very low. In view of this, the use of chlorine and coagulant was discontinued.

Also, as a first step, based on laboratory tests, SBS dosing in the feed was increased to 45 ppm and this slowed down the rate of permeate conductivity increase but the problem was not solved. Also, acid addition was stopped since the addition of SBS lowered the pH.

Continued investigative efforts indicated that the reaction time between SBS and OLS was insufficient as it was normally dosed downstream of the cartridge filters. Therefore, SBS dosing point was shifted much further upstream (suction of booster pumps). This almost increased the contact time from less than 1 minute to 20 minutes. SBS dosing was tuned down to about 38 ppm and this additional reaction time stabilised the conductivity.

Having achieved permeate conductivity stability, efforts to further optimise consumption of SBS were continued and it was noted that the incoming water into the feedwater tank was terminating at the top of the tank leaving about 50 cm. gap between the tank level and the incoming pipe. Due to this, the dissolved oxygen level was significantly higher and consequently resulted in higher consumption of SBS. In order to minimise the aeration of water, the inlet pipe was extended by 1.3 meters so that at any time, there is no aeration effect. This small but significant change helped in decreasing the SBS consumption from 38 ppm to 15 ppm.

As more and more analysis results came through, it became apparent that there is a notable presence of heavy metals like lead, copper and zinc etc. which are acting as catalyst and expediting the reaction between SBS and oxygen by enhancing the oxidation stage of dissolved oxygen by formation of nascent oxygen [O] for example Zincate (ZnO_2), Plumbite (PbO_2), Plumbate (PbO_3), Stanite (SnO_2) etc.

Plant performance review

Chart 1-4 indicate the normalised flow and salt passage of the plant which clearly indicate the successful operation of this plant and its performance which has exceeded design criteria.

Conclusion

1)Based on the investigation and analysis, the cause of this rare phenomena which created this problem was primarily due to the presence of traces of heavy metals. Presence of copper, lead, zinc, chromium etc. even in very small trace quantities under certain circumstances can induce the formation of nascent oxygen [O] from the dissolved oxygen or oxides. [O] is very difficult to detect and hence was undetectable by the methods available. It is also extremely reactive and this explains why the consumption of sodium metabisulfite was rapid in beachwell water while it was not the same in seawater.

2) It is also clear that after this problem has been resolved, the plant has performed satisfactorily, with low operating cost, high on-stream availability and reliability. Based on the plants performance, it can be concluded that when the plant design, construction, key equipment / material selection, efficient O&M along with effective client supervision are done, Sea Water Reverse Osmosis technology is reliable and cost effective.

4) It is commendable to note that the team spirit and perseverance with which all concerned parties conducted the investigation efforts helped in formulating the solution to this unique phenomena.

References:

1) Text book of inorganic chemistry by ***P.L. Soni*** (Published by S.C. Publishers, New Delhi)
2) Research note on A New Catalyst Technology For Aqueous Phase Sanitation by ***Charles F. Heinig, Jr*** (Published by Ozone Science and Engineering)

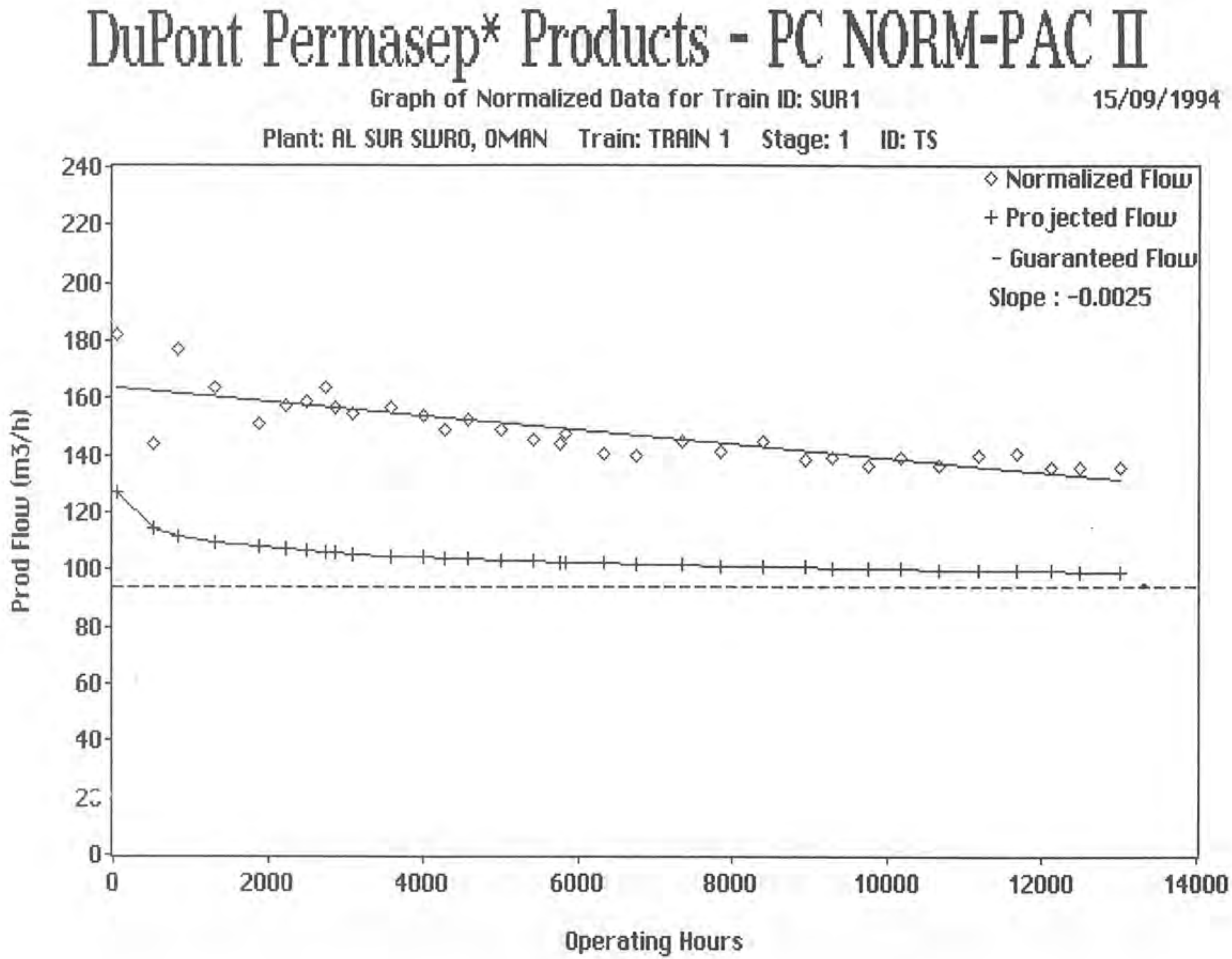

DuPont Permasep* Products - PC NORM-PAC II
Graph of Normalized Data for Train ID: SUR1
15/09/1994
Plant: AL SUR SWRO, OMAN Train: TRAIN 1 Stage: 1 ID: TS
◇ Normalized Flow
+ Projected Flow
- Guaranteed Flow
Slope : -0.0025
Prod Flow (m3/h)
240
220
200
180
160
140
120
100
80
60
40
0
0
2000
4000
6000
8000
10000
12000
14000
Operating Hours

CHART I

DuPont Permasep* Products - PC NORM-PAC II

Graph of Normalized Data for Train ID: SUR1 15/09/1994

Plant: AL SUR SWRO, OMAN Train: TRAIN 1 Stage: 1 ID: TS

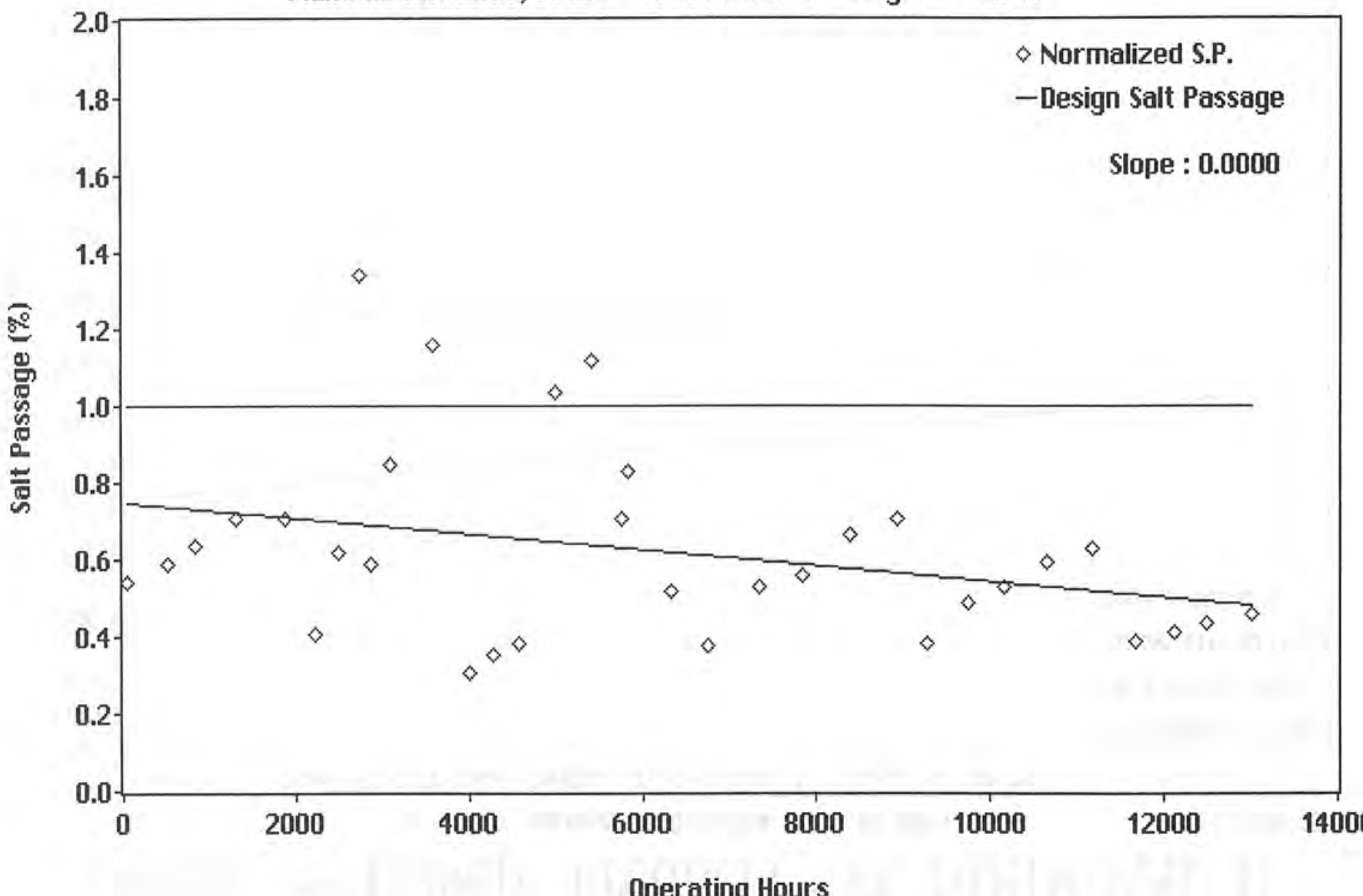
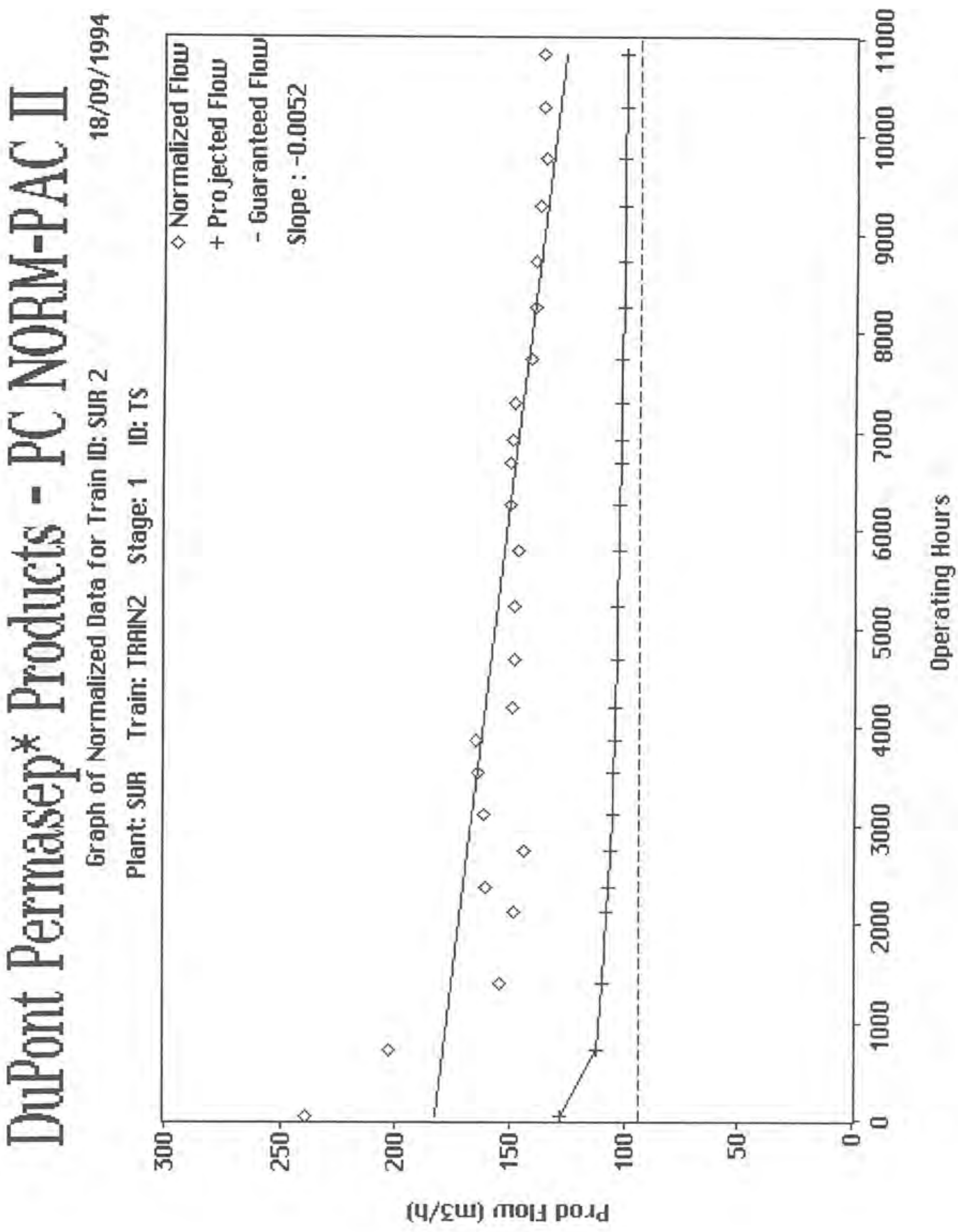


CHART 2


DuPont Permasep* Products - PC NORM-PAC II
Graph of Normalized Data for Train ID: SUR 2
18/09/1994
Plant: SUR Train: TRAIN2 Stage: 1 ID: TS
◇ Normalized Flow
+ Projected Flow
- Guaranteed Flow
Slope : -0.0052
Prod Flow (m3/h)
300
250
200
150
100
50
0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
Operating Hours
CHART 3

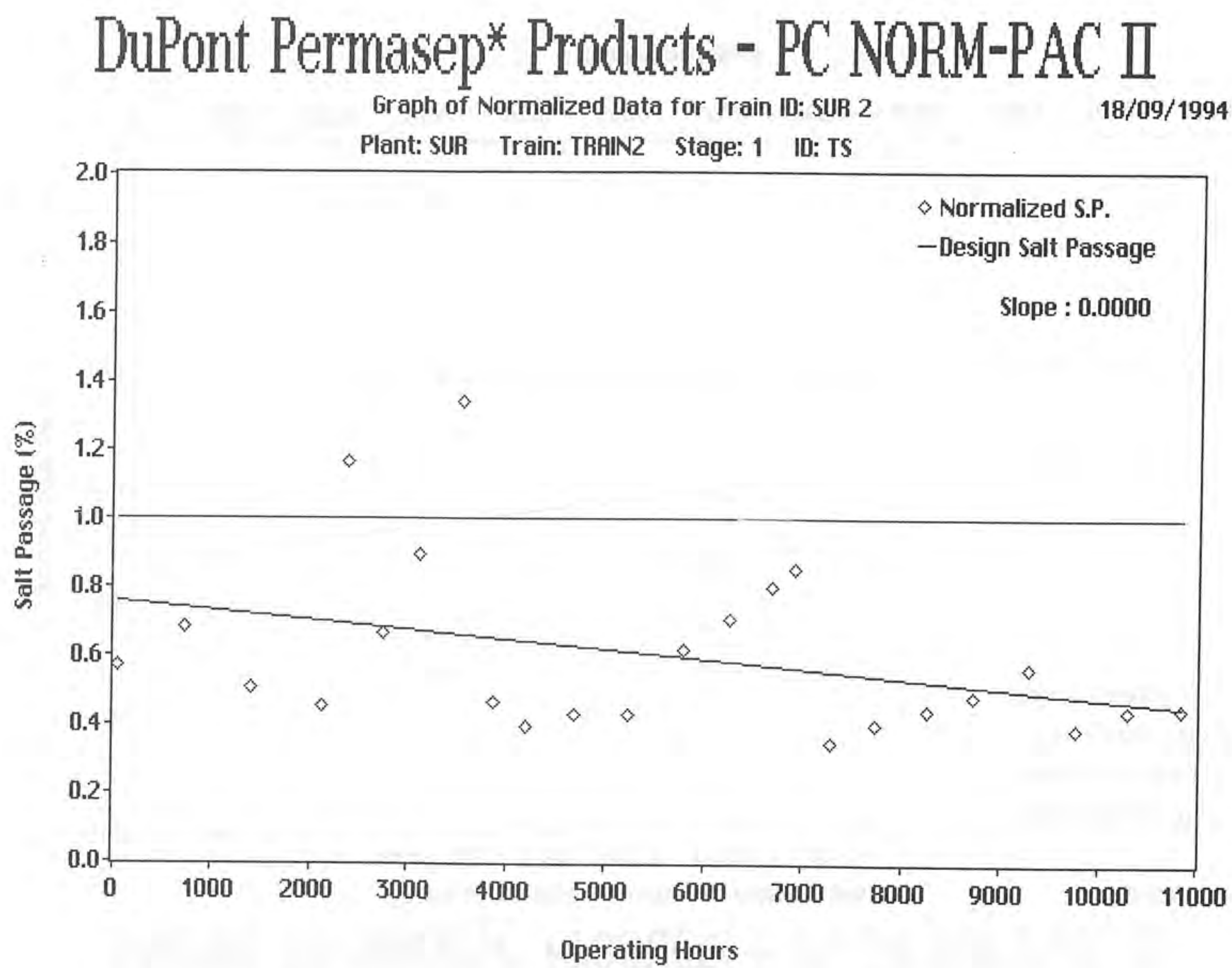

DuPont Permasep* Products - PC NORM-PAC II
Graph of Normalized Data for Train ID: SUR 2
18/09/1994
Plant: SUR Train: TRAIN2 Stage: 1 ID: TS
◇ Normalized S.P.
—Design Salt Passage
Slope : 0.0000
Salt Passage (%)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
Operating Hours

CHART 4

EFFECTS OF INCUBATION TEMPERATURE AND NUTRIENTS ON GROWTH POTENTIAL OF MARINE BACTERIA (AL-JUBAIL SEAWATER)

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ABSTRACT

The agar plate count method is widely used for quantifying viable bacteria in seawater, specially in routine laboratories. Twelve agar media were evaluated for colony forming ability at 18^{0}C, 26^{0}C and 35^{0}C incubation temperatures. After 72h incubation the maximum number of colonies were observed on Marine agar (agar#7). Plate Count agar (agar #1) was also found suitable for bacterial culture. However, Nutrient agar (agar #4) was found unsatisfactory for bacterial growth. Incubation temperature had a decisive role in the initiation of bacterial growth, and during the initial days of incubation more colonies appeared on the agars at higher temperatures. An incubation temperature of 30^{0}C was found suitable for colony development within 72-96h.

Based on the data obtained, agar plate count is being carried out on Plate Count agar or Marine agar at 30^{0}C incubation temperature for a period of 72h to evaluate disinfectant effectiveness in pretreatment and biofouling monitoring of SWRO pilot plant. This study demonstrated that hetrotrophic aerobes of Al-Jubail seawater are able to tolerate high nutrient concentration and grow rapidly in nutrient rich media such as Marine Agar and Plate Count Agar. Relevance of the ability of bacteria to thrive in nutrient rich media especially at elevated temperatures is discussed with reference to biofouling in seawater reverse osmosis desalination.

INTRODUCTION

Marine habitat shelters a vast number of diverse microfauna and flora that are similar to the types found in terrestrial and freshwater realms. The enumeration of bacteria in seawater at various locations, depths and during different seasons of the year is a basic requirement in marine microbiology. However, quantification of bacteria in nature still remains to be fully accomplished. This is largely due to the lack of suitable methods which can be applied to vastly different ecosystems and complex nature of microbial community they sustain (Karl, 1986). Reliable and easy to use methods such as agar plate count, and the development of general and specific culture media facilitated the accurate enumeration of viable bacteria in medical and food microbiology. The situation is quite different in environmental microbiology. Living bacteria from soil and aquatic environments cannot be accurately enumerated (Lewin, 1974; Atlas and Bartha, 1987). Non-critical use of methods widely employed in counting may result in inaccurate or even misleading data as microorganisms live in microenvironments having distinctly different physico-chemical properties (Karl, 1986).

Many slow growing bacteria, which take more than 72h to form visible colonies are present in water (Reasoner and Geldreich, 1985). In addition, there is a time lag before the initiation of cell division, which is not identical for every bacteria or every cell of a strain (Hattori, 1988). From plots of visible colonies appearing over time the kinetics of colony formation and the optimal incubation time can be determined. Similarly, incubation temperature has a decisive effect on bacterial growth (Brock and Madigan, 1991). Environments experiencing wide temperature fluctuations may harbor psychrophilic and thermophilic bacteria in addition to mesophilic forms. Ideal incubation temperature for environmental microbes is presumably different from that selected for common pathogenic bacteria. Three different incubation temperatures represented by winter, summer and spring plus autumn averages, were selected to evaluate the suitability of different agar media in the cultivation of marine bacteria from Al-Jubail.

MATERIALS AND METHODS

Sample : The study was conducted during the month of June 1993. Surface seawater from Al-Jubail desal plant's intake basin was collected in sterile glass bottles, adopting the sampling procedure outlined in Standard Methods (APHA 1985). Samples were serially diluted in sterile seawater. Dilutions 10^{-1} through 10^{-4} were plated within 30 minutes after the collection of samples.

Media tested : Twelve media of different nutrient composition were used in the study (Table I). Standard Plate Count Agar (PCA) was tested at x1, x1/5 and x1/10 strength - media 1, 2 and 3 respectively. Media 4, 5 and 6 were Nutrient Agar (NA) x1, x1/5 and x1/10 respectively. Marine Agar (MA) was also tested at the same strengths (media 7, 8, and 9). Media 10, 11 and 12 were prepared by combining peptone, yeast extract and glucose at different concentrations as given in Table I. All the reagents used in the study were obtained from DIFCO Laboratories.

During the preparation of dilute PCA, MA and NA sufficient quantity of agar was supplemented to keep the agar concentration at 12-15gm/liter. All media except 7, 8 and 9 were prepared in filtered (0.45um) seawater collected from the intake. Media 7 was prepared using deionized water. Media 8 and 9 were prepared in deionized water supplemented with sufficient sea salts to get an osmotic strength equal to that of media #7. Media were dissolved and sterilized in an autoclave at 121^0C and pressure 15 psi for 15 min.

Plating : Standard pour-plate method was followed for the cultivation of bacteria. An aliquot of 1.0 ml sample was aseptically pipetted into sterile plastic petri dish. Sterile molten agar maintained at 46^0C in a water bath was poured into the plate and mixed well with the inoculum. Agar was allowed to gel for about 15 minutes, plates were inverted and incubated at appropriate temperature. Three petridishes of each dilution were incubated at 18^0C, 26^0C and 35^0C. Plates were counted every 24hr upto 10 days using a *Fisher* Accu-Lite colony counter. Colony forming curves were constructed by taking the daily means of total counts on a semi-log paper.

RESULTS

Incubation Time : The number of colonies formed on different agar media varied widely. In some agar media bacteria attained the maximum number within a shorter period of incubation, whereas, others took longer time to attain the maximum growth. In general, the highest values were reached between 6 and 10 days of incubation, depending on the type of agar and incubation temperature. During the initial 72h the colony forming rate on all the agars was greater at higher temperature (26^0C and 35^0C), attaining maximum growth comparatively earlier than at 18^0C(Table II). Colony formed on the agars between the 3rd and the 10th days was not directly related to the duration of incubation, signifying a lower growth rate after 72h (Table II). Best bacterial growth was achieved on media #1 and 7, on which between 65% to 94% of the total colonies were formed within 72h of incubation. Subsequent incubation for additional 7 days resulted only in a marginal increase in number of colonies, specially at 26 and 35^0C. In some agars, those that developed higher number of colonies during the initial days of incubation, displayed a tendency to register less number of colonies on 10th day, attaining a peak between 6 to 9 days. This tendency again, was more prevalent at higher incubation temperatures.

Incubation Temperature : Colony formation during the initial hours of incubation at 18^0C was substantially low compared to 26^0C and 35^0C. Following 72h incubation at 18^0C, agar #1 yielded $6.4x10^3$ CFU/ml. After a similar period of incubation at 26^0C and 35^0C, agar #1 developed $9.6x10^3$ and $1.1x10^4$ CFU/ml respectively (see Fig. I). Similarly, on incubation at 18^0C, 26^0C and 35^0C agar #7 developed $7.4x10^3$, $1.1x10^4$ and $1.7x10^4$ CFU/ml respectively after a period of 72h (see Fig. II). At 18^0C incubation temperature between 24h and 72h, bacterial colonies multiplied in the range of 10-176 times in different agars, whereas, at 26^0C and 35^0C the increase was substantially low, between 1-16.7 times (at 26^0C) and 1.3-2.6 times (at 35^0C), as evident from Table II. The mean $\pm$ standard deviation of colonies formed after 10 days on all the agars incubated at 18^0C was $1.0 \pm 0.3x10^4$ CFU/ml. This value is 11% lower to the mean CFU/ml obtained at 26^0C ($1.12 \pm 0.22x10^4$) and 21% lower than that recorded at 35^0C ($1.27 \pm 0.31x10^4$). Total colonies formed after a similar incubation period at 26^0C was 12% less than at 35^0C.

Nutrient Composition : Nutrient composition of the agar media was decisive in the initiation of colony formation and also in the total colony formed after 72h incubation. This effect was less pronounced on prolonged incubation and at higher incubation temperatures of 26^0C and 35^0C. It is interesting that agar #12, the lowest in nutrient content, developed more or less the same number of colonies as observed in nutrient rich agars # 1, 4 and 7 following 10 day incubation at 18^0C and 26^0C. The number of colonies formed on agar #12 after 10 day incubation at 35^0C was about 50% less than those formed on nutrient rich agars. Agar # 4 and 5, though rich in nutrients, performed poorly at all incubation temperatures.

DISCUSSION

Temperature, nutrients and ionic strength are very important in bacterial growth and its regulation (Brock and Madigan, 1991). It is essential to maintain these parameters at optimum levels for reliable enumeration of marine heterotrophs. Natural seawater is characterized by extremely low nutrient concentration. Total organic carbon (TOC) in seawater ranges frcm an average of 1.0 mg/l in surface waters to 0.5 mg/l of deep seawater (Menzel and Ryther 1968). Of the TOC in natural waters only a small percentage (2-5) are available for bacterial consumption. Marine bacteria are adapted to live in very low levels of organic carbon, and the term 'oligotroph' is coined to describe those bacteria that can grow in a medium containing < 1.0 mg/l organic carbon (Ishida and Kadota, 1981).

For cultivating marine bacteria marine agar - a medium containing about 10,000mg/l organics, is often used. Such a medium presents the bacteria with an environment very different from the one they are familiar with. Adaptability is the key to overcome this hurdle and their success is essential for a realistic estimation of bacterial biomass in seawater. High concentration of organic nutrients is expected to inhibit the growth of at least some of the bacteria which normally have to survive and grow in seawater with only a few milligrams of organic matter per liter. Ishida and Kadota (1981) reported that many oligotrophs failed to grow in medium containing organic carbon concentrations of 5g trypticase/l. In a separate study Lewin (1974) found that growth of only a small percentage (8%) of inshore bacteria was inhibited by yeast extract at a concentration of 1.0 mg/l. Some heterotrophic bacteria found in Al Jubail coast are able to grow in media containing 0.075 to 8.5 g organics per liter. This observation suggests that some inshore bacteria are are able to grow in very dilute nutrient media. Nutrient rich media such as, marine agar or plate count agar could generally be used without any appreciable reduction in colony numbers due to inhibition by organic supplements. Such excesses in nutrient are probably unnecessary since low nutrient media gives satisfactory bacterial growth.

In addition to the total organic content of culture medium, the actual nutrient composition of the media in terms of presence or abundance of nitrogen rich compounds are reported to affect colony formation. During their study on nutritional requirement of heterotrophs in cooling water Brozel and Cloete (1992) observed that lower nitrogen content was beneficial to colony formation. The results of the present study show that initiation of growth of marine bacteria invariably was better on media containing yeast extract. Presence of glucose in the medium was not as beneficial in growth initiation, suggesting that carbon source was not a growth stimulatory factor in this environment. In another study (unpublished) it was observed that addition of yeast extract into seawater samples resulted in rapid growth of bacteria. This phenomena was less evident after the addition of peptone and was the least following glucose addition. To culture marine heterotrophs of the Gulf coast it is, therefore, proposed to use media containing yeast extract which contain nitrogen rich compounds. The mechanism by which yeast extract could enhance bacterial growth initiation was neither specified nor explained. It is probable that components present in yeast extract exert some form of inducing action on dormant cells (Roth *et al.*, 1988) or prevents unbalanced growth of starved or damaged cells (Dawes 1989).

Higher incubation temperature selected in the present study was apparently below the tolerance limit of bacteria. Higher temperature in fact, initiated metabolic processes and growth of bacteria immediately, thereby reducing the lag period before exponential growth which is evident from the initial burst of growth observed at this temperature. Total number of colonies formed on agar #12 at 35^{0}C after 10 days was relatively lower than that recorded on other agars. This is contrary to formation of comparable number of colonies on all the agars at 18^{0}C and 26^{0}C. It is assumed that in this case, initial increase in metabolism exhausted available nutrients to support continuous growth at 35^{0}C. An optimum temperature of 25^{0}C was suggested by ZoBell (1954) for incubation of marine bacterial culture. It is apparent that this temperature was selected in accordance with the prevailing temperature range in Pacific coast, and it cannot be applied indiscriminately for all the marine bacteria. Although surface seawater temperatures could reach 35^{0}C during summer in the Arabian Gulf, it was established that a median temperature of 30^{0}C is satisfactory for routine bacterial culture.

The present study highlighted some important points regarding the characteristics of bacteria in the Gulf having great significance in microbial fouling of SWRO systems. TOC level in Al-Jubail seawater is in the range of 0.75-2.6 mg/l, the average being 1.8 mg/l. Bacteria living in such a low TOC environment was able to adapt and thrive in a medium containing about 10,000 mg/l organics. Their ability to respond positively to high nutrient input is a matter of concern from microbial fouling point of view. In a SWRO plant possible introduction of nutrients at any stage of pretreatment is quite likely to result in rapid multiplication of bacteria in the feed. Previous studies have shown that in general pioneering bacteria present in seawater quickly take up nutrients and grow rapidly on surfaces initiating a chain of events culminating in biofouling of the RO system (Winters, personal communication). In addition, temperature initiates rapid growth, significantly reducing the lag period before exponential growth. In these experiments nutrients and temperature acted together promoting a rapid proliferation of bacteria. As the Gulf region experiences more of a warm to hot climate 6-8 months per year, any nutrient input during pretreatment will have a highly significant effect on bacterial growth resulting in membrane biofouling. Great care is, therefore, advocated in design and implementation of RO pretreatment procedures. Close monitoring of bacterial population is very essential at every stage of pretreatment to facilitate early detection and effective control of biofouling during the initial stages.

CONCLUSIONS AND RECOMMENDATIONS

A range of 12 agar media was studied for their ability to support growth of marine bacteria from Al-Jubail. Nutrient content in these agars varied between 0.075g - 8.5g organics per liter. By the end of 10 days incubation bacterial count on all these agars did not vary drastically among the agars as would be expected from the vast difference in their nutrient contents. Inclusion of yeast extract in agar media was found to stimulate rapid growth of marine bacteria, possibly due to its high nitrogen content. On the other hand, addition of glucose, an easily assimilable carbon source, had no significant effect on colony formation. Of the 12 agars tested, Plate Count agar (medium # 1) and Marine agar (medium # 7)were the best in terms of growth initiation and sustenance.

Incubation temperature had direct effect on initiation of colony formation. Rate of colony formation increased with increase in temperature. In general, this effect was more pronounced on the plates incubated at 26^{0}C and 35^{0}C compared to those incubated at 18^{0}C. Based on the colony forming curves obtained at three different incubation temperatures a median temperature of 30^{0}C was selected for incubation of agar plates. This median temperature is also the upper limit commonly selected for the cultivation of environmental bacteria.

A steady increase in the number of colonies developed was observed on most of the agars with incubation time. In general, a peak was reached between 6 and 10 days depending on the agar type and incubation temperature. Most of the bacterial colonies were formed on the agars within 72h incubation and further incubation upto 10 days resulted only in a marginal increase in colonies. For practical applications an incubation period of 72h was found to be satisfactory.

Some bacteria in the Gulf are not obligate oligotrophs. They are able to tolerate high concentration of nutrients and grow rapidly. Temperature further accelerated their growth. This property of bacteria demands extra caution in RO feed pretreatment not to increase nutrients in the system. Continuous supply of nutrients and a favorable temperature, which is an intrinsic component of the Gulf environment, could result in rapid proliferation of microbes and subsequent biofoulig of SWRO facilities.

From this experiment it can be concluded that for enumeration of marine bacteria from Al-Jubail, agar media such as, Plate Count agar or Marine agar can be used. Incubation of agar plates at 30^{0}C yields representative growth within a reasonable period of 72h. It is recommended to adopt this procedure for bacterial count in studies on disinfectant evaluation and biomonitoring of feed pretreatment and membrane fouling in SWRO plants.

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Fig. I
BACTERIAL GROWTH IN AGAR # 1
AT DIFFERENT TEMPERATURES

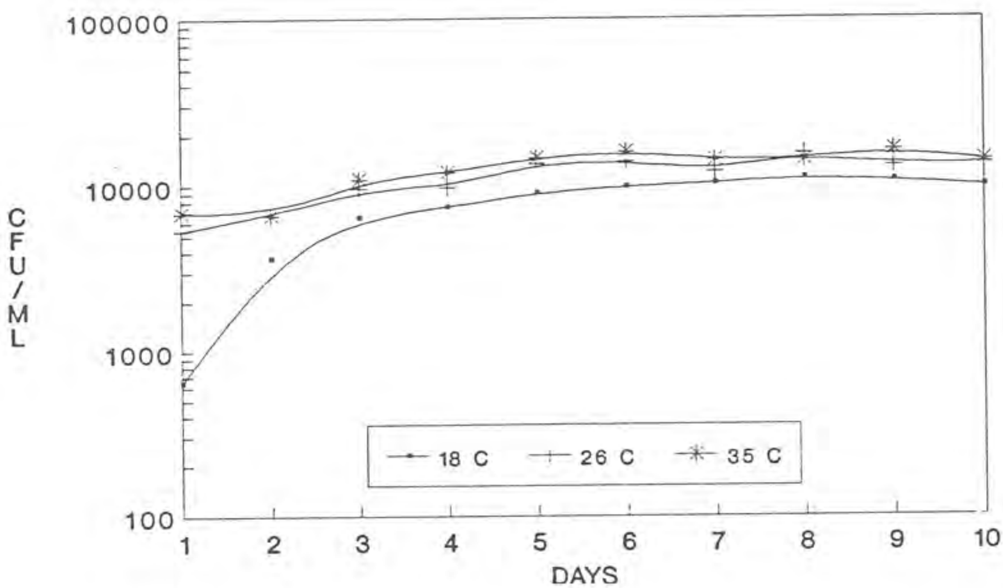

Fig. II
BACTERIAL GROWTH IN AGAR # 7
AT DIFFERENT TEMPERATURES

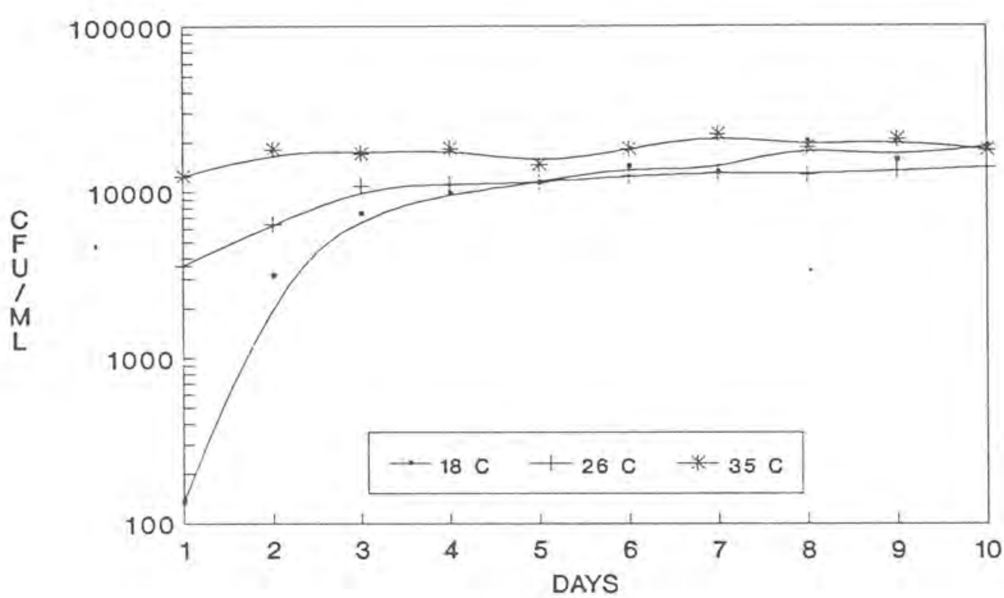

TABLE I

COMPOSITION OF AGARS USED IN THE STUDY
(Data given as g^{-1} of final medium)

Component / Media	Peptone	Tryptone	Yeast Ext.	Beef Ext.	Glucose	Total	Agar
1		5.0	2.5		1.0	8.5	15
2		1.0	0.5		0.2	1.7	12
3		0.5	0.25		0.1	0.85	12
4	5.0			3.0		8.0	15
5	1.0			0.6		1.6	12
6	0.5			0.3		0.8	12
7	5.0		1.0			6.0	15
8	1.0		0.2			1.2	12
9	0.5		0.1			0.6	12
10	0.1		0.1		0.1	0.3	12
11	0.05		0.05		0.01	0.15	12
12	0.025		0.025		0.025	0.075	12

TABLE II

INCREASE IN BACTERIAL COLONIES ON VARIOUS
AGAR MEDIA AT DIFFERENT INCUBATION TEMPERATURES
(CFU/ML x times)

DAY:DAY*	3:1			10:1			10:3		
	TEMPERATURE ^{0}C								
AGAR MEDIA	18	26	35	18	26	35	18	26	35
1	10	1.8	1.6	15	2.4	2.0	1.5	1.4	1.3
2	13.3	4.7	2.6	38	10	4.6	2.8	2.1	1.8
3	21.6	1.6	1.8	52	4.0	3.1	2.4	1.6	1.8
4	132	1.9	1.8	360	2.7	2.6	2.7	1.4	1.4
5	NA	1.0	1.6	NA	2.4	4.8	4.5	2.4	3.0
6	NA	16.7	1.6	NA	32	2.1	2.5	1.9	1.3
7	53.0	3.1	1.4	136	3.9	1.5	2.6	1.3	1.1
8	18	2.5	2.4	65	3.5	4.4	3.5	1.4	1.8
9	53	2.8	2.2	139	5.9	3.1	2.6	2.1	1.4
10	26	2.5	1.9	48	4.2	3.0	1.9	1.7	1.6
11	87	2.0	2.2	216	2.4	3.1	2.5	1.6	2.0
12	176	5.2	1.3	400	11	1.6	2.3	2.1	1.3

* Ratio of bacterial count on days indicated

Fouling Prevention in Desalination Plants

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FOULING PREVENTION IN DESALINATION PLANTS

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ABSTRACT

Combined low concentrations of copper (5 ug/l) and chlorine (50 ug/l) have been effective in preventing both micro- and macro-fouling in over 120 seawater installations since 1987. this paper will outline the development of the technology, and demonstrate how it can be applied to the desalination industry. Recent trials with a copper / chlorine dosing unit on a reverse osmosis (RO) test rig in the Gulf will be discussed.

KEY WORDS

Copper, chlorine, biofouling, desalination.

INTRODUCTION

The BFCC copper / chlorine system prevents the attachment and growth of micro- and macro-fouling organisms by the simultaneous use of low levels of copper (5 ug/l) and chlorine (50 ug/l). The effect of the two together is synergistic, such that the combined antifouling effect is greater than would be predicted by summing the effects of the individual components.

Laboratory research at the University of Sheffield indicates that in the case of macrofouling, the mode of action of the BFCC system is sub-lethal. Copper and chlorine were used to discourage the post larval settlement stage of the mussel **Mytilus edulis (L.)** from attaching to an otherwise suitable substrate. Once in clean water, the mussels rapidly recovered (1). In field trials with the now privatised Central Electricity Generating Board (CEGB - UK), untreated natural seawater was pumped on a continuous once through basis through a number of mild steel pipes. The pipes were treated as follows; 2 with electrolytically generated copper ions at 35 ug/l (ppb), 1 with electrolytically generated copper and aluminium ions (35 ug/l copper & 5 ug/l aluminium; manufacturer's standard), 1 with conventional chlorination at 200 ug/l, 1 with combined copper/chlorine treatment (5 ug/l copper & 20 ug/l chlorine) and 1 control. The results are summarised in Figure 1.0. In pipes treated with 200 ug/l total residual chlorine (TRC) there was a significant reduction of biofouling compared to the control. In lines protected by the copper / chlorine system, the greatest degree of protection was observed. Copper / aluminium and copper alone had no significant effect in reducing fouling biomass.

The effectiveness of the copper / chlorine system against bacteria and microalgae can be inferred from microfouling studies conducted by the University of Miami, Department of Engineering, using a purposely built US Electricity Power Research Institute (EPRI) designed condenser tube test heat transfer resistance (Fig. 2). Conventional chlorination and the BFCC copper / chlorine treatment were tested against a control. In the chlorination line, chlorine was dosed at the maximum level permitted in the USA (200 ug/l Total Residual Chlorine (TRC) for 2 h/d). In the copper / chlorine treated lines 5 ug/l of copper was used in conjunction with 20 ug/l of TRC for 2 h/d. The copper / chlorine treatment prevented microfouling, and was more effective than the conventional chlorination regime (Fig. 3). The use of copper alone was not tested since copper is known to increase bacterial sliming and hence heat transfer resistance (2). The development of the copper/chlorine system is described in full elsewhere (3).

It is postulated that chlorine alters the permeability of the cell membrane, thereby facilitating the passage of copper into the cell where it attaches to enzymes, causing cellular disfunction. This is borne out by permeability experiments on artificial membranes (Nelson, personal communication with BFCC).

Problems caused by macrofouling include reduced water flow, pump head losses, blockage of screens caused by the sloughing of large sections of the fouling mat and blockage of coarse filters. The growth of micro-organisms, especially bacteria, reduces the efficiency of heat exchange processes, increases corrosion of susceptible materials and predisposes surfaces to colonisation by macrofouling organisms. In RO plant, bacteria can adversely affect the process by growing on the RO membrane. Symptoms of fouling are a rapid build up of differential pressure followed by a reduction in flux and decrease in salt rejection (4). An additional problem, specific to RO desalination , is that the usual method of bacterial control is via chlorine. Whilst chlorine is effective in the control of bacteria, it will physically damage polyamide (PA) membrane material at the concentrations of chlorine required to control growth (5). For this reason, certain membrane manufacturers specify a zero mg/l chlorine content for water entering certain designs of PA permeator (6). The use of high concentrations of chlorine has also been implicated in bacterial aftergrowth on RO membranes through the formation of assimilable organic carbon (AOC) from organic material, which then acts as a food source for bacteria (7).

The BFCC copper / chlorine solution offers the prospect of being able to:

1) prevent macrofouling in the seawater intake system of thermal and RO desalination plants with minimal chlorine and copper usage;

2) prevent microfouling at all stages of the RO process without causing membrane damage, and without stimulating bacterial growth;

3) prevent biofouling without adversely affecting coarse and fine filter performance.

THE EXPERIMENT

In view of the success of the BFCC technology in preventing biofouling in a range of marine installations around the world, a joint project was set up between the Saline Water Conversion Corporation (SWCC) and BFCC to explore possible applications of the copper / chlorine technology in the desalination industry.

In June 1993, an electrolytic copper / chlorine dosing system was installed at the SWCC Research Development and Training Center at Al-Jubail, Kingdom of Saudi Arabia, to demonstrate the effects of a copper / chlorine system on the seawater inlet pipe-work and pre-treatment stage of an RO process train.

The layout of the trial plant is shown in Fig. 4. Between 36 m^3/h and 66 m^3/h of raw, unchlorinated seawater was drawn from the inlet bay of the Al Jubail plant by one of 2 pumps on a floating platform. Between 25% and 100% of the main flow was directed through a purposely built dosing chamber where it was treated with copper and chlorine. The dosed water was then reintroduced into the main seawater flow, travelling through 900 m of 110 mm ID pipe to the pre-treatment skid. The effective BFCC copper / chlorine dose is 5 ug/l and 50 ug/l, respectively. However, in view of the transit time (6 to 13 minutes) of the water between the dosing point and the pre-treatment plant (sand filters), 200 ug/l of chlorine was added along with 5 ug/l of copper.

The copper / chlorine dosing unit comprised a single 200 mm diameter, 300 mm high mild steel dosing chamber with one vertically mounted copper anode and one vertically mounted Mixed Metal Oxide (MMO) chlorine producing anode. The inside surface of the dosing chamber acted as the cathode in the electrolysis reaction. Constant direct current was impressed on the electrodes from a transformer rectifier control panel. The magnitude of the impressed current determines the mass of chlorine formed, according to Faraday's Law. The current, and hence mass of chlorine produced, remained constant throughout the experiment. The water flow rate varied, however, so the recorded copper and chlorine concentrations were not constant. The direct current to the copper and MMO electrodes was pulsed 1 minute ON and 1 minute OFF. This is standard BFCC practise in industrial installations, its purpose being to reduce cathode scaling and extend anode life.

The facility for biofilm monitoring at different stages of the RO process was included in the design by providing removable pipe sections at various points in the process train.

Water quality was measured as Silt Density Index (SDI). Copper and chlorine concentrations were determined using colourimetric methods. Copper was assayed with sodium diethyl-di-thiocarbamate, colour absorbance being measured at 440 nm with a spectrophotometer. Chlorine was measured using the standard DPD (N,N Diethyl-P-phynylene diamine) technique, absorbance being measured at 565 nm. At very low concentrations (less than 500 ug/l), chlorine was determined using the Orion 9770 residual chlorine electrode which measures the amount of iodine liberated from potassium iodide added to water samples. The iodide is oxidised by chlorine produced oxidants (CPO) in the water.

Samples for chemical and bacterial analyses were taken at different stages of the RO process, i.e., raw seawater (RSW), before the feed entered the pre-treatment (BPT), after coagulation and sand filtration (ASF) and after the cartridge filter (ACF), as shown in Figure 4.

RESULTS

The measured chlorine concentrations in the outflow of the dosing chamber varied between 190 ug/l and 22,500 ug/l. This variation was mainly due to variation in flow rate plus changes in chlorine demand and the effect of pulse dose timing. Pulsing the current to the electrodes ON and OFF results in slugs of copper / chlorine treated water passing through the process plant. If a water sample is taken when the current to the electrodes is OFF, relatively low chlorine levels will be measured, and vice versa. The further away the sampling point is from the dosing chamber, the harder it is to predict whether a treated or untreated slug is being sampled. In any event, Faraday's Laws of electrolysis make it possible to state that the mass of chlorine generated remained constant throughout the trials, based on the fact that a constant direct current output was impressed on the MMO electrode.

Copper concentrations (copper normally present in the water plus BFCC generated copper) ranged from 3 ug/l to 9.1 ug/l at the inlet of the pretreatment plant. Coagulation and sand filtration resulted in a reduction in concentration to between 0.9 ug/l and 1.5 ug/l. This was due to the complexation of the copper ions (Cu^{2+}) with organic material in the water, and the subsequent coagulation and removal of the chelated copper in the filter.

Both copper and chlorine concentrations were reduced in the time that it took the treated water to get from the BFCC treatment unit to the entry of the pre-treatment plant. The chlorine would have been consumed in demand reactions, whilst the copper would have been complexed with organic material in the water.

DISCUSSION

In terms of plant performance, the slug dosing effect is not significant. Biofouling can be defined as the growth of organisms on surfaces (periphytic growth). Once a fouling organism leaves a water flow and becomes fixed relative to the flow of water, it experiences the slug doses and either detaches or is killed. Planktonic organisms entrained in the water flow that pass straight through the system do not constitute a biofouling problem.

It is possible that the BFCC system is acting as a bactericide and/or a bacteriostat. If there is a bacteriostatic action then water sampling will remove the inhibitory effects of a copper / chlorine mixture. Once a seawater sample has been taken, chlorine will rapidly disappear from the sample through demand reactions. When the sample is subsequently poured onto a cultivating medium and left to incubate in an oven, the medium, the temperature and the time will destroy any residual chlorine. This effect would not be seen in an industrial application, since the BFCC treatment would be applied on a continuous basis.

The 5 ug/l of added copper is well within the 23 ug/l water quality criteria level set by the Unites States Environmental Protection Agency (EPA), and one third of that emitted by a thermal desalination plant (8). The 200 ug/l (0.2 ppm) added chlorine compares favourably with the typically 1 ppm concentrations added by conventional chlorination systems. A reduced chlorine requirement has capital expenditure, maintenance and environmental advantages. It also reduces the risk of PA membrane damage, and reduces the requirement for feed water dechlorination. The use of sodium bisulphate (SBS) as an oxygen scavenger and/or chlorine scavenger can provide a food source for sulphate reducing bacteria (SRB) which reduce bisulphite to sulphide ions (9).

It is may not be necessary to achieve 100% bacterial kill at the pre-treatment stage. Dead bacteria are more likely to release organic material into the water through autolysis, thereby providing a potential food source for bacteria downstream. Frequent backwashing of filters would help in this respect, minimising the opportunity for bacterial degradation in the sand filters. Keeping in mind that backwashing frequency must remain within appropriate limits as dictated by other process restrictions. The data for bacterial numbers before and after the sand filters highlight the effectiveness of filtration in controlling bacterial numbers.

It was not expected that the BFCC system would have any direct effect on filtration, however increased frequency of filter backwashing was required when using the BFCC system. In view of the importance of pre-treatment in removing bacteria, the coagulative effect of the BFCC system is a useful feature which is likely to lead to a reduction in the usage of coagulants such as $FeCl_3$.

The results reinforce the potential benefits of target dosing copper and chlorine. Target dosing involves introducing small doses of copper and chlorine treated seawater at different stages in the RO process (Fig. 5). Results of second injection of low dose of copper and chlorine after pretreatment will be reported in a later work. This is preferable to adding high (parts per million) concentrations of copper / chlorine at the seawater intake, for a number of reasons.

1) The addition of high chlorine concentrations in the raw seawater feed will result in increased available organic carbon (AOC) concentrations, which will then predispose the Reverse Osmosis (RO) membrane to bacterial fouling (7). Dosing with small copper / chlorine concentrations at key points in the process is the preferred water treatment strategy (see Fig. 5).

2) Copper introduced into the raw seawater is likely to be scavenged in the coagulation / filtration phase. Any residual chlorine produced oxidant, (CPO), will most likely be eliminated during filtration, especially if an activated carbon filter is in use. Therefore, it is wasteful to try and control biological fouling with one large copper/chlorine dose at the intake and/or ahead of the pretreatment.

3) The use of a number of low copper / chlorine doses at strategic points in the process train minimises the possibility of chlorine induced polyamide (PA) membrane damage. It may also be possible to prevent fouling without having to add a reducing agent (eg. sodium metabisulphite) to destroy excess chlorine prior to filtration in the membrane stack.

4) One of the main difficulties in maintaining an effective antibiofouling dose is the short half-life of chlorine. However, filtration removes organic matter, which represents a major part of the chlorine demand of the seawater. The more the chlorine demand is reduced, the longer the chlorine will last, and the easier it will be to maintain stable concentrations of copper and chlorine in subsequent stages of the RO process. Chlorine concentrations of 20 ug/l can be used in conjunction with 5 ug/l of copper just before the permeator, in comparison with concentrations of 200 ug/l of chlorine used at the inlet of the system (10).

5) The introduction of chlorine into natural waters can give rise to the formation of trihalomethanes (THMs) and other halogenated organics (11). These are persistent molecules, some of which are carcinogenic. There is concern about their accumulation in the food chain. Efforts are under way in a number of countries to limit the discharge of chlorine into fresh water and marine environments. The use of low initial concentrations of chlorine in conjunction with copper will help to minimise the concentrations of THMs produced. This would have a tangable effect if organic matters are removed by coagulation/filtration process ahead of copper/chlorine injection. In view of the fact that small halocarbons are poorly rejected by some membranes (12), any measure which will reduce overall chlorine usage in the RO process must be attractive.

CONCLUSIONS

Biological fouling in Gulf desalination plants may be controlled with low concentrations of copper and chlorine. the technology offers a number of benefits over other antifouling technologies. Due to the relatively small masses of copper and chlorine required, the size of generating plant employed is small compared to other systems. Environmental impact is minimised through reduced copper and chlorine emissions. The formation of trihalomethanes is minimised, which is important both in terms of carryover into potable water, and the impact of rejected brine on marine ecosystems. The use of low chlorine concentrations minimises corrosion. The need for dechlorination prior to the RO membrane would thus be eliminated. Reduced chlorine concentrations may also improve filter performance, thereby improving the efficiency of the pre-treatment stage, and possibly resulting in a reduced requirement for other coagulants. Of interest to RO operators is the concept that reduced chlorine usage will remove the possibility of chlorine induced bacterial aftergrowth on RO membranes due to the formation of assimilable organic carbon (AOC).

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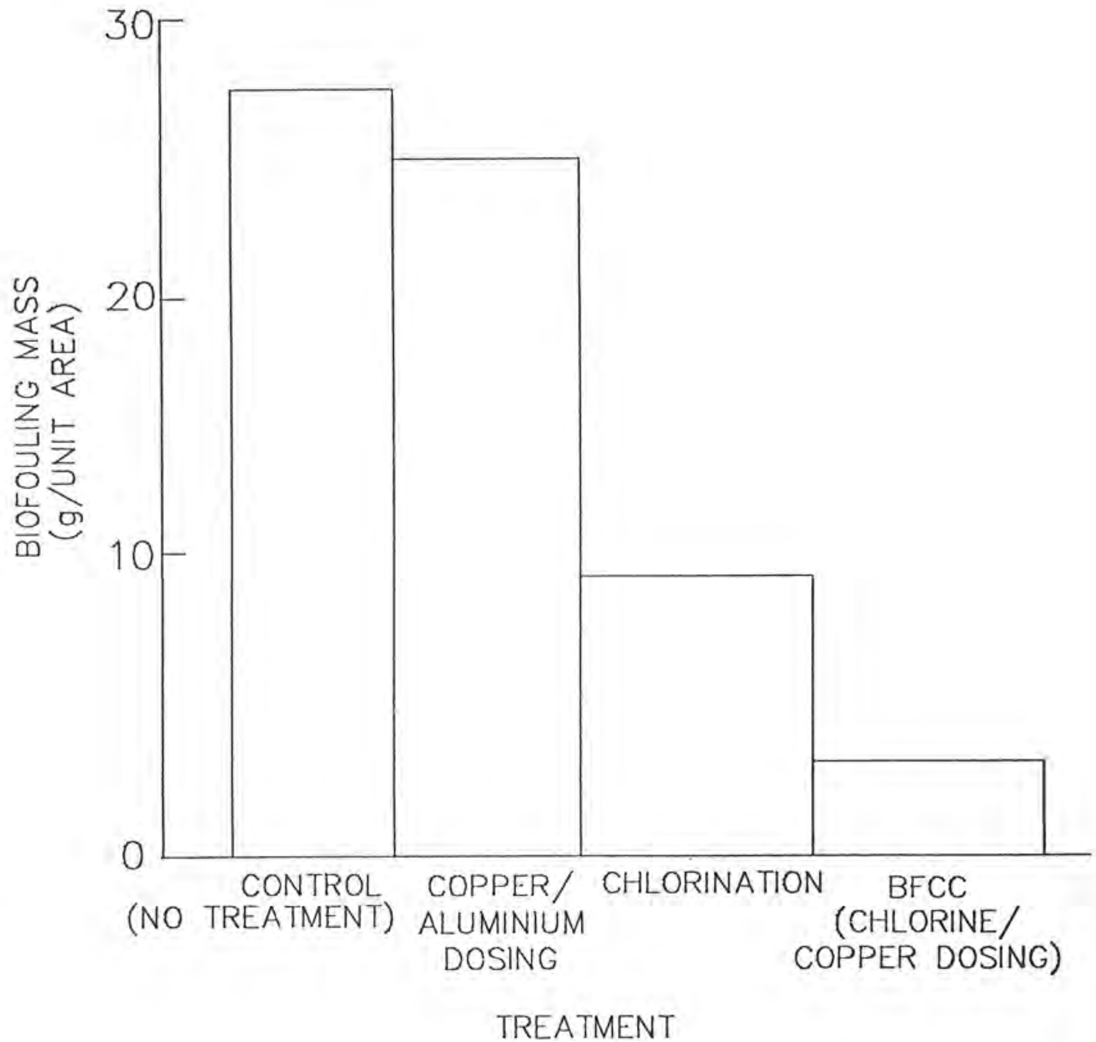

Fig. 1 Results of a macrofouling experiment using a mild steel test rig after 4 months. Copper dosage was at 35ppb in the copper/aluminium treatment; total residual chlorine was at $200\mu g \cdot l^{-1}$; $5\mu g \cdot l^{-1}$ copper and chlorine at $20\mu g \cdot l^{-1}$ were used for the copper/chlorine (BFCC) treatment.

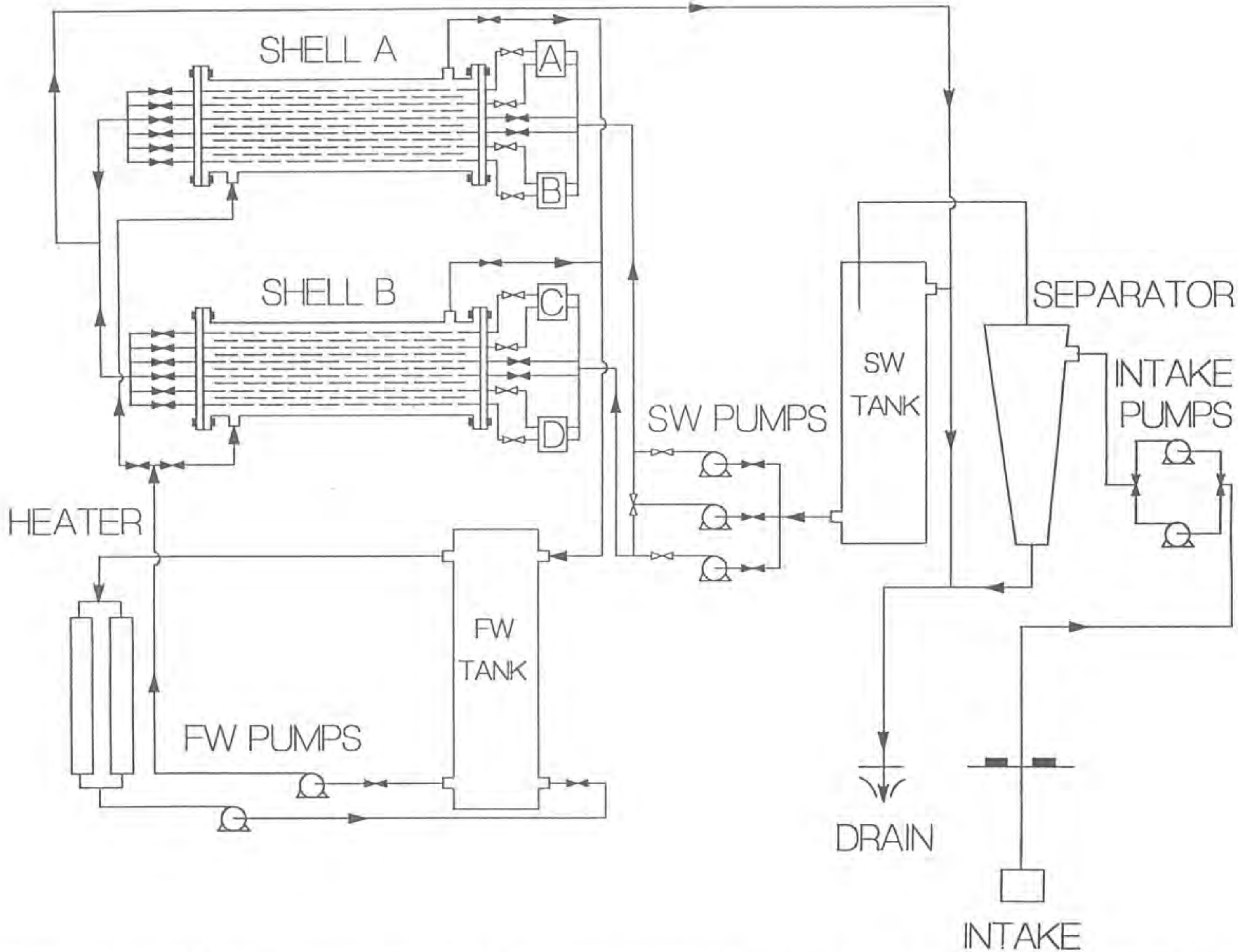

Fig.2 Schematic of US Electricity Power Research Institute (EPRI) heat exchanger test facility. Shells A and B contained 7HE tubes. A, B, C and D contianed the copper and chlorine electrodes.

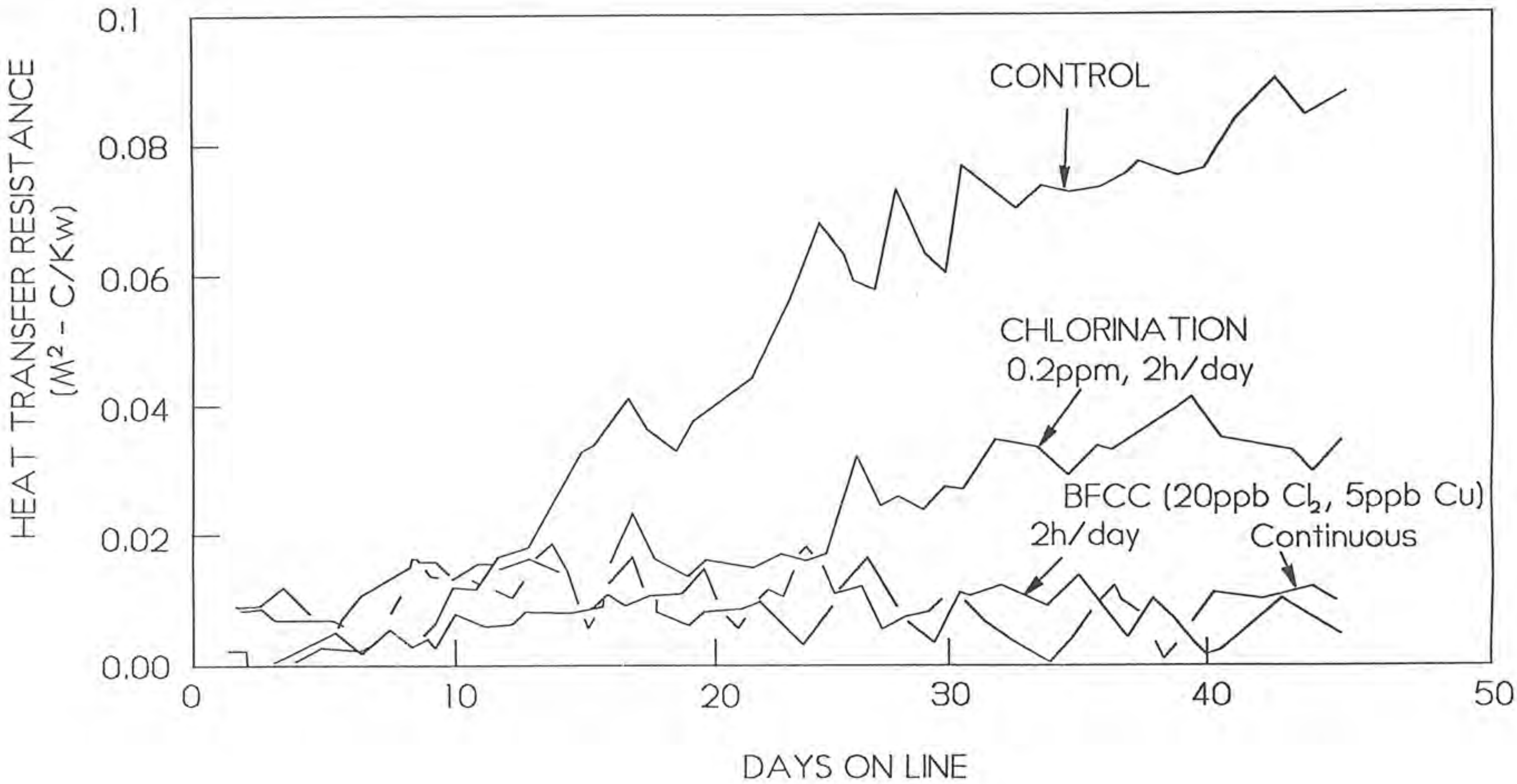

Fig. 3 Microfouling experiment test results, using the EPRI test rig to compare conventional chlorination (200 mg · 1^{-1} for 2h · d^{-1}) and BFCC copper/chlorine (5 ppb copper/20 ppb chlorine) treatments.

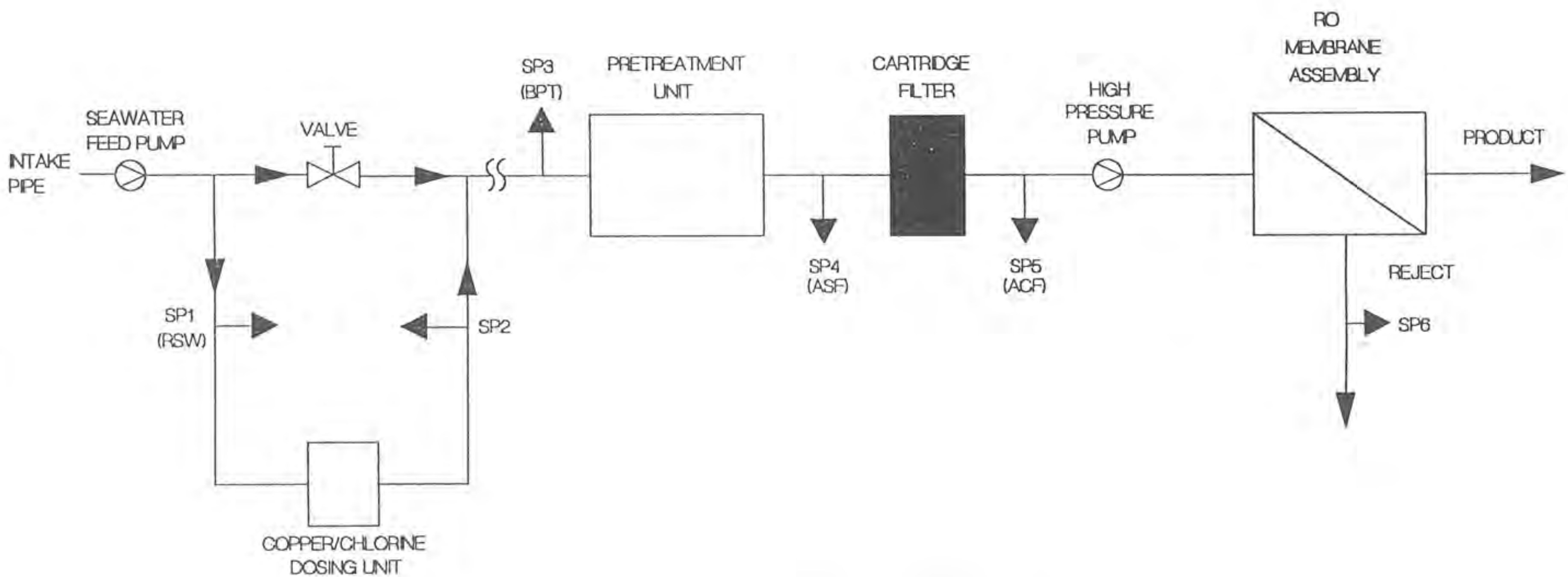

Fig. 4 Layout of SWCC experimental Seawater Reverse Osmosis (SWRO) pilot plant, showing Sampling Points (SP). See text for legend.

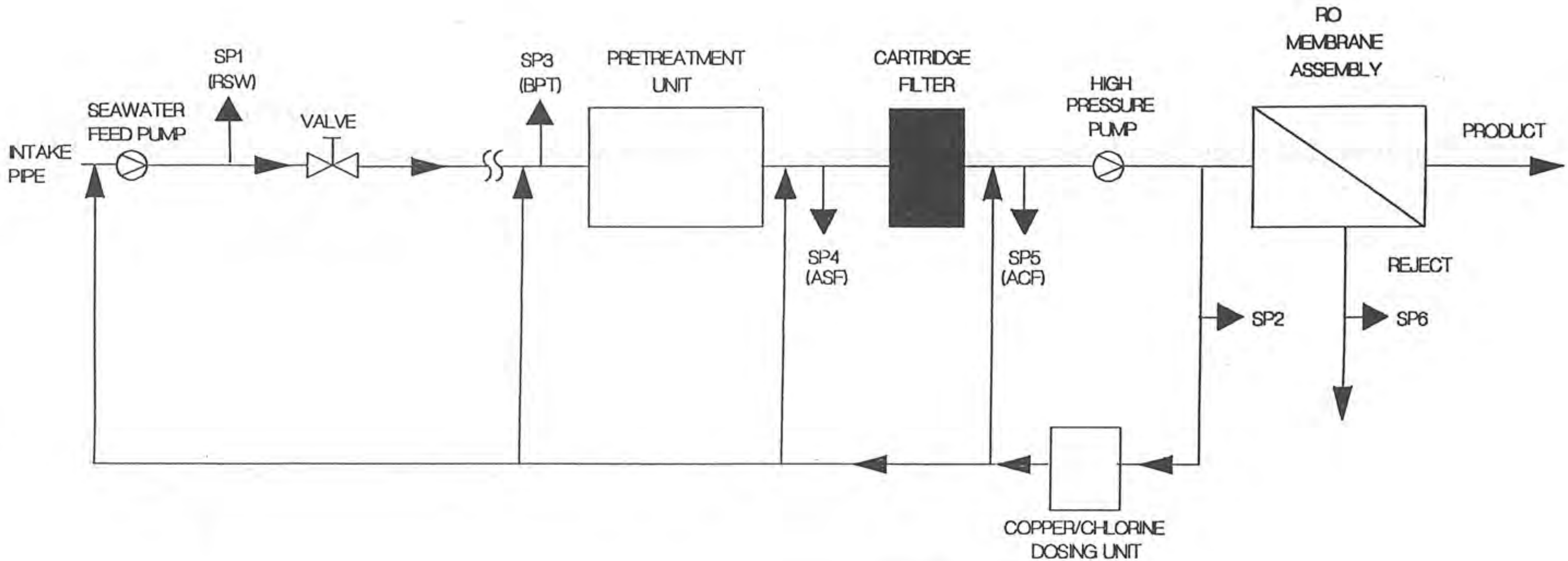

Fig. 5 Targeted copper/chlorine system.

Session - 4
Corrosion and Scale Cleaning in Water Utilities

Chemical Cleaning of Doha East Boilers

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CHEMICAL CLEANING OF DOHA EAST BOILERS

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Abstract

The internal surfaces of boiler water side components accumulate deposits even though standard water treatment practices may be followed. Excessive deposits impair heat transfer or could ultimately cause failure. Hence, chemical cleaning is an essential option so that to remove these deposits and to restore the thermal efficiency. The decision to chemically clean or the selection of the cleaning method can be made by considering the following factors:

1. Characteristics and quantity of deposit,
2. Boiler design-type, flow, operating temperature and pressure,
3. Adequate solvent based on deposit analyses,
4. Method of spent solvent disposal,
5. Case of application, and
6. Cost.

So accordingly, the need of acid cleaning for some Doha East boilers are considered due to the representative tube samples from boiler furnace which are analyzed and tested at Global Corrosion Consultants

Laboratories in U.K. Thus, the analyses shows boilers 4, 5, 6 and 7 should be chemically cleaned.

1. INTRODUCTION

The boiler tubes usually got internal scaling due to presence of dissolved solids in the water and corrosion product. The corrosion products usually contain a predominating amount of magnetite and amount of a ferric oxide and copper oxide. While dissolved solids are traces of inorganic salts such as calcium and magnesium phosphates.

These scales if not removed, it will accumulate to the extent of causing tube failure due to over heating. [1] Therefore, chemical cleaning is an important option to get rid of these scales and deposits. So the need of acid cleaning for Doha East boilers (1-7) due to the representative tube samples from boiler furnace which are analyzed and tested by Global Corrosion Consultant (GCC) Laboratories in U.K., to assess the need for cleaning and to make recommendations for the most suitable solvents and procedures. Thus, table (1) shows Doha East boiler tests results by GCC laboratories. It is clear that boilers 4, 5, 6 and 7 should be chemically cleaned due to high oxide thickness into he internal boiler tubes.

In turn, tenders were invited from qualified contractors for the chemical cleaning of these boilers by Ministry of Electricity and Water (MEW). Moreover, MEW requested from the contractor to provide the most suitable solvent and effective chemical process for removal of the scales from the boiler tubes. Also, tube samples, from each boiler, were submitted to the winner contractor for further analysis.

Table (1) : Mean Oxide Thickness on Individual Tubes of Doha East Boilers

Boiler No.	Mean Typical Thickness (μm)	
	Heat Side	Unheated Side
1	48	23
2	41	22
3	38	17
4	77	18
5	183	29
6	115	41
7	138	29

Chemical cleaning of Doha East boilers were carried out by the following steps:

2. CHEMICAL CLEANING PROCEDURE

2.1 First Flushing

Preparatory to acid-cleaning, a boiler was brought down and the tubes washed with water at a temperature of around 40°C to remove loose debris and dirts from the system and to check the system tightness. At the beginning all drains remain open until the flushing water becomes clear. The furnace tubes were flushed by using the autocirculation method injecting air. Then, the system was drained completely. The super heater was filled up with water through attemperator.

2.2 First iron removal by hydrofluoric acid

The boiler was filled up with a solution of hydrofluoric acid 1% at a temperature 60°C. Lithsolvent 609-S inhibitor was added at a concentration of 0.2 %. The cleaning solution circulation was achieved by the introduction of nitrogen (N_2) into down comer tubes as shown in Fig. 1 (which known as autocirculation). The solution was drained when the iron content reaching a constant level.

2.3 Flushing

The boiler was flushed with hot demineralized water (40°C) until reaching a conductivity below 50 μs/cm.

2.4 First Copper Removal

The boiler was filled up with a solution of 0.5% citric acid and ammonia to increase pH value up to 9.5. Hydrogen peroxide was added as oxidant.

The solution was circulated by injection of air through down comer tubes. After reaching a constant level of copper in that solution the whole system was drained.

2.5 Flushing

The boiler was flushed with water to a low pH. Then, the boiler was filled up with hot water (80°C) in order to heat the iron masses of the boiler tubes.

2.6 Second Iron Removal by Hydrochloric acid

The boiler was filled up with a mixture of inhibited acid and warm water to give a cleaning solution of 3% in strength and 60°C in temperature. The circulation of acid was continued until a stable iron level was attained.

2.7 Flushing

After draining the acid solution, the boiler was flushed by water to a low conductivity.

2.8 Second Copper Removal

This was done in the same way as the first copper removal stage.

2.9 Final Flushing

The boiler was flushed with ammonia solution at a pH-value of 10.3 and then drained.

2.10 Effluent Disposal

All the cleaning solution disposal were neutralized to the pH-value of 6 to 8 by adding line into the discharge line.

3. CHEMICAL PROCESS CONTROL

3.1 Chemical Monitoring

The following methods of analysis should be employed during various stages of chemical clean. The sample must be filtered before any analysis carried out.[1]

- pH
- Concentration of hydrochloric acid or hydrofluoric acid (*)
- Total citrate
- Free citric acid
- Alkalinity
- Sodium
- Total iron (*)
- Ferric iron in citric acid or HCl acid
- Copper (*)
- Temperature (*)

Table (2) represented the analysis result of iron content in the 2nd iron removal stage.

3.2 Method of Circulation and Temporary Connections

The cleaning solutions were circulated through the economizer by acid pumps, and through the furnace tubes by auto circulation method. The autocirculation method which was developed for treatment of natural circulation boilers can be achieved by injecting gas or air at suitable points through down comer tubes as shown in Fig. 1.

*** Doha East Lab. measured parameters.**

Table (2) : Analysis Result of Iron Content in the Second Iron Removal Stage

Time	Temp.	pH	Acid Strength	Total Iron
14.18	60	1.5	2.85	2225
14.30	60	1.8	2.8	2000
14.45	60	2.0	2.65	1900
15.00	59.8	2.0	2.55	1900
15.15	59.8	2.2	2.48	1950
15.30	59.8	2.2	2.46	2000
15.45	59.7	2.2	2.40	1800
16.00	59.6	2.3	2.38	1800
16.15	59.6	2.3	2.38	1900
16.30	59.6	2.3	2.38	1950

4. OXIDE / SCALE REMOVAL

The oxide layer or internal scale inside boiler tubes is predominantly magnetic (Fe_3O_4) but usually with a number of other substances incorporated, principally copper and its oxide, which requires a further stage of chemical clean. For the removal of iron oxide deposits, acids of a certain strength are typically used to remove these deposits. But, neither may be satisfactory when copper or copper oxides are present. Thus, the standard formulation for this purpose is 1% citric acid, plus ammonia to pH 9.5, plus an oxidant. Copper oxides are soluble in ammonia (to form the cupric or cuprous ammonium complexes), but copper metal is not and so requires prior oxidation to convert it to the oxide. A variety of oxidants has been used such as sodium bromate, nitrite, hydrogen peroxide and even air. Depending upon the copper content of the deposit, either one or two stages of copper removal are applied. When the copper oxide exceeds 5%, two stages are required. [2]

CONCLUSION AND RECOMMENDATION

(1) At the beginning of annual maintenance of the boiler, tube samples of one meter length to be cut from furnace walls by operation and maintenance departments and take into account;

   (i) Design and measured metal temperature distribution.

   (ii) Furnace flux distribution.

   (iii) Boiler operational history.

   and whenever possible, the results of the current boiler remnant life assessment shall be used in selection of samples. [3]

Then, the tube sample to be sent to the manufacturer or any qualified laboratories for analysis of inside deposits and to get their advise about boiler cleaning.

(2) If any boiler found in need of cleaning, a program should be established and the cleaning firms to be asked for quotations. The summary of boiler design should be submitted to these firms with the results of tube samples analysis as summarized below:

- Boiler Type
- Boiler Capacity
- Design Pressure
- Pressure at Superheater Outlet
- Temp. of Superheater Steam
- Feed Water Temperature
- Air Temperature
- Firing System
- Draught System
- Quantity of Water in Boiler Units
- Total running hours since commissioning or since last acid cleaning.
- Analysis results of deposit inside the sampled tubes.

(3) Chemical cleaning can be classified into three categories:

(i) Iron oxide removal which is the main stage in most chemical cleaning process. Acids are widely used for this stage either alone or in mixture with other chemicals. [2]

(ii) Copper oxide removal stage which can be removed by 1% citric acid, plus ammonia to pH 9.5 and oxidizing agents. These agents reacts with metallic coppers to form cupric oxide. This oxide dissolves in the ammonium solution.

(iii) Passivation stage means treatment applied to the steel parts of the steam and water cycle with the intention to produce a continuous, coherent, and corrosion-resistant layer of magnetite on the surface. This stage is essential to be done after acid cleaning if the boiler will not be in service.

(iv) Chemical cleaning process should be controlled at each stage by station laboratory during the operation. Regular analysis for metal content (particularly iron and copper) of the circulating cleaning solution will indicate the rate of dissolution of the metal oxides, and when the iron/copper concentration become stable for process is considered to be completed.

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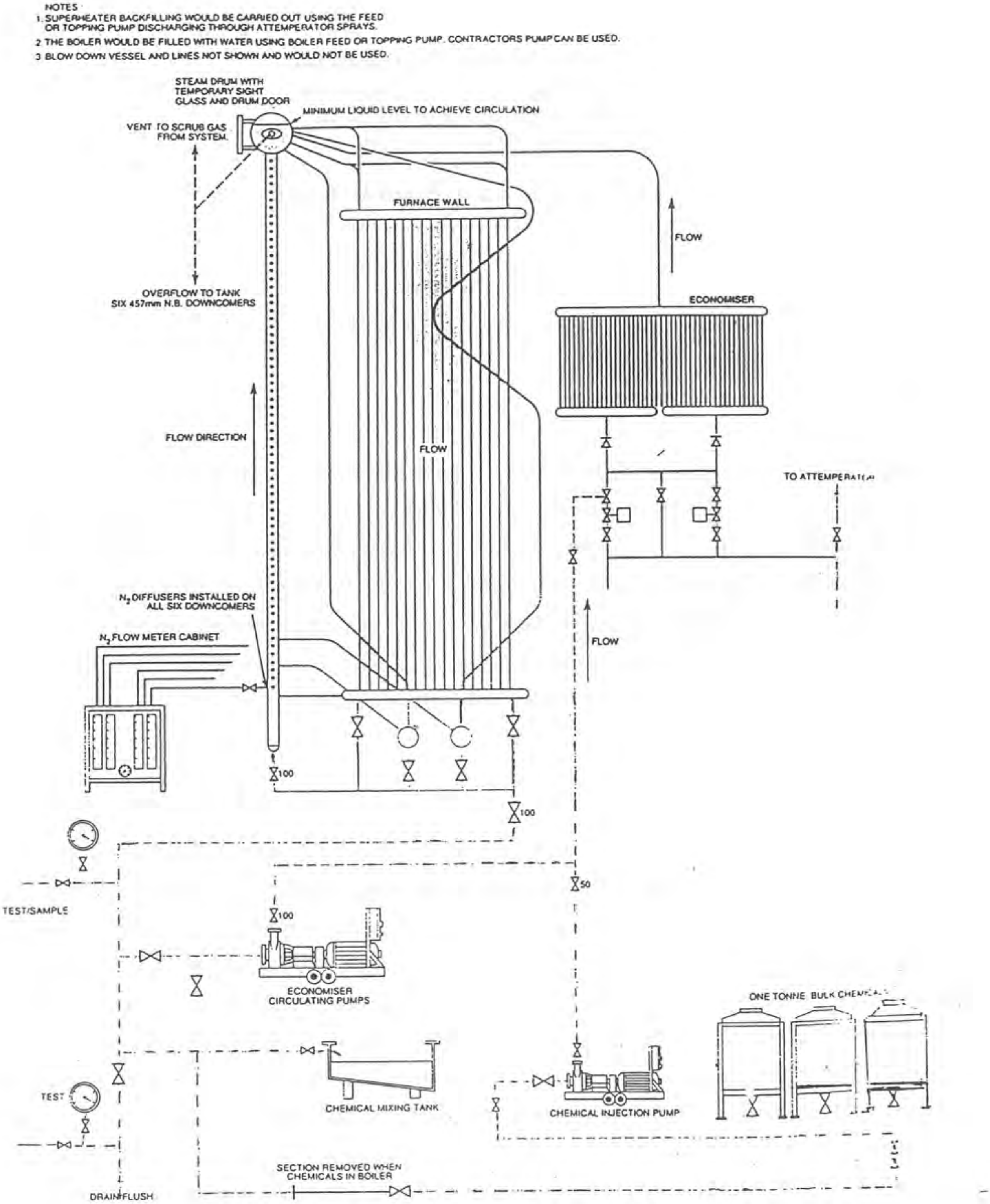

Fig.1. Gas circulation of chemical cleaning solutions in a natural circulation boiler.[1]

Decomposition of Hydrazine in Preserved Boilers

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DECOMPOSITION OF HYDRAZINE IN PRESERVED BOILERS

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ABSTRACT

A detailed study was carried out in the R&D Center laboratory at Al Jubail using bench top experimental set up to investigate the decomposition of hydrazine in preserved boilers. Effects of temperature and the presence of trace concentrations of metal oxides on the decomposition of hydrazine were determined by monitoring hydrazine and ammonia concentrations in hydrazine solutions kept in contact with trace metals for periods of 1-60 days. This report describes the details of the study, discusses the results and proposes some recommendation for consideration during boiler preservation.

INTRODUCTION

Hydrazine and sodium sulfite are the most widely used chemical oxygen scavengers for boiler feed water. Though sodium sulfite reacts with dissolved oxygen at a faster rate even at low temperatures it increases the dissolved solids in boiler water and causes loss of thermal energy due to the requirement of increased frequency of blow downs to reduce the solid contents. Decomposition of sodium sulfite also becomes significant at boiler pressures above 950 psig and hence not used in high pressure boilers. In comparison hydrazine does not contribute to the dissolved salt contents of the boiler water. Hydrazine is slightly volatile but starts to decompose to ammonia at high temperatures reducing its efficiency as oxygen scavenger. In comparison with sodium sulfite hydrazine reacts with oxygen slowly and the kinetics of reaction increases as temperature increases.

Hydrazine may be lost from the boiler water by a variety of reactions as shown below:

(a) In the presence of oxygen,

$$N_2H_4 + O_2 \rightarrow 2H_2O + N_2 \tag{1}$$

(b) At high temperature or in the presence of catalysts:

$$3N_2H_4 \rightarrow N_2 + 4NH_3 \tag{2}$$

Or

$$2N_2H_4 \rightarrow N_2 + 2NH_3 + H_2 \tag{3}$$

(c) In the presence of metal oxides,

$$N_2H_4 + 6Fe_2O_3 \rightarrow 4Fe_3O_4 + 2H_2O + N_2 \tag{4}$$

$$N_2H_4 + 4CuO \rightarrow 2Cu_2O + 2H_2O + N_2 \tag{5}$$

Considering some of the disadvantages of sodium sulfite and hydrazine as noted above various all volatile amines such as diethyl hydroxylamine, cyclohexylamine, morpholine and some filming amines such as octadecylamine have been successfully used for high pressure boiler water treatments.

Since 1982 in all boilers under operation in the Phase-II of SWCC Desal/Power plants in Al Jubail, hydrazine has been successfully used as oxygen scavenger with coordinated phosphate treatment to control pH and scale formation. Phase-II A and B areas have 5 BTG units each and two spare boilers, one on each area. The spare boilers are usually kept under long term preservation under wet storage condition. The boiler parts are filled with DM water containing 300-500 ppm hydrazine and a nitrogen atmosphere is maintained above the water level. Hydrazine concentration is monitored periodically and maintained at the required concentration by topping up when needed.

During a prolonged period of Jan. 89 to June, 92, boilers #66 and 86 in Phase-II A and B, respectively, were under long term preservation. Periodic analysis of the boiler water revealed a dramatic drops in hydrazine concentrations. Frequent addition of hydrazine was required to maintain its concentration at the required level. Fig. 1 indicates the variation of hydrazine level in boiler #66 during this period. Despite frequent topping up with fresh addition of hydrazine (indicated by arrows in Fig. 1) loss of hydrazine continued to occur. Analysis revealed the presence of ammonia in substantial quantities in the boiler water.

The incidence caused serious concern as hydrazine is not expected to show such rapid decomposition at low temperatures unless an active surface or ionic impurities that can catalyze the decomposition reaction is present. Traces of copper and iron, normally found in the solution at less than 10 PPB, are not considered to be sufficient to cause decomposition of such large amounts of N_2H_4. Source of Cu is from the corrosion of Cu/Ni heat exchanger tubes used in H.P. heaters as well as the brine heaters of the SW desalination plants. To overcome the problem efforts were made to find an alternative chemical for long term preservation. Several proprietary chemicals, claimed to be "hydrazine alternatives", were evaluated based on informations from the suppliers. A chemical with the trade name "Betz Layup 1", manufactured by Betz, USA was tested in boiler #66. For a period of 4 months (Aug-Dec. 1991, see Fig. 1) the boiler was preserved with 2000-3000 ppm of Betz Layup-1 and nitrogen blanketing. Two corrosion coupons (one mild steel and one copper) were installed in the steam drum. Periodic analysis of DO, iron and copper in the drum water was carried out during this period. After 4 months the boiler was drained and the steam drum was inspected. The condition of the drum was found to be good and coated with a black film of magnetite. The corrosion rates determined with test coupons were very low (<0.1 mpy).

After draining Betz Layup-1 the boiler was subsequently preserved in hydrazine (<300 ppm). An interesting observation was subsequently noted - after the Betz Layup-1 treatment, hydrazine concentration in the boiler water did not show significant reductions, as was the case before the treatment. However, copper contents in the preservation water showed significant increases (see Fig. 1).

The incident was reported to the SWCC O&M Seminar conducted at Al-Jubail in April 1992 by Mr. Ghazzai Al Mutairi, Plant Chemist, Al Jubail Plant and a co-author of this report. The report generated considerable interest and the Engineering Department, SWCC (EP) suggested in their letter dated 5 July, 1992 that the R&D Center conduct further studies to confirm the findings and suggest possible alternative measures. R&D Center has subsequently conducted a series of tests using bench top experimental set up. The details of the tests and the results are described below:

EXPERIMENTAL

Degradation of hydrazine at various conditions, expected to be present in Al-Jubail desal power plant boilers, were studied in the R&D laboratory.

(a) Test Procedures

300 ml volumes of hydrazine solution at initial concentrations of about 300 ppm prepared in ultrapure demineralized water were used during the studies. Amber colored bottles, filled with the solution without leaving voids on the top, were maintained at the experimental temperatures in an incubator for various contact periods. Hydrazine, ammonia and pH were monitored after fixed contact times under the following conditions:

(1) Hydrazine kept at 45°C for 1 - 60 days to obtain reference data. 45°C was chosen for all experiments so as to bring the temperature closer to the steam drum condition that may be prevalent in summer.

(2) Hydrazine containing 100 ppb Fe^{+++} ions as ferric chloride. Temp = 45°C, contact time = 1 - 60 days.

(3) Same as above, but with 100 ppb Cu^{++} ions, as cupric chloride, in place of Fe^{+++} ions.

(4) As above, but the solution was kept in contact with about 10 gr of carbon steel turnings which was allowed to corrode for 3-4 days in the open air before the experiment.

(5) Same as in 4. But precorroded Cu/Ni 70/30 turnings were used instead of iron turnings.

(6) Experiments described in 4 & 5 above were repeated at 20, 30 and 45°C, and at 14 days contact time to study the effect of temperature on the decomposition rate.

(7) 50x25x2 mm specimens of carbon steel and Cu/Ni alloy, precorroded, were immersed in 1000 ppm Betz Layup-1 for one week. The specimens were

removed, rinsed with DM water and kept in contact with 300 ppm hydrazine in separate bottles for one week. Hydrazine decomposition, and Cu^{++}, Ni^{++} and Fe^{+++} ions in the solutions were determined.

(b) Analytical Procedures

Hydrazine was estimated iodometrically by titrating the sample against standard sodium thiosulfate using starch as indicator [Ref.1]. The titration was carried out after buffering the sample to pH 7.0-7.2 by the addition of sodium bicarbonate.

Ammmonia in hydrazine was estimated by nesslerization method after oxidizing hydrazine completely by hydrogen peroxide in boiling acid solution with iodine as catalyst [Ref.2]. The nesslerized sample was analyzed at 410nm using a Shimadzu UV-2100S UV- visible spectrophotometer.

Metal ions were determined by Graphite Furnace AA spectrometry, after making appropriate dilutions wherever necessary.

RESULTS AND DISCUSSIONS

Degradation of pure hydrazine at 45°C, in the absence of any catalysts, is shown in Fig. 2. As the solutions were kept in amber colored bottles and inside an incubator possibility of degradation due to ultraviolet radiation was excluded. While about 3% degradation was observed after 24 hrs, 25 and 40% of the original hydrazine was found to have decomposed after 30 and 60 days, respectively. Hydrazine could decompose through reactions 1-3, to give nitrogen and ammonia. But these reactions in the absence of O_2, catalysts etc. appear to be slow as found in Fig. 2. Monitoring of ammonia does not appear to provide quantitative data of the decomposition reactions. Though traces of NH_3 was detected in the test solutions concentrations were less than those expected by equations 2 and 3. This may also be due to the escape of some NH_3 during handling etc. and pH changes were too low to be of any significance.

Decomposition of hydrazine in the presence of ferric ions (Fe^{+++} = 100 ppb) is shown in Fig. 3.

The results show that decomposition of hydrazine is not catalyzed by the presence of Fe^{+++} ions, with losses of 3, 22 and 22%, after 1, 30 and 60 days contact times, respectively. The magnitude of decomposition is similar to that found in the blank experiments (Fig. 2). But in the presence of Cu^{+} ions (Fig.4) decomposition was found to be quite rapid and quantitative within 60 days. Here again the ammonia content did not indicate proportionate increase. But the presence of substantial amount of ammonia (21.7 ppm) after 60 days contact time indicates that the decomposition in the presence of cupric ions proceeds through a catalytic reaction.

Experiments with precorroded iron and Cu/Ni turnings are shown in Figs. 5 and 6 respectively. The decomposition in the presence of Cu/Ni turnings was much more rapid and quantitative than found in the presence of iron turnings. Both iron and Cu/Ni turnings exhibited better catalytic property than their respective ions in solution. It appears that the thin oxide films on the surface of the corroded alloy turnings provide a more favourable catalytic environment than the dissolved ions, though copper even in the ionic form exhibited strong catalytic property.

To study the effect of temperature on the decomposition reactions, N_2H_4 solutions were kept in contact with rusted iron and Cu/Ni turnings for 14 days at 20, 30 and 45°C. The results (Figs 7 and 8) show that the decomposition of N_2H_4 depends significantly on the temperature and increases proportionately as the temperature increases. The effect was quite dramatic in the presence of Cu/Ni turnings while in the case of iron turnings the increase was marginal.

Due to the instability of hydrazine used for preservation of the Al Jubail boilers, a new product, Betz Layup-1, was used for a period of time. After draining out this chemical, the boiler drum was inspected and was found to have a uniform black coating of magnetite. The boiler was then filled with hydrazine and subsequent analysis indicated that hydrazine degradation had significantly reduced. To check this, small test specimens of iron and Cu/Ni alloy, previously corroded by exposing to the atmosphere for a few days, were first immersed in Betz Layup-1 solution (1000 ppm) for a week, removed and rinsed in DM water, and then immersed in 300 ppm N_2H_4 for a week at 45°C. The results (Table-1) indicate that Betz Layup-1 treatment did not have

Table 1 : N_2H_4 Decomposition after Betz Layup Treatment

Specimens	Betz Layup Filtrate ppm	Analysis in Hydrazine Solution						
		N_2H_4 ppm		NH_3 ppm		pH		Metals ppm
		Initial	Final	Initial	Final	Initial	Final	
Cu/Ni	Cu=1.95 Ni=0.08	245 245	30 30	0.95 0.95	105.8 106.1	9.6	10.2	Cu=0.04 Ni=0
Carbon steel	Fe=1.15	245 245	196 196	0.95 0.95	40.9 41.3	9.6	10	Fe=0.28

noticeable effect on N_2H_4 degradation. While approximately 87% of N_2H_4 was found to be decomposed in the presence of Cu/Ni, only 20% degradation occurred in the presence of Fe. These results are similar to those obtained in previous tests conducted without Betz Layup-1 treatment. The results also indicate that Betz Layup-1 is slightly aggressive to the alloys, dissolving Cu, Ni and Fe from the specimens. In comparison, attack by hydrazine was milder, as indicated by much lower concentrations of the metal ions in the solution.

CONCLUSIONS

The studies indicate that both cupric ions in solution and CuO film on rusted Cu/Ni turnings significantly enhance the decomposition of hydrazine. Decomposition is much faster at 45°C than at lower temperatures. Though Fe^{+++} does not influence the degradation of N_2H_4, its oxides formed on the rusted carbon steel surface appear to have significant effects on the kinetics of decomposition. Wide variations in the ammonia concentrations did not help to suggest which of the several reactions proposed above play a prominent part in the decomposition. But the presence of high concentration of ammonia in N_2H_4 solution degraded in the presence of Cu^{++} ions and the oxides on the alloy turnings indicate that these species might be acting as catalysts rather than taking part in chemical reactions resulting into stable compounds such as Fe_3O_4 or Cu_2O as indicated by reactions 4 and 5.

A treatment with Betz Layup-1 did not improve the stability of hydrazine as indicated by our studies. In addition, this chemical appears to be aggressive to the alloys and should be used with caution. The improvement in the stability of hydrazine found in boiler #66 after Betz Layup-1 treatment may possibly be explained as follows:

Traces of Cu^{++} ions present in boiler water originating from the Cu/Ni tubes in the desal brine heaters or boiler H.P. heaters could deposit on the carbon steel internals of the boiler as copper is a nobler metal than carbon steel. As the boilers have been in service for a long period of time, thin layers of Cu^{++} or CuO might have been formed on the surface of the boiler parts that had acted as catalysts enhancing N_2H_4 decomposition. When the boiler was treated with Betz Layup-1 it acted in two possible ways : (a) removed all copper deposits from the boiler surface by dissolution or dispersion as indicated by high increase of copper content in the solution phase, thereby diminishing the catalytic activity; (b) Betz Layup-1 helped to form a passive magnetite layer on the metal surface, as indicated by the presence of a black film on the surface of the steam drum. Both of these will virtually stop decomposition of the N_2H_4 if proper nitrogen blanketing is maintained.

RECOMMENDATIONS

(1) Regular monitoring of N_2H_4, copper and iron contents in the preserved boilers needs to be continued.

(2) If decomposition of N_2H_4 is unusually high cleaning of the boiler internals should be considered to remove the corrosion deposits. A short period of Betz Layup-1 treatment appears to be useful in removing any possible Cu deposits from the boiler internals as well as to provide passivation. Further work in these lines should be done with caution and the R & D Center Should be involved to carry out detailed studies.

(3) If the boiler has been under long time preservation using N_2H_4, care should be taken to drain the boiler before returning to load, to prevent ammonia getting into brine heaters causing corrosion of Cu/Ni tubes.

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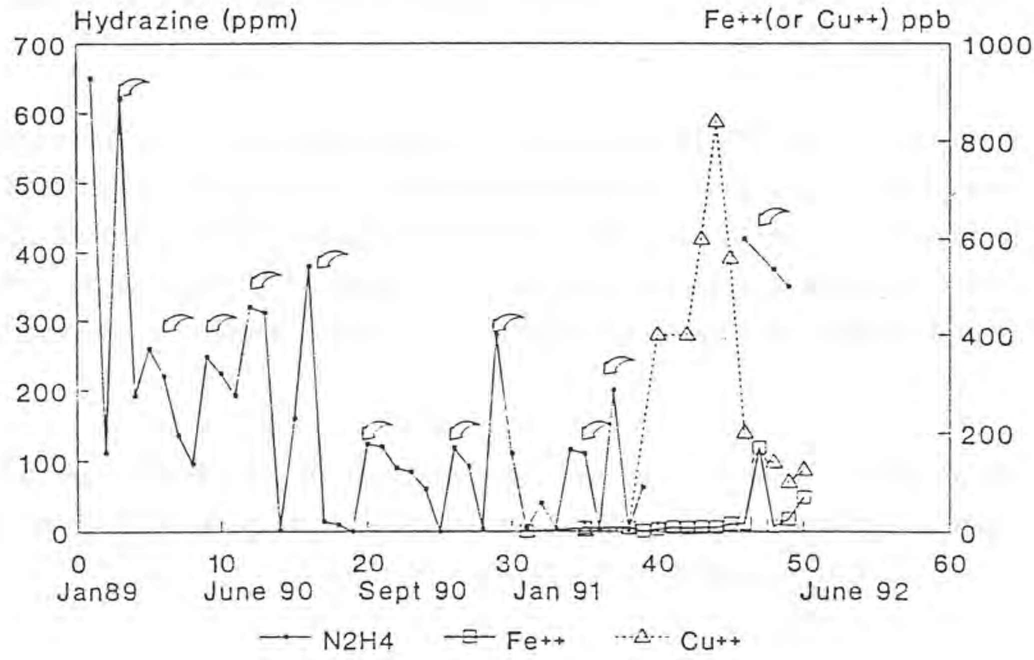
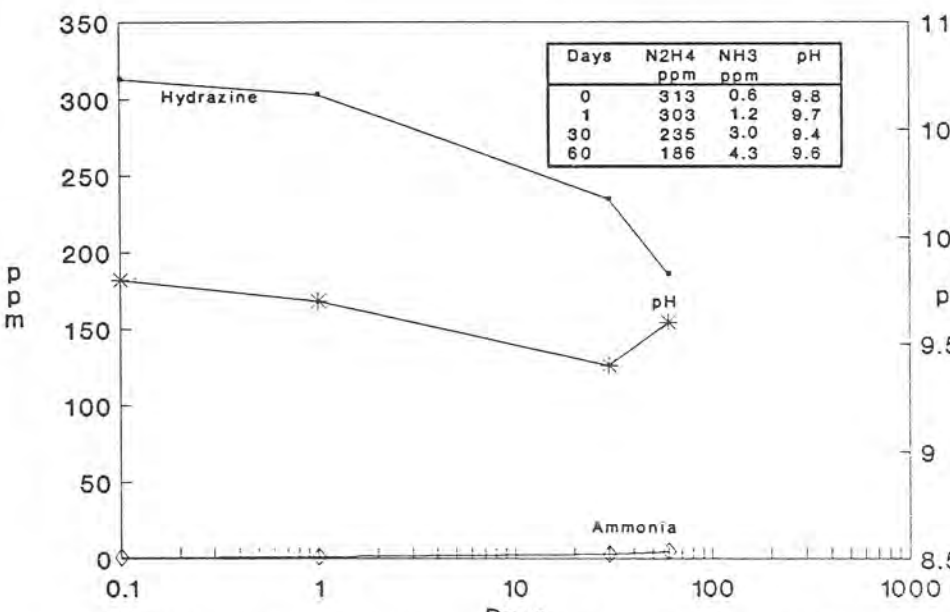
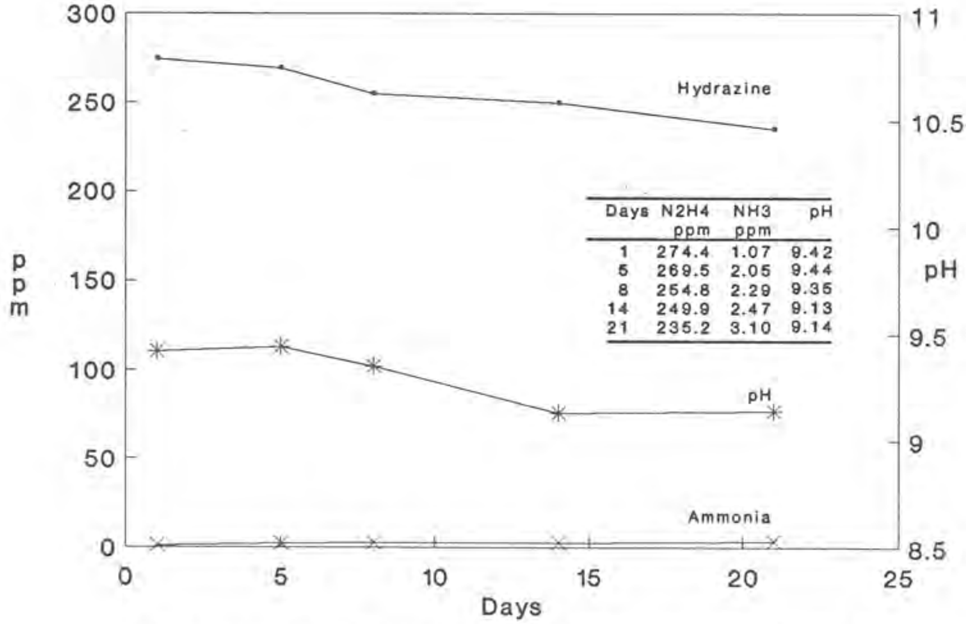


Fig.1 Analysis of N2H4 in Boiler #66, Al Jubail



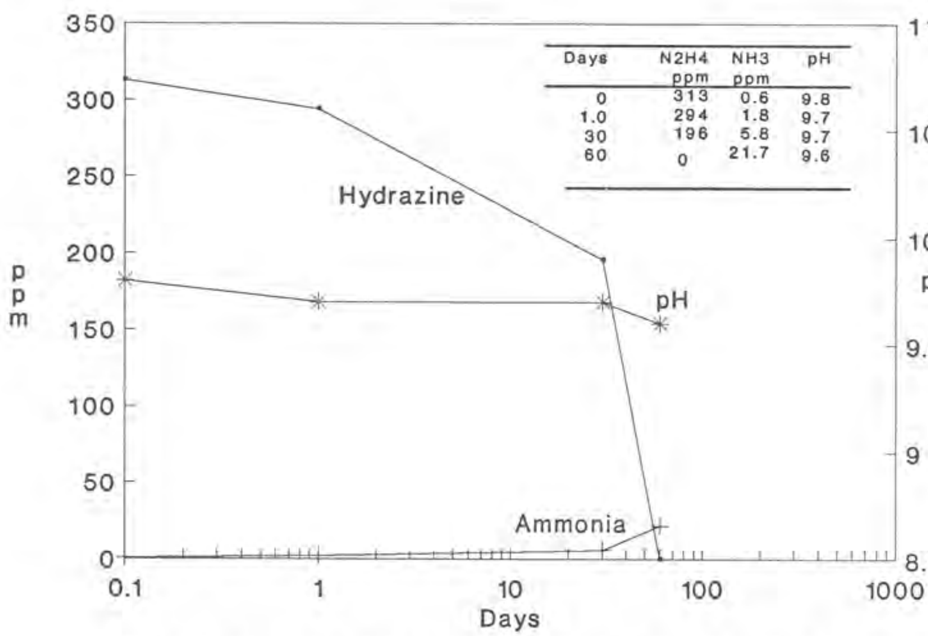
Days	N2H4 ppm	NH3 ppm	pH
0	313	0.6	9.8
1	303	1.2	9.7
30	235	3.0	9.4
60	186	4.3	9.6

Fig.2 Hydrazine degradation vs contact time-Blank (45oC)



Days	N2H4 ppm	NH3 ppm	pH
1	274.4	1.07	9.42
5	269.5	2.05	9.44
8	254.8	2.29	9.35
14	249.9	2.47	9.13
21	235.2	3.10	9.14

Fig.3 Hydrazine degradation vs contact time (Fe+++)45o C



Days	N2H4 ppm	NH3 ppm	pH
0	313	0.6	9.8
1.0	294	1.8	9.7
30	196	5.8	9.7
60	0	21.7	9.6

Fig.4 Hydrazine degradation vs contact time (Cu++) 45oC

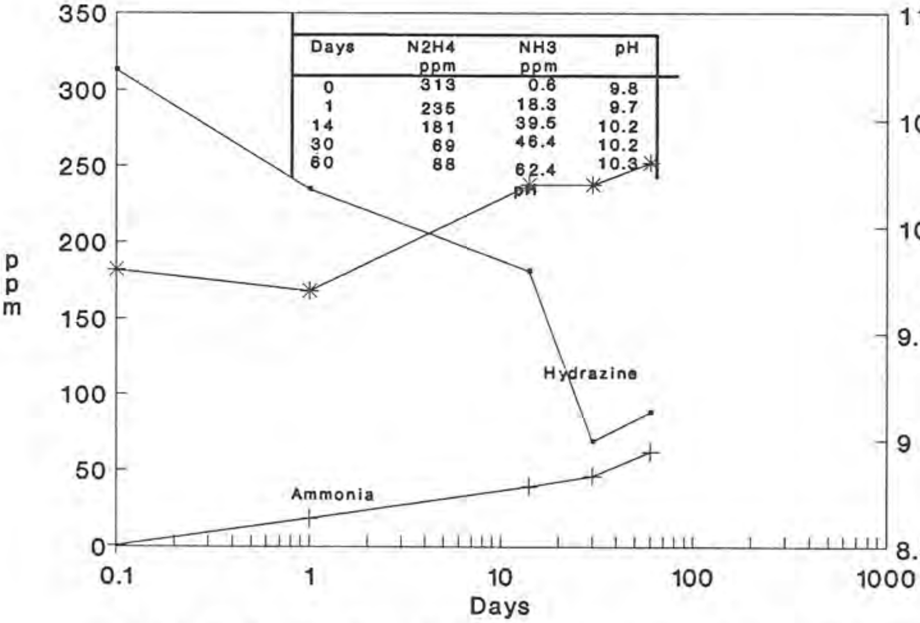


Fig.5. Hydrazine degradation vs contact time Fe (45oC)

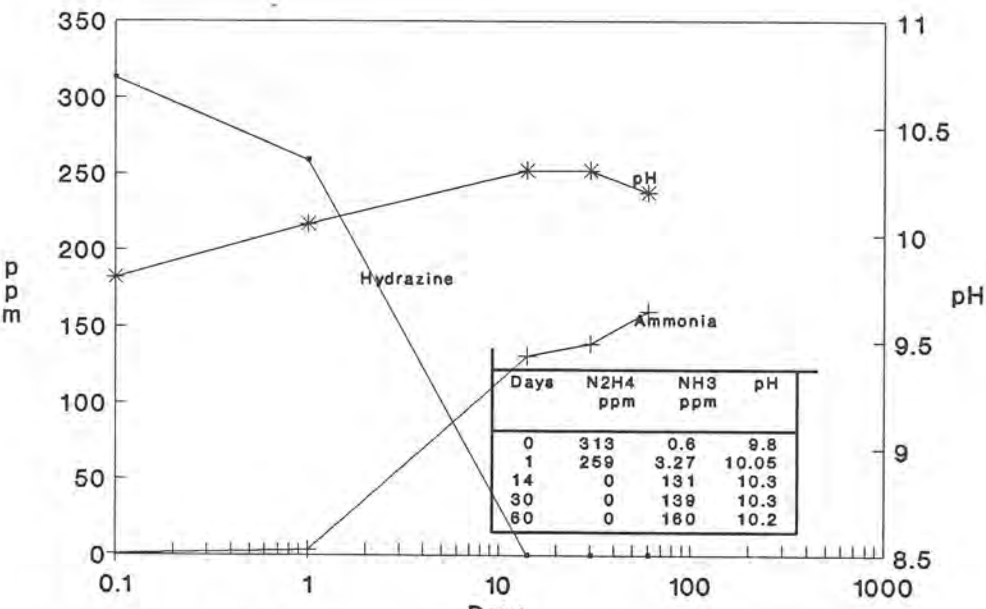


Fig.6 Hydrazine degradation vs contact time cu/Ni (45oC)

Fe

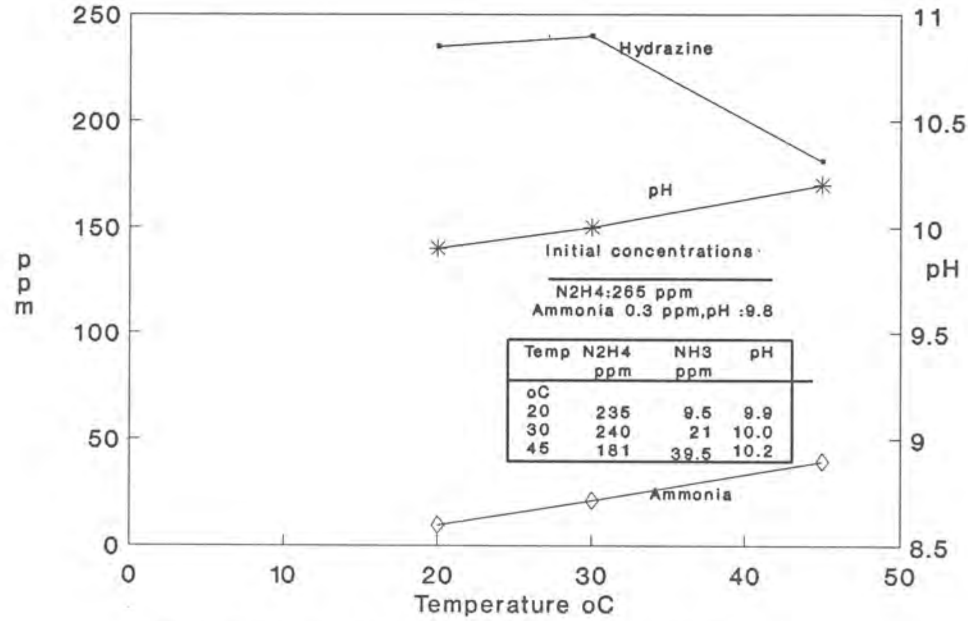


Fig.7.Hydrazine degradation vs Temperature

Cu/Ni

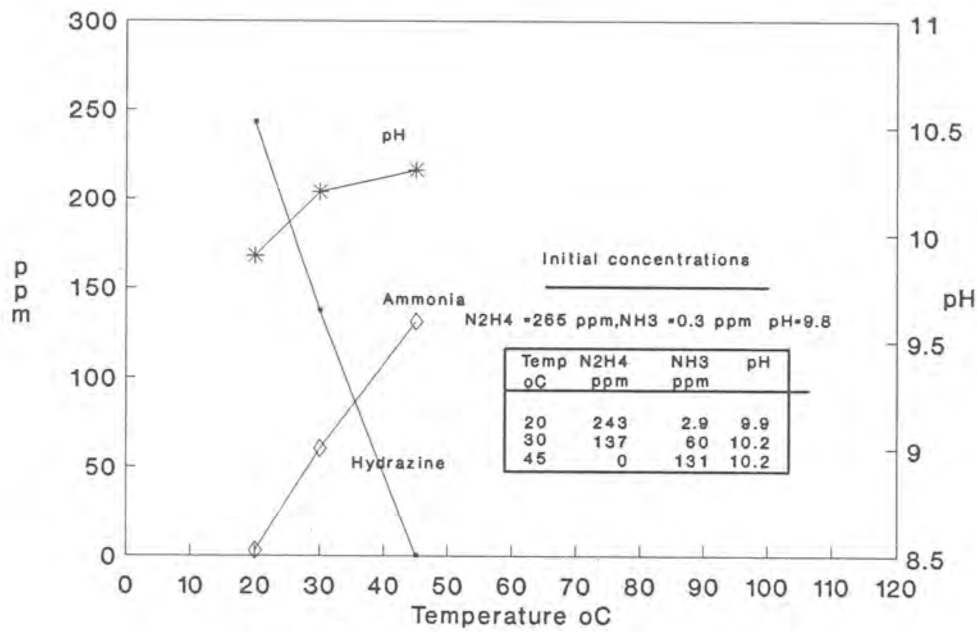


Fig.8.Hydrazine degradation vs Temperature

100% Solids Polyurethanes, State of the Art

Corrosion Protection for Water Systems

Howard Kennedy

100% SOLIDS POLYURETHANES, STATE OF THE ART CORROSION PROTECTION FOR WATER SYSTEMS

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ABSTRACT

The 100% solids polyurethane coating technology is an excellent alternative to the conventional epoxies that dominate the industry today for lining potable water storage tanks and pipes because it is safer, faster to apply, more economical and offers greater longevity. This paper covers the basic definition and history of polyurethane as well the wide range of applications in a variety of industries. Next, the paper covers the specific advantages of the 100% solids polyurethane coating systems compared to the conventional epoxies in terms of safety, application, performance and cost. Finally, the paper chronicles several case histories. These examples cover unique aspects involving service conditions, substrates and application conditions in both the Gulf and North America.

Key Terms: *100% solids, polyurethane, epoxy, lining, coating, tanks, potable water, tanks, pipes*

EXECUTIVE SUMMARY

The best overall protective coating for water tank and pipe internals, when considering performance, cost and safety, is 100% solids polyurethane. This simple chemistry, the reaction product between an isocyanate and a polyol, plays a vital role in many industries and in our every day activities. Polyurethane foams are used as thermal insulation in buildings and refrigerators. Mattresses and sofas use urethane foams as well. Shoe soles, elastic thread and car body panels are all different forms of polyurethane.

Recent developments in the technology have allowed polyurethane manufacturers to produce a 100% solids, instant setting, two component, highly cross-linked, additive-free polyurethane coating suitable for potable water contact that offers dramatic advantages over the conventional solvent based epoxy technologies. This new state of the art coating system is less toxic than the traditional epoxies for both the water user and the applicator. Not only do the unique curing and application features of the polyurethane systems drastically reduce the amount of time it takes to actually line the tank, they also cut the required curing time by over one-third. ***Based on experience in other industries and standard performance tests, polyurethane will outlast epoxies by 20% to 30%.***

The final reason that the polyurethane technology is playing a major role in protecting potable water tanks and pipes from internal corrosion is that even with all of these safety, application and performance advantages, the total applied costs are similar to those of comparable epoxies.

WHAT IS A POLYURETHANE?

Polyurethane is a very versatile thermoset plastic that was originally developed for military use by Otto Bayer in the late 1930's[1]. It is simply the reaction product of an isocyanate and a polyol (fig.1).

Figure 1

POLYURETHANE REACTION

Iso + Polyol = Polyurethane

$$R\text{-}NCO + R'\text{-}OH = R\text{-}\underset{}{\overset{\overset{\displaystyle H}{|}}{N}}\text{-}\overset{\overset{\displaystyle O}{\|}}{C}\text{-}O\text{-}R'$$

where R and R' are reacted groups

Today a wide variety of polyurethanes are used for many diverse applications. Flexible polyurethane foams are used to make bedding, sofas, cushions, carpet backs and car seats. Rigid foams are used for insulation in freezers, refrigerators and roofs. Many shoe manufacturers use the tough elastomeric polyurethanes for the shoe soles. The auto industry makes dashboards, bumper covers, mouldings and fenders out of polyurethane. The General Motor's Fiero sport car's fenders and door panels were all moulded out of polyurethane. There are many types of polyurethane coatings as well, ranging from bridge coatings to floor sealers to tank linings.

It is virtually impossible for anyone in the 20th century world to *not* come in contact with a variety of polyurethanes several times during a day. It is estimated that the average family probably owns somewhere between 10 and 50 kilos of polyurethane[2].

When the application demands a material with toughness and longevity, polyurethane is the most common answer.

WATER SAFETY

One of the most impressive characteristics of 100% solids polyurethane is its inherent safety feature. First, because the polyurethane is made by the reaction of two low viscosity liquids (iso and polyol), there is no need for solvents and hence, no VOC's. In recent years, primarily due to the demands of the auto industry, new isocyanates and polyols have been developed that allow the necessary performance criteria to be attained with the use of very few additives or catalysts. These developments further reduce the possibilities of dangerous materials leaching into the water.

The isocyanate and polyol are much safer than epoxies even in their liquid states. The oral toxicity (as indicated by the MSDS's of the suppliers of the raw materials for polyurethanes and epoxies) is two times greater for epoxies than it is for polyols and five times higher for isocyanates (fig.2)[3]. All isocyanates react with water. If unreacted isocyanate is leached or released into the water in any way, it simply reacts to form an inert, harmless polyurea solid.

Figure 2

ORAL TOXICITY

Raw Materials	Single Dose Toxicity (mg/kg body weight)
Polyurethane	
Isocyanate	10,000
Polyol	5,000
Epoxy	
Epoxy Resin	2,000
Amine Hardener	500 - 2,000

the higher the dose, the safer to material

source: MSDS's, Dow Chemical, ICI

Finally, the ultimate proof of safety is the American NSF International's (formerly National Sanitation Foundation) certification under the new NSF 61 standards. As of May 1, 1994, four polyurethane systems have been 'listed' by NSF International under NSF 61[4].

The 100% solids polyurethane systems for lining potable water tanks are safer than conventional epoxies because the polyurethane raw ingredients are less toxic than epoxies and because solvents are not required in the systems.

SAFETY

Polyurethanes offer substantial advantages over conventional technologies in terms of applicator safety. The hazards of polyurethane application are easily recognized and controlled. The isocyanates can be formulated and blended to reduce the handling risks to almost nil. The systems are solvent free and, as discussed above, have lower toxicity levels than the epoxies.

The main hazardous pre-cursor of polyurethane is isocyanate. Contrary to popular beliefs isocyanate is <u>not</u> carcinogenic[5]. The only problems are allergic reactions.

A very small percentage (<1%) of the population will exhibit immediate allergic reactions when exposed to isocyanate and should avoid all contact with the material. The average individual however, will only exhibit temporary irritation of the respiratory systems, skin and eyes when over-exposed.

This isocyanate hazard is easily recognized. Most applicators will know they are being over exposed because they get runny noses, itchy eyes and irritated throats. Once the over-exposure is stopped, the allergic reactions end. These 'early-warning' discomforts are a very effective way to avoid over exposure. Only prolonged unprotected exposure to isocyanate can cause irreversible sensitization problems.

Further, most formulators of polyurethane do not use monomeric isocyanate in their formulations. A much safer (more economical and better performing as well) polymeric type of isocyanate is used.

Applicators using polyurethane coatings can be assured that these systems are safer to handle than most epoxies or conventional painting systems.

APPLICATION

A number of unique advantages of fast setting polyurethanes over conventional tank lining technologies are found in the application of the systems. Fast cures and high application rates give the polyurethanes a much faster turn around time over epoxy systems.

Typically, the setting (or cured-to-touch) time for these polyurethane systems ranges from one minute to on hour. The pot life is effectively zero to 5 minutes. Two component airless spray pumps meter the components (iso and polyol) in the correct ratio[6]. The components are kept apart until they meet at (or just before) the spray gun. This type of equipment, although slightly specialized, is readily available from the main airless painting equipment manufacturers.

It is this fast setting time that provides many of the advantages. First, the coating can be applied to virtually any required thickness in one coat. Depending on the thixotropic properties, the applicator can usually put on 200 to 400 microns (8-16 mils) in one pass. Within 5-10 minutes (for some systems, seconds for others), this first pass has cured enough so that a second pass can be applied. This multi-pass process is continued until the required thickness is achieved.

Most coating formulators also make compatible slower setting systems that be brushed or rolled for doing 'hard-to-spray' roof areas or small repairs.

The large capacity of standard spray machines combined with the fast curing characteristics for these polyurethane systems generally result in very high application rates. A contractor doing large water storage tanks in the Abu Dhabi reported spraying over 150 m^2 per hour (1500 ft^2/hr) on complicated roof areas and up to 400 m^2 per hour (4000 ft^2/hr) on the clear floor area[7]. Those figures involved the application of a total film thickness of approximately 500 microns (20 mils). Conventional epoxy application equipment rarely achieve rates above 125 m^2 per hour (1250 ft^2/hr) for only one (of several required) coat (5 mils or 125 microns dft)[8].

In addition to curing quickly, the 100% solids polyurethanes will fail by blistering or not curing almost instantly if there is a problem with the surface preparation or the mixing ratio. If the coating is not dry-to-the-touch in the standard 1 to 15 minute time frame, the applicator and/or inspector knows that something is wrong. If there is any water on the substrate surface, the polyurethane will foam immediately with the water to create an easily identified problem area. Within minutes the film thickness can be checked with a magnetic gauge and the adhesion to the substrate can be tested with a knife. Typical slow curing epoxies may take several days or weeks to develop any recognizable problems. This 'instant-failure/idiot-proof' characteristic helps the applicator, inspector and owner easily recognize and correct application problems *before* the tanks or pipes are put into service.

Again, due to the rapid curing properties, the polyurethane systems can be put into immersion service usually within 24 - 48 hours. Epoxies need seven to ten days to fully cure and to allow the solvents to evaporate.

Finally, the nature of the iso/polyol reaction is exothermic. Because it generates its own heat, it can be applied at almost any ambient temperature. In the underground steel fuel storage tank and pipeline industries, where 100% solids polyurethanes are used extensively, it is not uncommon for applicators to apply the materials at temperatures down to -20°F (-29°C). Epoxy systems normally require temperatures above 50°F (10°C). Using the polyurethane technology allows tanks to be lined (or coated) during the cold months of the year and allows for easier maintenance scheduling.

All of these factors add up to a coating application that is very fast and very efficient. Not only can the coating be applied at any time of the year, but it also can be put into service two to three times faster than conventional epoxy.

PERFORMANCE, HOW LONG WILL IT LAST?

The question (how long will it last?) is a very difficult one to answer because it depends on so many variables. Factors such as surface preparation, film thickness, temperature, damage in handling, cathodic protection and so on, all effect the longevity of a coating.

Based on experiences within other industries and performance test results, it is estimated that ***100% solids polyurethane systems will outlast epoxies by 20 to 30 percent.*** A survey of the technical data sheets for several polyurethane and epoxy systems indicate this fact[9]. The results are shown in Figure 3.

The first tests are those measuring toughness. Quite often the hardest job for a coating is to resist the wear and tear of the handling between the application and the installation. It also must be able to resist expansion and contraction (due to temperature changes). Generally speaking, the tougher a coating is, the longer it will last[10]. Good tests are those measuring flexibility, abrasion resistance and impact resistance. Polyurethanes, in general, are much tougher than epoxies. Specifically, in the categories of impact, flexibility and abrasion, the 100% solids polyurethanes have numbers that are almost twice as good as the epoxies.

The next job of the coating is to prevent corrosion. Technically, the coating must provide a dielectric barrier between the metal and the soil (electrolyte). Widely considered to be one of the best measures of coating longevity is the ASTM G8 cathodic disbondment resistance test[11]. A holiday (small hole in the coating) is created on a coated panel. The section with the holiday is then immersed into a conductive solution (saltwater) and a small current is passed through the holiday to simulate a corrosion cell. The test is usually performed for 30 days at ambient temperature. After the 30 days, the coating that can be disbonded (scraped or lifted away) from the holiday, is measured. The smaller the area, the better results. The beauty of the CD test is that it approximates what will happen to a coating over a lifetime in 30 days.

The validity of this test has been shown over and over again in various industries, such as the pipeline industry[12]. The typical 100% solids polyurethanes achieve results that are 3 times better than the epoxies. *The better the CD test, the greater the longevity of the coating.*

Another measure that is a good indication of longevity is the adhesion of the coating to steel. The stronger the bond between the coating and the steel substrate, the harder it is to come off and the longer it will last[13]. The polyurethanes routinely achieve numbers in the 2000 psi (1400 N/cm^2) range for their adhesion to steel. Epoxy manufacturers rarely report figures over 1000 psi (700 N/cm^2).A final test is water vapour permeability. This test measures how much water vapour can be passed through a film of coating at a given thickness and pressure. If water can pass through a coating, it will cause corrosion underneath and almost certain failure. The lower the permeability, the longer the coating will last. In this categories both the polyurethane and epoxy manufacturers report similar test results.

When selecting a coating system, a specifier should consider these performance measures to make the best choice. Based on these standard tests, the polyurethane systems can be considered to outperform the epoxies in most cases. This higher level of performance means that the 100% solids polyurethane will last longer.

PERFORMANCE TEST RESULTS

Figure 3

Test	Polyurethane	Epoxy	Rating
Impact Resistance ASTM D2749	2.3 joules (20 in.lbs)	1.8 joules (16 in.lbs)	higher is better
Flexibility ASTM D4145	110°	90°	higher is better
Abrasion Resistance ASTM D4060	52 mg loss	120 mg loss	lower is better
Cathodic Disbondment Resistance ASTM G8	10 mm radius	18 mm radius	lower is better
Adhesion ASTM D4541	1410 N/cm^2 (2000 psi)	705 N/cm^2 (1000 psi)	higher is better
Permeability ASTM E96B	0.0041 perm.cm	0.0041 perm.cm	higher is better
test results have been gathered from various PUR and epoxy manufacturers technical data sheets [9]			

PER M^2 APPLICATION COSTS

Figure 4

	Polyurethane	Epoxy
Material (375 microns)	$ 6.00	$ 5.00
Coat 1	$ 6.00	$ 5.00
Coat 2	$ 0.00	$ 5.00
Inspection	$ 0.20	$ 0.20
Equipment	$ 0.40	$ 0.20
TOTAL	$12.80	$15.30
CONCLUSION	***Polyurethane is 16% less expensive.***	

Costs were estimated based on responses to a survey of American applicators.

COSTS

The true cost of any coating system is not the 'cost per bucket' or the even the applied cost per square meter. The true coating cost is:

Coating Cost = Cost/M^2/Year of Life

The true cost of using the polyurethane technology will be significantly lower than that of conventional coating systems because the technology is less expensive to apply (1 coat vs. several coats) and because the polyurethane coating will last 20% to 30% longer.

The one coat system substantially reduces application costs. In early 1990 three contractors were asked to supply costing for the material and application of a polyurethane and epoxy system for a typical one million gallon (four million liter) elevated water tank. Sandblasting costs were not included because they were the same for both technologies. Both coatings were to be applied to a dry film thickness of 375 microns. The polyurethane in one coat, the epoxy in two coats. The numbers are shown in figure 5. The costs were based on a price of $12.00/liter for the 100% solids polyurethane and $6.50/liter for 60% solids epoxy. A 30% waste factor was used for both systems.

The polyurethane is actually applied in three passes, all at one time (as one coat). It takes an applicator about 20% more time to apply the complete coat of polyurethane than it does to spray on the first coat of epoxy. The second coat of epoxy takes the same amount of time as the first coat. The inspection costs include doing minor repairs and touch-ups and were estimated to be the same in both cases. Because a two-component heated spray unit is usually required to apply the polyurethanes, there is a higher equipment cost. The figures in the chart are based on a total equipment cost depreciated and amortized over five years.

In conclusion, the total applied cost of the polyurethane system is 16% less than the typical epoxy. If the epoxy must be applied in a three coat system, the polyurethane is almost 40% less expensive. Not only is the polyurethane technology significantly less expensive, it's application would result in the tank being out of service for seven days versus sixteen days for the epoxy. Also, given the indication that the polyurethane system will last 20% to 30% longer than the epoxies, the true cost/m^2/yr is dramatically in favour of the polyurethane.

CASE HISTORIES

Large Water Tanks in Abu Dhabi[14]

Abu Dhabi, due to rapid development of oil and gas reserves, has grown from a Bedouin desert camp twenty years ago to a major metropolis. This tremendous growth has increased Abu Dhabi's need for potable water.

The Abu Dhabi Water and Electricity Department (WED) required a high performance lining system that would not only protect their tanks from corrosion but meet their safety requirements for potable water contact. The corrosive nature of their desalinated water had lead to the failure of several epoxy systems.

The pipe was spiral welded steel manufactured by Ameron Corp. of Tracey, California. The pipes were approximately 1.8 meters in diameter by 12 meters long. The total length of the line was almost 5 kilometers.

Of particular note in this application was that all of the interior coating was applied automatically. An electrically powered cart with a rotating spray nozzle applied the entire thickness of 500 microns in one pass down the length of the pipe. The blasting was also done in the same manner.

This process combined with the very fast cure rate of the coating allowed the applicator to complete over 20 pipes (blasting and coating) or almost 1400 square meters each day.

The project was completed in less than 1/3 of the time that it could have been completed with any comparable epoxy system at only 2/3rds of the cost.

Park Water Company - Santa Paula History[16]

The Park Water Company is a private utility based in Downey, California. Park Water has a customer base of approximately 60,000 people and serves various communities in the Los Angeles area as well as one in Montana. One of theses areas, the town of Santa Paula, has used various 100% solids polyurethane materials as interior tank linings.

Three 500,000 gallon (2,000,000 liter) ground storage reservoirs 80 feet (24 meters) in diameter by 24 feet (7 meters) in height have been lined to date. The oldest one was lined over five years ago in 1986. An inspection in November, 1991, reported that the tank was in excellent condition with no areas of rust or coating failure whatsoever. Both of the remaining two tanks were used tanks that were re-erected from another site. One was completed (re-erected and lined) in 1988 and the other in 1989. At the 'one-year' inspection, the three year old tank was found to have some minor holidays. These were easily repaired.

Not only does Park Water does not anticipate any further problems with the 100% solids polyurethane coatings but they expect to be able to enjoy excellent tank longevity as a result of their coating selection.

Vienna Duct Tape Tanks[17,18]

Vienna is a small city of 10,000 people sitting on the West Virginia side of the Ohio river. In the winter of 1990, by using regular 'hardware-store' duct tape and a 100% solids polyurethane coating, the Vienna Public Works department saved almost $60,000 when they repaired their water treatment tank.

The tank in question is a ground storage reservoir (6m in diameter by 5m high) that is used for water treatment. Inside the tank are several 'full-height' baffles designed to provide for good mixing of the water treatment chemicals into the incoming water.

The Public Works officials knew the tank was in extremely poor condition with extensive corrosion. The inner baffles had holes of approximately 2 inches in diameter and the some of the welds on the outer shell were leaking. The only option they thought they had was to replace the tank with a new one at a cost of between $80,000 and $90,000 (interior and exterior painting included).

Lewis Estep of Estep Painting out of Charleston, WV provided the city with a very cost effective and high performance alternative. The worst of the holes were patched with steel plate and welded. The so-called 'duct-tape' method was employed on the remaining 30 to 40 holes.

After the steel was blasted, duct tape was patched over the back side of the holes. The 100% solids, fast setting polyurethane was then applied to a thickness of 6 mm in about 20 passes with 30 seconds to 1 minute of waiting between passes. The balance of the pitted steel surface received 1.5 mm to 2 mm in order to fill the pits and completely seal any leaks.

One year later (March 1991) the tank was emptied for inspection. Bob Chauncey, the Director of Public Works for the City of Vienna, reported that "the tank and coating were in perfect shape". Mr. Chauncey expects the tank to last at least 20 years before any repair work is necessary.

Lewis Estep was quoted as saying "in my 30 years of painting water tanks I have tried every new think that has ever come onto the market and this polyurethane is the best stuff that I have ever seen".

The total cost of the rehabilitation (inside and out) was approximately $20,000. Not only did the City of Vienna realized a saving of $60,000 but it only took Estep Painting 1 week (and 1 roll of duct tape) to complete the interior coating works and to get the tank back into service.

CONCLUSION

The solvent-free polyurethane technologies will eventually replace the conventional epoxies that dominate the industry today because they are safer, faster to apply, more economical and offer greater longevity.

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Proposed On-Load Tube Cleaning for MSF Plants Without Circulating Pumps And Without Extra Energy Cost

Abunayib S.A.

PROPOSED ON - LOAD TUBE CLEANING FOR MSF PLANTS WITHOUT BALL CIRCULATING PUMPS AND WITHOUT EXTRA ENERGY COST.

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ABSTRACT

This study presents a theoretical alternative to an existing on-load tube cleaning system used with a 1140 m^3/h MSF desalination plant. It mainly consists of two interchangeable ball collectors plus the existing ball strainer and spares all the existing ball circulating pumps. Cold brine from discharge of the brine recirculation pump transports the balls from the selected collector to the injection point at around 0.25 bar pressure difference. The hot brine separated from the balls caught in the other collector is connected to the first flash chamber. Hence, by avoiding mixing of hot and cold brine, the extra steam consumption associated with operation of the existing system is theoretically eliminated. Also sparing the ball circulating pumps saves the major service and spare parts cost.

INTRODUCTION

This study is based on practical experience with the existing on - load tube cleaing (OLTC) system used for tube cleaning of the heat gain section (HGS) and the brine heater (BH) of each of the four 1140 m^3/h cross - tube type MSF desalination plants at Ghubrah Station, Sultanate of Oman.

An inherent problem with the existing systsem is that during its operation the MSF plant production drops by little while the steam consumption increases. This phenomenon tributes mainly to the fact that the existing system injects the cleaning balls with a little portion of brine of the highest temperature into brine of the lowest temperature.The proposed system presented in this study overcomes the above problem by avoiding mixing of hot and cold brine. It mainly consists of two ball collectors plus the existing ball strainer.

Another advantage of the proposed system is that it works without ball circulating pumps. The five ball circulating pumps of the existing system are its major serviceable and spare parts consuming item.

EXISTING OLTC SYSTEM

The OLTC system is provided to remove the deposition of soft scale and sludge from the inner surface of the HGS and the BH tubes while the MSF plant is in operation. This is done by circulating sponge rubber balls which are little larger than the inner diameter of the tubes. Main components of the system are the ball strainer (SI), No.1 ball circulating pumps (P1 A/B), the ball collector (S2) and N0.2 ball circulating pumps (P2 A/B/C) as shown in Fig 1 . There is provision for selecting one of two cleaning cycles. One is the standard cycle for cleaning both the HGS and the BH tubes and the other is the by - pass cycle for cleaning the BH tubes only. In the standard cycle, which is normally used at Ghubrah, the balls are transported from the collector by the ball circulating pumps (One of No.1 and two of No.2 pumps) and get injected into the recirculated brine at inlet to HGS. The balls then travel with the main stream of brine and are randomly forced by the brine hydraulic pressure through the tubes of the HGS and the BH. At the outlet from the BH, the balls are separated by the strainer with a definite volume of hot brine and again recycled. In the by - pass cycle only one No.1 pump is used to transport the balls from the collector to the injectin point at the inlet to the BH. The ball collector encloses a basket which is fitted with an electrically actuated valve. In the operation mode this valve is open allowing the balls to pass through. In the ball catch mode the valve is closed, retaining the balls and allowing the hot brine to pass through the perforations of the basket and hence only hot brine is pumped to the injection point during the catch mode. The OLTC system at Ghubrah is operated twice a day for one hour operation and half an hour ball catch.

PROBLEMS WITH THE EXISTING SYSTEM

A problem associated with the existing system is that during its operation the production of the MSF plant drops by around 0.5 % while the steam consumption increases by around 2.5 %. This estimation is based on the data collected at Ghubrah phase III plant (Fig 2 a,b and Table 1a,b,c). This adverse effect is attributed mainly to the fact that during operation of the OLTC system, including ball catch mode, the little portion of brine the system circulates is taken from the BH outlet, where the temperature is the highest, and injected into the inlet to the HGS where the temperature is the lowest (Last stage brine temperature). This results in a slight temperature increase of the brine entering the tubes of the HGS and hence lower cooling effect which causes the drop in distillate production. On the other hand the lower cooling effect results in a drop of the quantity of heat gained in the HGS which is compensated by an extra amount of steam in the BH. From observation, the symptoms associated with OLTC are that the brine inlet temperature to the BH drops by around 0.5°C and the BH brine outlet temperaturre momentarily drops, causing the temperature control valve (TCV) to open more to maintain the same preset temperature, There is no provision for measuring the increase in temperature of the brine entering the HGS, but still that can be worked out as follows:

Consulting the main design data, Ref [1] ,

let	m_1	= Brine recirc. pump rated flow	=3194 kg / sec.
	m_2	= Ball circulating pumps rated flow	= 11.9 kg/sec.
	T_1	= Inlet temperature to HGS tubes	
	T_2	= BH brine outlet temperature	= 105°C
	T_3	= Inlet temperature to HGS during OLTC.	
	C_p	= Brine heat capacity	
	T_o	= A reference temperature	

Now making a simple heat balance equation,

$$C_p m_1 (T_1 - T_o) + C_p m_2 (T_2 - T_o) = C_p (m_1 + m_2) (T_3 - T_o) \qquad (1)$$

Assuming C_p Constant. $T_o = 0$ and $m_1 + m_2 = m_1$ as m_2 is too small campared to m_1 , then (1) becomes.

$$m_1 T_1 + m_2 T_2 = m_1 T_3 \qquad (2)$$

Substituting for m_1,m_2 and T_2 from above data,

$$\text{Then} \quad T_3 = T_1 + 0.4 \qquad (3)$$

Hence the inlet temperature to HGS increases by around 0.4°C during the operation of OLTC system. The fact that this phenomenon is mainly attributed to mixing of hot and cold brine is supported by the observation that even during ball catch mode the adverse effect is persisting. The estimated time for ball catch is half an hour. The balls were observed to collect completely in less than half the estimad period and still the plant performance normalizes only after complete stoppage of the OLTC system. Also during incidents when the system failed to stop automatically at the preset time, i.e.longer ball catch period than normal. the same phenomenon was observed to be persisting. This phenomenon is also discussed by Eimer, Ref [2], who estimated the increase in energy consumption of a 22000 m^3 /d MSF plant by as much as 5% during OLTC and attributed that mainly to mixing of hot and cold brine.

On the other hand routine and breakdown maintenance of the system is mainly servicing and repairing the ball circulating pumps. Involving five pumps, the existing system therefore, has a relatively high maintenance cost.

PROPOSED OLTC SYSTEM

The proposed OLTC system does not use ball circulating pumps and does not involve mixing of hot and cold brine. It mainly consists of two interchangeable ball collectors(A/B), the existing ball strainer , valves and piping as shown in Fig 3. One collector is used for ball injection and the other for ball catch and then change over after a preset period of time.

The ball collector selected for injection works under the pressure difference btween the Brine Recirculation Pump (BRP) discharge, where the proposed upstream connection is to be fitted. and the existing injection point further downstream. From observation,the pressure at the discharge of the BRP as measured by the pressure gauge fitted just upstream to the discharge valve is 4.50 to 4.60 bar. This, while the pressue at the injection point is around 4.3 bar as measured by the pressure gauge at the discharge of the selected last ball circulating pump when the system is out of service. This pressure is not the boundary pressure as the injection point (Fig,4a) is fitted close to the axis of the BRP discharge line where the kinentic head is little higher than the average due to accelerated streamlines by curvature around the openings of the injection pipes as they are slant cut facing the downstream (Fig 4b) . This pressure difference can still be increased by fitting the proposed upstream connection close to the axis and making it slant cut facing the upstream.(Fig 5).

On the other hand the ball collector selected for ball catch works under the pressure difference between the BH outlet, where the pressure is 1.8 to 2.0 barA, and the first flash chamber where the pressure is slightly less than atmospheric (around 0.95 barA)

Philosophy of operation

The two ball collectors are fitted with valves which enable selecting one collector for ball injection and the other for ball catch and then changed over after a certain period of time. For convenience refer to Fig.3 and let collector "A" be in the injection mode and collector "B" in the catch mode. Cold brine from discharge of the BRP enters collector "A" via the bottom inlet valve "VA1" and through the perforations of the enclosed basket it passes to the upper half of the collector where it takes the balls though the upper outlet valve "VA2" to the existing injection point. All other valves of collector "A" are in closed position. From the injection point, same as in the existing system, the balls travel with the main stream of brine through the tubes of the HGS and the BH. At the outlet from the BH the balls are seperated by the existing strainer and piped with a definite volume of hot brine to collector "B" entring via the upper inlet valve "VB3". The balls are hence retained in the upper half while the hot brine passes through the basket leaving the collector via the bottom outlet valve "VB4" and through the downstream piping to the first stage flash chamber. All other valves of collector "B" are in closed position. Now after all the balls from collector "A" are collected in collector "B" the two collectors are charged over. To ensure continuous ball catch, collector "A" is first changed over from injection to catch mode. Hence, inlet valve "VA1" and outlet valve "VA2" are closed . When proved closed, drain valve "VA 5" is open to depressurize the collector. With drain valve still open, the top inlet valve " VA3" is open and the collector is warmed up for a certain period of time and then the drain valve is closed. When proved closed, the bottom outlet valve "VA4" is open and hence collector "A" is in catch mode in paralled with collector "B". Now collector "B"is changed over by first closing inlet valve "VB3" and outlet valve "VB4". When proved closed the top outlet valve "VB2" is open. When proved open, the drain valve, "VB5" is open to flush and cool down the collector and then the drain valve is closed. When proved closed, the bottom inlet valve "VB1" is open and hence collector "B" is in the injection mode.

The differential pressure (dp) existing measuring and monitoring system is used to backwash and protect the ball strainer. When (dp) goes high the ball injection will be stopped and the concerned collector will change over to catch mode. The strainer will go to backwash positon after a preset period of time if (dp) is still high. When (dp) goes extra high then the strainer will immediately go to back wash position and at the same time ball injection collector will change over to catch mode. The injection mode is interlocked with the strainer position. To put any of the two collectors in the injection mode is not possible without first putting the strainer in the operation position.

Connection to the MSF plant

Only two new connections are required to tie the proposed OLTC system to the MSF plant. One is the cold brine upstream connection at the BRP discharge and the other is the hot brine downstream connection at the first stage flash chamber. The cold brine connection is proposed to fit through the cover of the 600 mm diameter manhole that exists just upstream the discharge valve of the BRP. Thus only the manhole cover can be modified as shown in Fig.5. The connection is so arranged to allow for opening the manhole when required. The hot brine down stream connection is even easier as it connects to the drain of the first flash chamber. The arrangement, as shown in Fig.6 allows for flash chamber drainage when required.

Features and Conclusion

Beside the two main advantages of saving the extra energy influenced by operation of the existing system and sparing all the ball circulating pumps, the proposed system is still featured by the following:-

i- In the absence of the extra energy , the system can be operated for longer time and hence better cleaning is possible.

ii- As pumps are not used the system can be operated manually.

iii- When power actuators are used for the strainer and the valves then, unlike the existing system, power failure will not increase the differential pressure across the strainer as ball catch and ball injection will still work.

iv- Though abrasion free, the pumps will however squeeze the balls by the centrifugal force it exerts on them. Hence, by eliminating the pumps, one ball damaging factor is eliminated.

v- The total number of valves used is less than in the existing system.

However, the proposed system is still theoretical and requires experimentation before it can be used as an alternative to the existing system. The size of the collectors, the rate of ball injection and the duration of operation and ball catch are main points to be considered in the experimental work.

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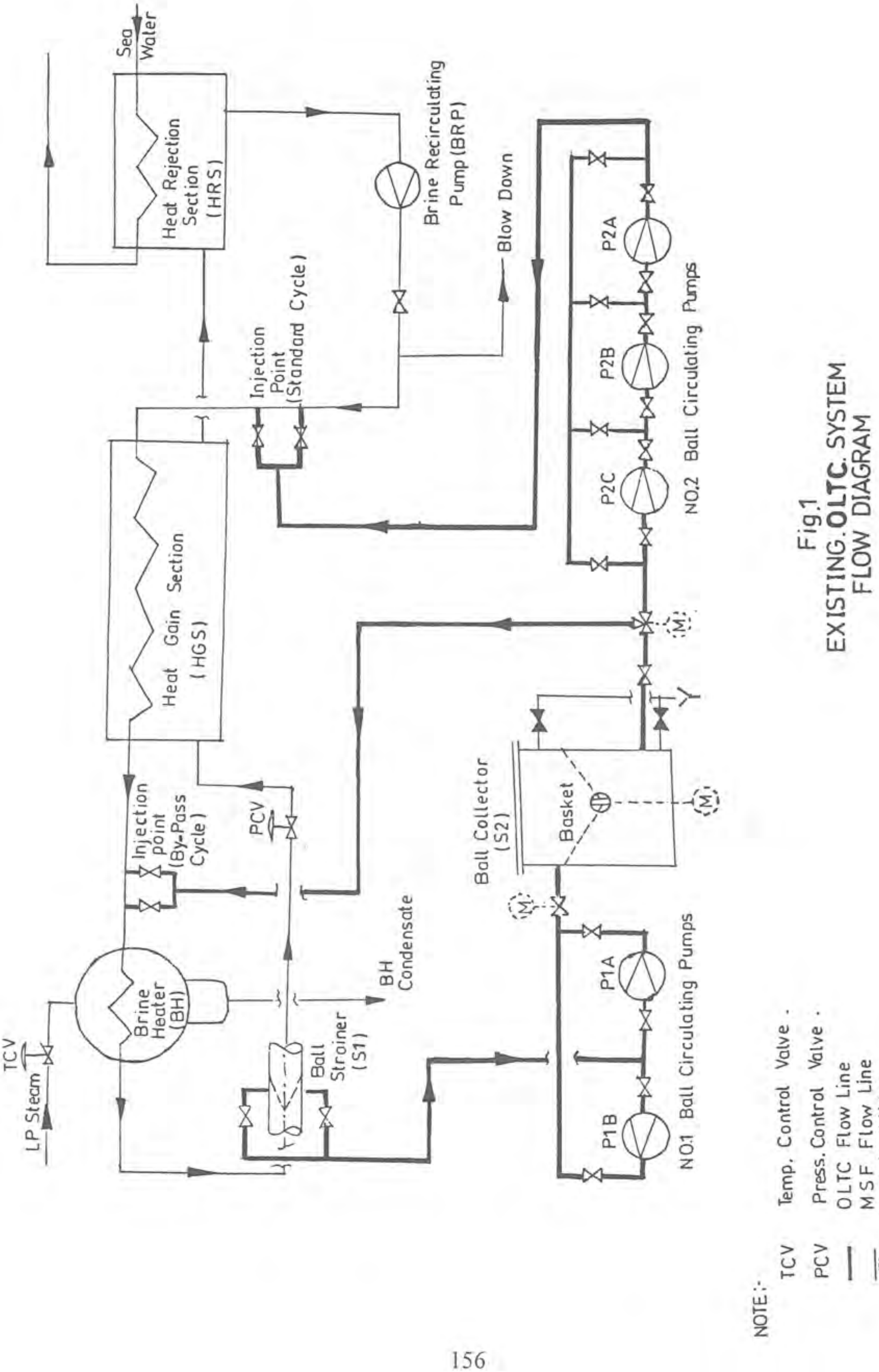


Fig.1
EXISTING **OLTC** SYSTEM
FLOW DIAGRAM

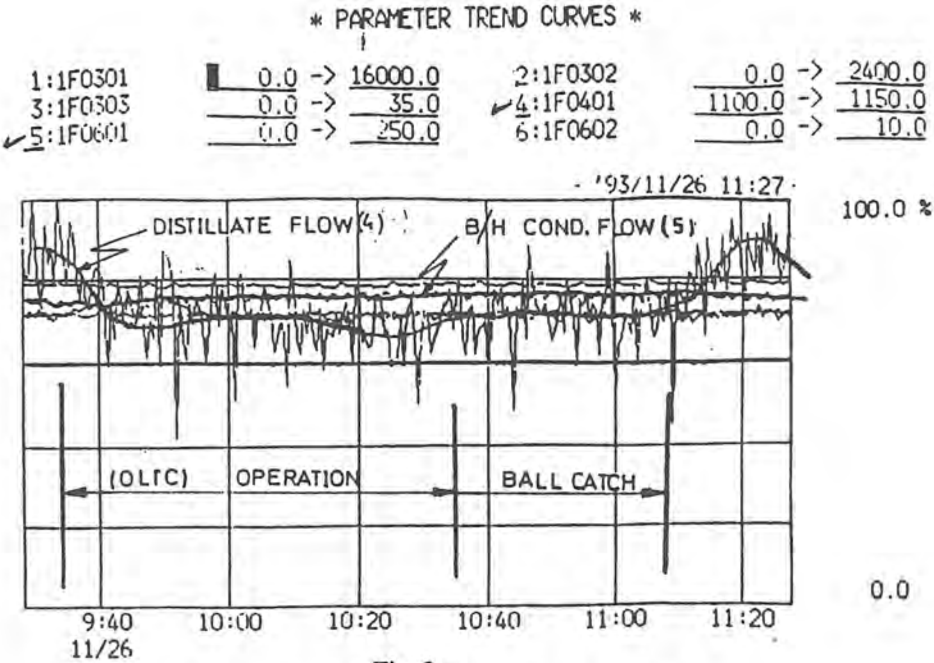


Fig.2a
* DISTILLATE & BH CONDENSATE FLOW DURING OLTC

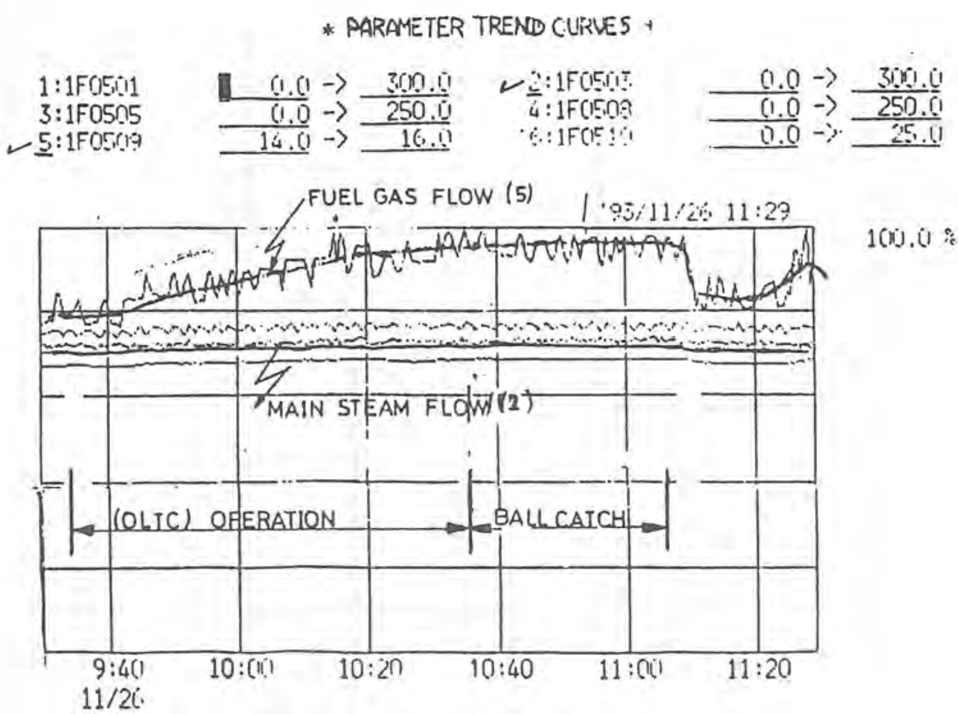


Fig.2b
* FUEL GAS & MAIN STEAM FLOW DURING OLTC

* Computer printouts photocopy from GHUBRAH Phase III plant.

* Table.1a : DISTILLATE FLOW HOURLY INTEGRATED VALUES.

* TAG DATA MAINTENANCE (ANALOG) *

TAG No. [1F0401] SERVICE NAME 1 DISTILLATE FLOW
LOOP No. 64 SERVICE NAME 2 DISTILLATE FLOW
SCAN(SEC) 3 OPE. GROUP DESAL 1 TYPE PV

DATE [1993.10.21]

TIME	INT	AVE	TIME	INT	AVE	TIME	INT	AVE
1:00	1145.1	1145.1	9:00	1141.6	1141.6	17:00	1140.0	1140.0
2:00	1143.7	1143.7	10:00	1140.1	1140.1	18:00	1134.9	1134.9
3:00	1142.7	1142.7	11:00	1141.7	1141.7	19:00	1138.8	1138.8
4:00	1143.4	1143.4	12:00	1140.3	1140.3	20:00	1143.1	1143.1
5:00	1142.3	1142.3	13:00	1137.3	1137.3	21:00	1142.2	1142.2
6:00	1142.3	1142.3	14:00	1141.6	1141.6	22:00	1142.2	1142.2
7:00	1143.1	1143.1	15:00	1143.3	1143.3	23:00	1143.8	1143.8
8:00	1143.2	1143.2	16:00	1142.0	1142.0	24:00	1143.6	1143.6

* Table.1b : LP.STEAM FLOW HOURLY INTEGRATED VALUES.

TAG No. [1F0106 SERVICE NAME 1 LP STEAM FLOW.
LOOP No. 54 SERVICE NAME 2 LP STEAM FLOW
SCAN(SEC) 3 OPE. GROUP DESAL 1 TYPE PV

DATE [1993.10.21]

TIME	INT	AVE	TIME	INT	AVE	TIME	INT	AVE
1:00	188.0	188.0	9:00	187.7	187.7	17:00	187.5	187.5
2:00	188.0	189.0	10:00	187.3	187.3	18:00	189.7	[illegible]
3:00	187.8	187.8	11:00	187.0	187.0	19:00	187.1	187.1
4:00	187.9	187.9	12:00	186.3	186.3	20:00	187.4	187.4
5:00	187.8	187.8	13:00	187.2	187.2	21:00	186.9	186.9
6:00	187.9	187.9	14:00	192.5	192.5	22:00	186.7	186.7
7:00	187.8	187.8	15:00	188.2	188.2	23:00	187.2	187.2
8:00	187.8	187.8	16:00	186.9	186.9	24:00	187.2	187.2

* Table.1c : FUEL GAS FLOW HOURLY INTEGRATED VALUES.

TAG No. [1F0509] SERVICE NAME 1 FUEL GAS FLOW
LOOP No. 9 SERVICE NAME 2 FUEL GAS FLOW
SCAN(SEC) 3 OPE. GROUP BOILER 1 TYPE PV

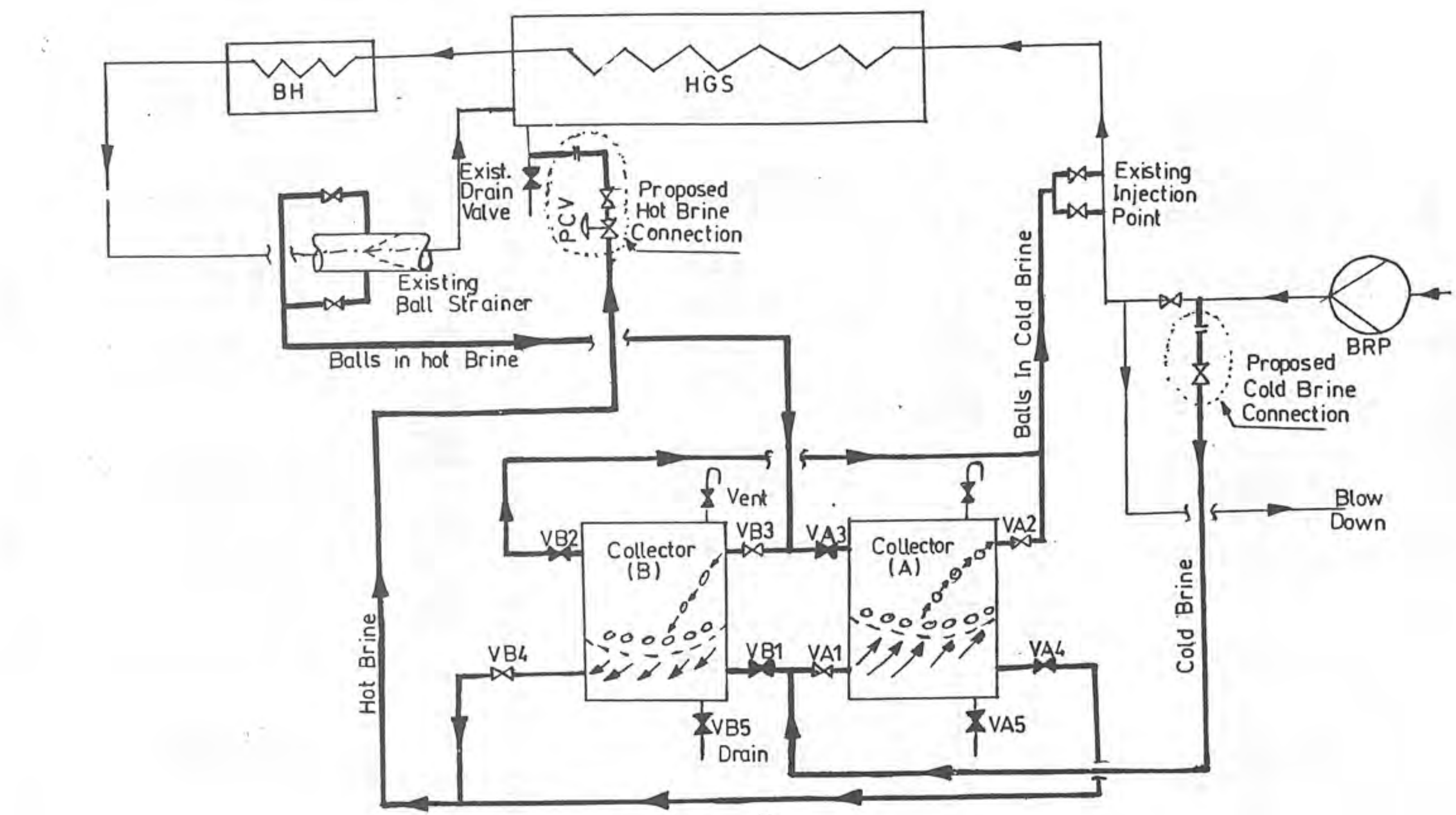
DATE [1993.10.21]

TIME	INT	AVE	TIME	INT	AVE	TIME	INT	AVE
1:00	15.5	15.5	9:00	15.5	15.5	17:00	15.4	15.4
2:00	15.5	15.5	10:00	15.4	15.4	18:00	15.7	15.7
3:00	15.5	15.5	11:00	15.4	15.4	19:00	15.5	15.5
4:00	15.5	15.5	12:00	15.4	15.4	20:00	15.5	15.5
5:00	15.4	15.4	13:00	15.4	15.4	21:00	15.4	15.4
6:00	15.4	15.4	14:00	15.7	15.7	22:00	15.5	15.5
7:00	15.4	15.4	15:00	15.4	15.4	23:00	15.5	15.5
8:00	15.5	15.5	16:00	15.4	15.4	24:00	15.5	15.5

NOTE :-

OLTC :	STARTED	BALL CATCH	STOPPED	
TIME :	1250	1350	1420	HRS
:	1645	1745	1815	HRS

* Computer printouts photocopy from GHUBRAH phase III plant.



NOTE :-
- Open Valve.
- Closed Valve.
- OLTC Flow Line
- MSF Flow Line

Fig. 3
PROPOSED OLTC SYSTEM
FLOW DIAGRAM.

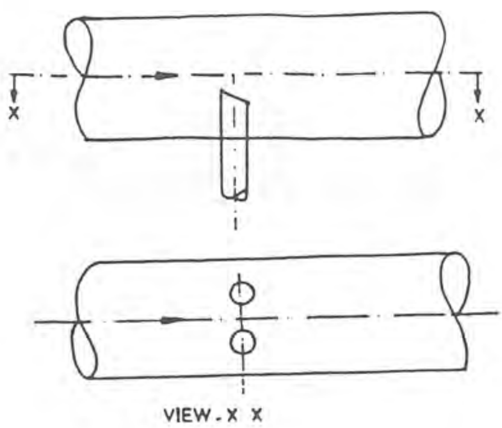


Fig.4a
EXISTING BALL INJECTION
POINT

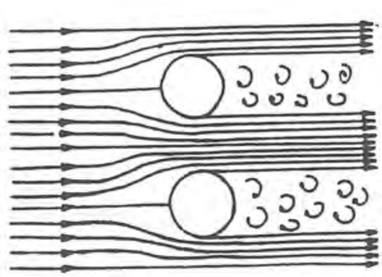


Fig.4b
ILLUSTRATION OF FLOW STREAMLINES
AROUND THE OPENINGS

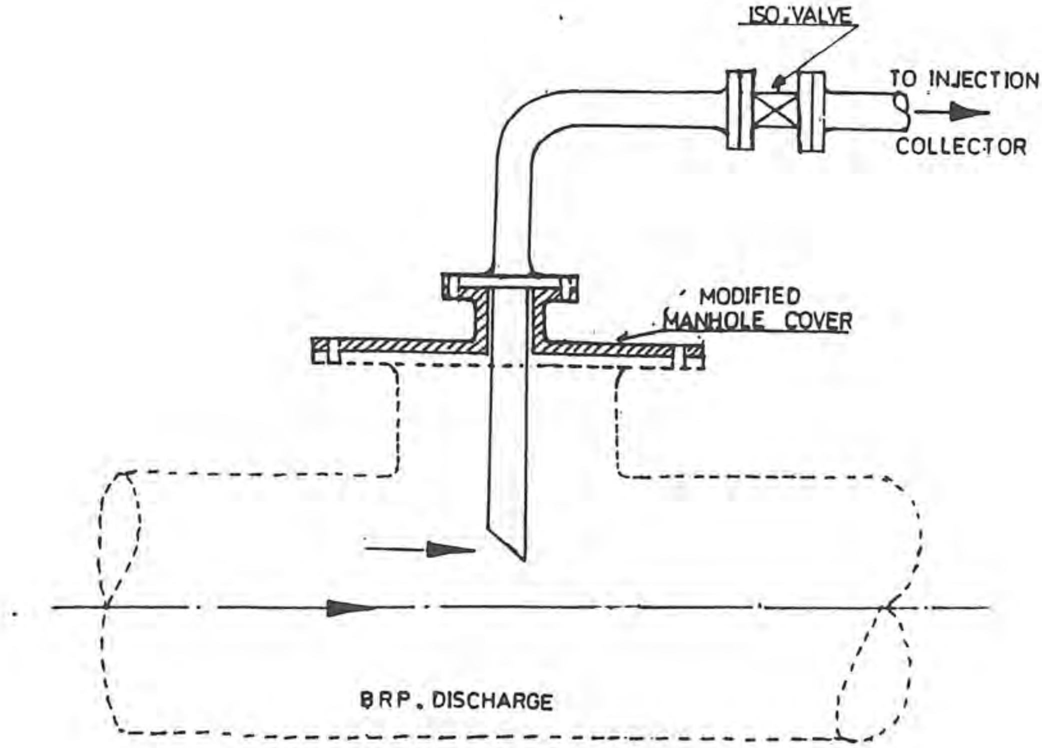


Fig.5
PROPOSED COLD BRINE
UPSTREAM CONNECTION

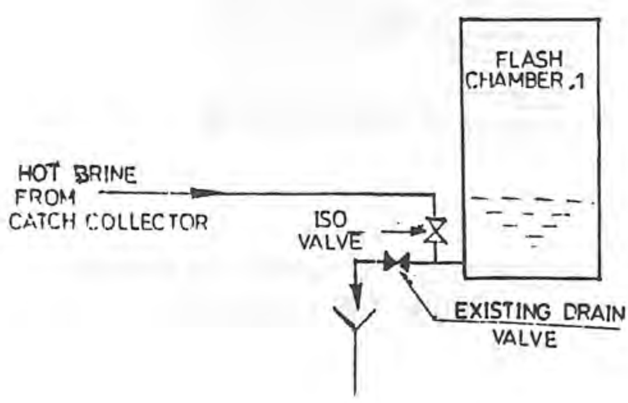


Fig.6
PROPOSED HOT BRINE
DOWNSTREAM CONNECTION

Session - 5
Natural Water

Hydrogeochemistry of Khobar Aquifer In Easter Saudi Arabia

Hassan M. Hassan

Hydrogeochemistry of Khobar Aquifer in Eastern Saudi Arabia

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ABSTRACT

Major ion distribution and computerized equilibrium studies were conducted on the Khobar aquifer 'of the Eocene age' in Eastern Saudi Arabia. The aquifer is oversaturated with respect to calcite and dolomite, and undersaturated with gypsum. The Piper's trilinear plots show the presence of four facies in the Khobar aquifer. The groundwater evolutionary pattern in the aquifer generally follows Chebotarev's sequence. However, the presence of probable seawater intrusion near Jubayl and water mixing on structural highs are the main superimposed processes. The total dissolved solids (TDS) in the aquifer exceeds the recommended levels for most uses.

INTRODUCTION

Groundwater hydrochemical data give important information on basic geochemical processes in an aquifer system, such as mineral dissolution/precipitation, ion exchange, and seawater intrusion.

Few hydrogeochemical studies had been previously carried out to investigate the controls on the composition and distribution of hydrochemical parameters in the aquifers of the Eastern Saudi Arabia. One such study by Sen and Al Dahkheel (1986) has shown the dominance of NaCl and Ca SO_4 type waters in the Umm Er Radhuma aquifer.

In the present study, the hydrogeochemical charactersitics of the overlying Khobar aquifer, in the eastern province of Saudi Arabia, have been investigated (Fig. 1). The main objectives of this study are to investigate the distribution of different types of water in the province and to investigate the possible geochemical processes which are affecting the ground water composition.

GEOLOGY

The Eocene Dammam Formation in Eastern Saudi Arabia is divided into five members : Alat, Khobar, Alveolina Limestone, Midra and Saila Shale members (Fig. 2). Alat and Khobar which are mainly carbonate members constitute aquifers of regional importance (Powers et al., 1966).

Khobar Member is subdivided into two units, lower and upper units. The lower unit consists of light gray to tan dolomitic marl, whereas the upper unit consists predominantly of limestone with subordinate marly layers (Al Sayyari and Zottle., 1978). The Member conformably overlies the Alveolina Limestone Member and bounded at the top by the Alat Member (Fig. 2). It crops out at the south-southwest of Dhahran and on the Dammam Dome.

In the study area, the Khobar Member is karstfied and fissured, and provides water for domestic and agricultural purposes (Edgell, 1990). The aquifer is confined at the top by the lower marly unit of the Alat Member, and it is bounded at the bottom by a set of aquitards consisting of the lower three members of the Dammam Formation and the underlying Rus Formation.

HYDROGEOCHEMICAL DATA AND METHODS OF STUDY

The hydrogeochemical data used in this study were taken from the chemical analyses published by the Ministry of Agriculture and Water (MAW, 1984), and Groundwater Development Consultants (GDC, 1980). Chemical data on 234 water samples with less than 5 % charge balance error were used in the present study.

PCWATEQ (Rollins, 1990) program, which is a special modification of WATEQF (Plummer, 1984), is used for the calculations of the equilibrium conditions of the Khobar aquifer.

DISTRIBUTION OF MAJOR CONSTITUENTS

The distribution of major chemical constituents are shown in the form of isocons maps. The distribution of total dissolved solids (TDS) in Khobar aquifer varies from less than 1000 mg/l in the southern part of the study area to more than 7000 mg/l in the north (Fig. 4). The coastal area of the Arabian Gulf is characterized by an increase in TDS, however, this general trend is interrupted around Qatif and near Jubayl.

Calcium and magnesium have more or less the same pattern. The calcium concentration varies from about 200 to 1200 mg/l. Three distinctive areal distribution patterns of calcium are evident in the study area (Fig. 5). The southern part of the study area is characterized by low concentration of calcium. Near the coast, however, the calcium concentration increases gradually where it reaches 500 mg/l. In the central area, there are two highs of calcium concentration; one is near Hofuf and the other is around Dhahran-Abqaiq area, where it reaches 500 mg/l. Between these two highs, there is a low calcium concentration zone of less than 200 mg/l. Likewise, there is another area of low calcium concentration of 200-300 mg/l in the Qatif area. In contrast with the southern and central parts, the north is charactezied by high calcium concentration of more than 1100 mg/l.

Sodium and chloride distributions are identical in the Khobar aquifer. Sodium increases gradually seaward at the south of Dhahran (Fig. 6). A zone of low sodium extends from Qatif westward. This zone is bounded in the north and south by areas of high sodium concentration. The highest sodium concentration of more than 3000 mg/l occurs in the north of Jubayl, whereas around Jubayl the concentration drops to 300 mg/l.

sulfate in Khobar aquifer varies from 400-1600 mg/l (Fig. 7). Its concentration increases seaward, however, in Qatif and in the surrounding coastal land, sulfate assumes a low value of about 400 mg/l. Sulfate concentration increases near Hofuf and in Dhahran-Abqaiq area. In the area north of Jubayl, the rate of increase is similar to that of chloride.

Khobar aquifer has an average bicarbonate concentration of 180 mg/l (Fig. 8). The bicarbonate concentration is anomalously high near Hofuf, with a value of 480 mg/l. This high concentration zone extends north to Qatif and drops sharply seaward. In the north of Jubayl as well as in the southern part of the study area, a very low bicarbonate concentration occurs. This pattern contrasts with the distribution pattern of the other anions, chloride and sulfate, which increases from south to north and occurs in very high concentration to the north of Jubayl.

DISTRIBUTION OF SATURATION INDICES

The aquifer is oversaturated with respect to calcite (Fig. 9). Although the degree of saturation varies, there is no discernable generalized pattern of the variation. The calcite saturation index (S.I) values in the extreme north increases seaward. To the south, the S.I. values decreases from Jubayl all the way to Dhahran. South of Dhahran, at Hofuf, the calcite S.I. values drop rapidly eastward within a short distance. Here the high calcite S.I. values coincide with high bicarbonate (Fig. 8). The dolomite S.I. values are similar to those of calcite (Fig. 10; the aquifer being generally oversaturated with respect to dolomite.

Khobar aquifer is undersaturated with respect to gypsum (Fig. 11). The degree of undersaturation decreases from south towards north and east. Near Abqaiq and Hofuf, areas of relatively low undersaturation with gypsum are evident.

DISTRIBUTION OF HYDROGEOCHEMICAL FACIES

Four hydrogeochemical facies are identified in Khobar aquifer. Plots on the Piper's trilinear diagram (Fig. 12) show the presence of :-

1. Ca- SO_4 facies
2. Na-Cl-Ca- SO_4 facies
3. Na-Ca-Cl facies
4. Na-Cl facies

The Ca- SO_4 facies is present at the extreme south of the study area (Fig. 13).

The second (Na-Cl-Ca- SO_4) facies is as widely distributed as the first facies; the two water types cover adjacent areas in the south.

The third (Na-Ca-Cl) facies is abundant in the NE of the study area and it extends into the sea at the latitude of Jubayl. The facies divides two Na-Cl type waters which are located at the south and north of Jubayl.

The fourth (Na-Cl) facies represents the most abundant water type. It is present in two distinctive areas, at the north of Jubayl and at the central part of the study area. This water type is common in areas along the Arabian Gulf, except around Jubayl. Almost all the areas on which the big cities are situated in the Eastern Province are characterized by this water type.

The Na-Cl type water seems to be associated with the Na-Cl-Ca- SO_4 facies and Na-Ca-Cl facies. However, it seems that the latter two water types are related geochemically to the Ca- SO_4 in two different ways. The calcium in facies Na-Cl-Ca- SO_4 and Na-Ca-Cl facies could be generated by dissolution of aquifer minerals. The anomalous sulfate in the Na-Cl-Ca- SO_4 facies might have been introduced to the system by local gypsum dissolution or regional geochemical evolution.

HYDROGEOCHEMICAL PROCESSES

Chebotarev (1955), suggested that groundwater tends to evolve chemically towards the composition of seawater. He indicated that the evolution was normally accompanied by regional change in the proportion of major anions along the flow path, as shown below.

$$\xrightarrow{\text{Travel along flow path}}$$

$$HCO_3 \rightarrow HCO_3 + SO_4^2 \rightarrow SO_4^2 + HCO_3 \rightarrow SO_4^2 + Cl \rightarrow Cl + SO_4^2 \rightarrow Cl$$

$$\xrightarrow[\text{Increasing age}]{}$$

From a geochemical view point, the above sequence can be explained in terms of two main variables, mineral availability and mineral solubility (Freeze and Cherry, 1979).

The hydrogeochemical process which are taking place along three hydrogeochemical profiles are considered. Along profile AA' at well no. 203 there is a drop in TDS (Fig. 14). This well is located right over the Ghawar structure, whereby the confining units decrease in thickness allowing the interaction of different aquifers. Here the mixing of water from Khobar aquifer with the underlying better quality water aquifer, Umm Er Radhuma, is taking place. In well no. 222 the concentration of calcium, magnesium and sulfate increase while sodium, chloride and bicarbonate concentrations decrease. The SO_4/Cl ratio of this well is greater than 3 and it fits the gypsum dissolution trend (Fig. 15). This supports gypsum dissolution and concomitant dedolomitization.

Hydrogeochemical profiles along the flow path BB' (Fig. 16) depicts the presence of mixing process in well no. 113. The well falls on the Abqaiq structure where the confining layer drops in thickness. Umm Er Radhuma aquifer water dilutes the overlying Khobar aquifer water due to leaky-confining layer and pressure head differences.

In the northern part of the study area, profile CC', there is an absence of prominent geological structures. Also there is a clear absence of an appreciable ion exchange process (Fig. 17). All along the flow path the water body undergoes a normal hydrogeochemical evolution.

CONCLUSIONS

(1) The TDS in Khobar aquifer varies from less than 1000 mg/l to more than 7000 mg/l. The lowest TDS values are found in the southern part of the study area, while the TDS increases northward.

(2) Four hydrogeochemical facies, which are Na-Cl, Na-Ca-Cl, Na-Cl-Ca- SO_4 and $Ca-SO_4$, are identified in Khobar aquifer. The predominant water type is the Na-Cl facies.

(3) Khobar aquifer is oversaturated with respect to calcite and dolomite, and undersaturated with respect to gypsum.

(4) In the north of Jubayl, the water body undergoes a normal hydrogeochemical evolution, however, the presence of nascent, chemical-gradient dependent seawater intrusion could not be ruled out. Along Abqaiq flowpath, there is a dilution effect in Khobar aquifer. Near Hofuf the evident process is the gypsum dissolution and concomitant dedolomitization.

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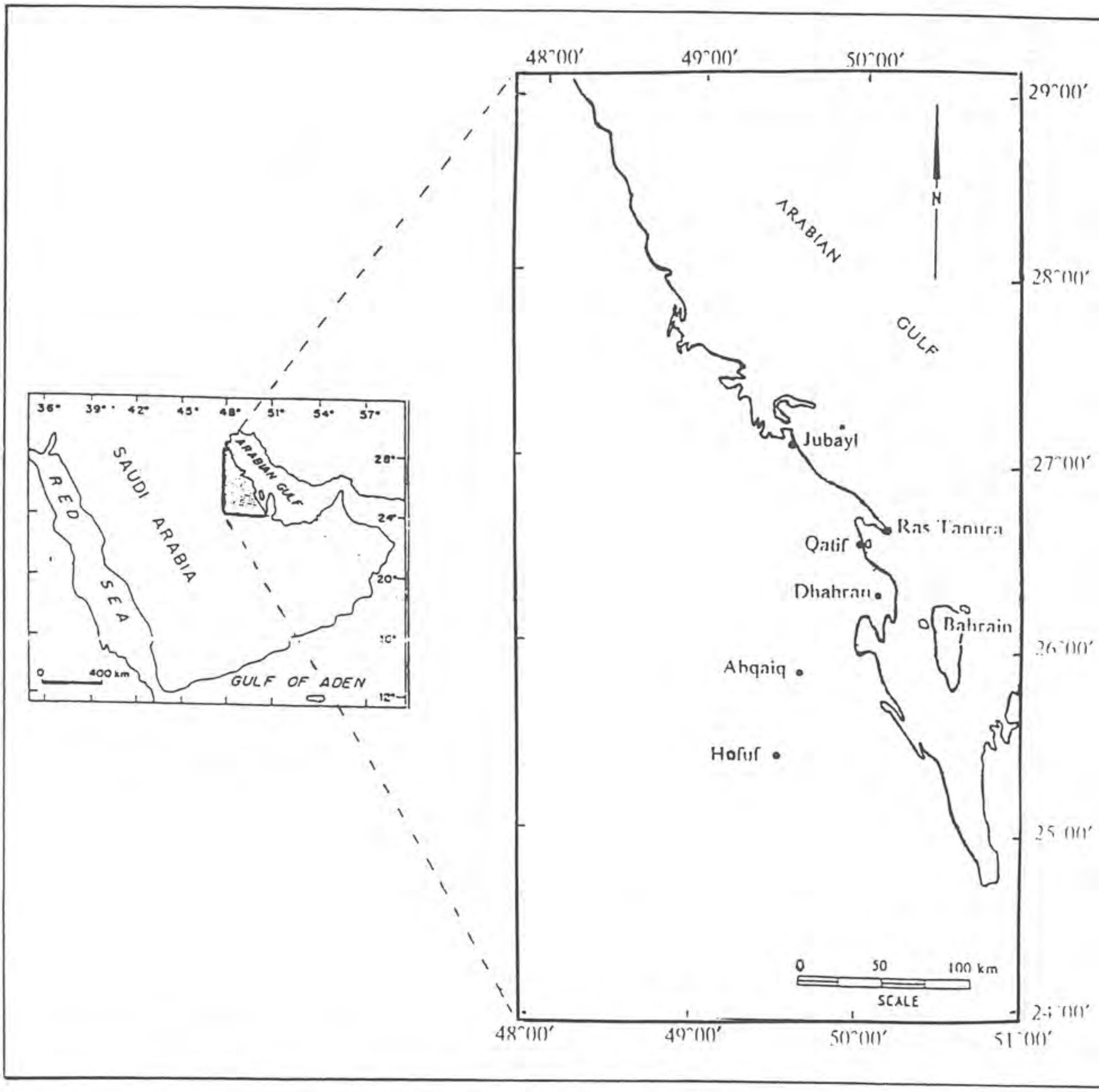

Figure 1. Location map of the study area.

AGE		FORMATION	MEMBER	ROCK UNIT	GENERALIZED LITHOLOGIC DESCRIPTION	THICKNESS (m)	HYDROGEOLOGIC UNIT
QUATERNARY		Surficial Deposits			Gravel, Sand and Silt	3 - 30	Variable productivity depending on recharge
Tertiary	Neogene	Hofuf			Sandy marl and sandy limestone	0 - 95	Neogene Aquifer (Limited productivity)
		Dam			Sandy marls, silty clays and skeletal limestones	0 - 100	
		Hadrukh			Silty marls and shales, sandy limestones	0 - 90	
	Eocene	Dammam	Alat	Limestone	Skeletal detrital limestone	0 - 110	Aquifer
				Marl	Dolomitic marls with limestone intercalations (orange color)	0 - 35	Aquitard
			Khobar		Skeletal, detrital, porous and friable limestones, dolomitic limestone	0 - 75	Aquifer
			Alveolina Limestone		Limestones interbedded with shales and marls	0 - 20	Aquitard
			Midra and Saila shales		Blue and dark grey, fissile shales with gypsiferous lenses	0 - 20	
		Rus			Chalky limestones; anhydrite, dolomitic limestones & shales	20 - 110	
	Palaeocene	Umm Er Radhuma			Partially dolomitized chalky limestones, detrital, skeletal limestone	Average 320	Aquifer
Cretaceous		Aruma			Variocolored limestone, subordinate dolomite and shale		Poor Aquifer

Figure 2. Generalized lithostratigraphic sequence of the study area (after Italconsult,1969)

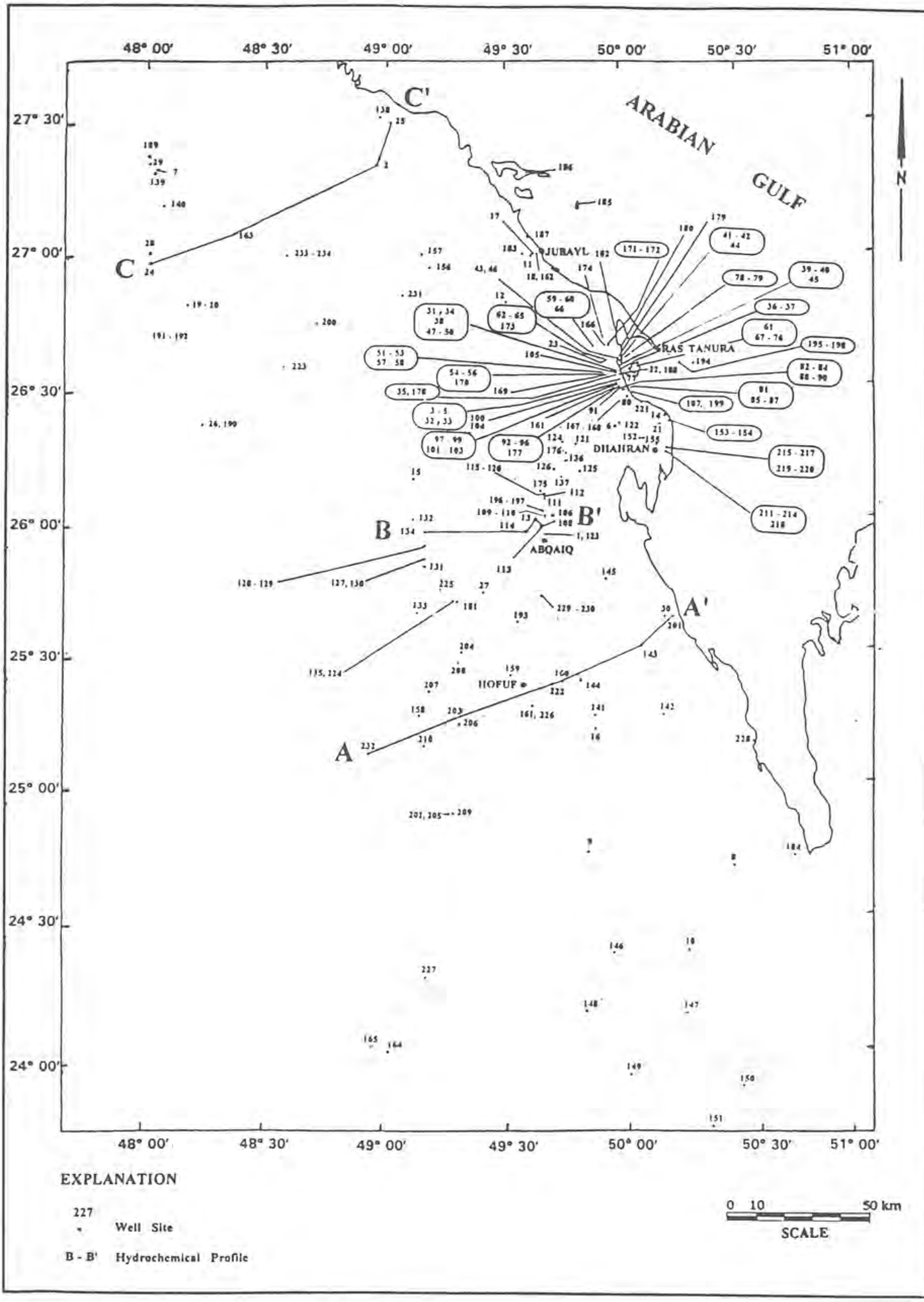

Figure 3. Water samples location map of the Khobar aquifer.

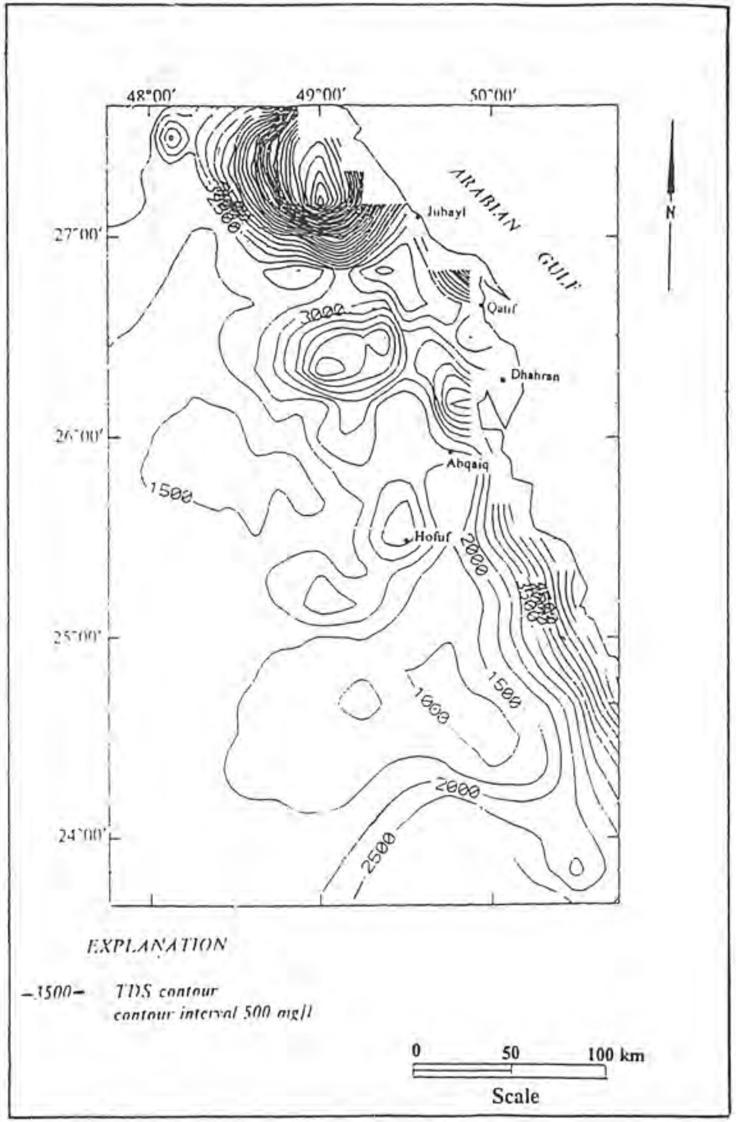

Figure 4. Total dissolved solids map of waters in the Khobar aquifer.

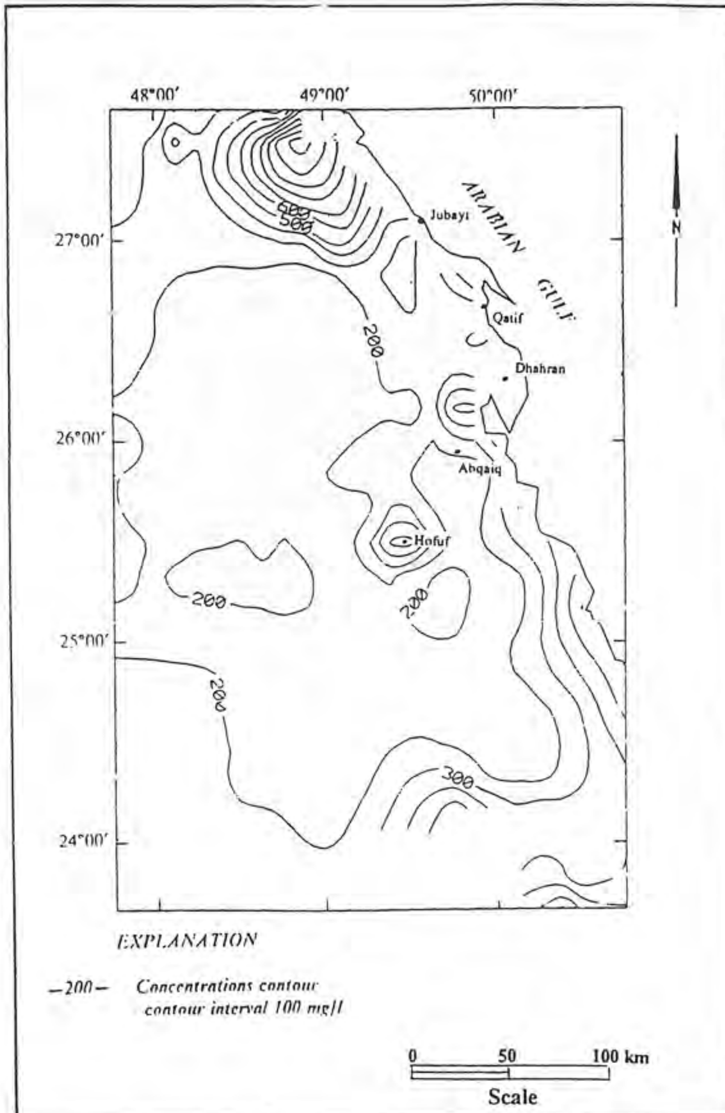

Figure 5. Areal distribution map of calcium (mg/l) in Khobar aquifer.

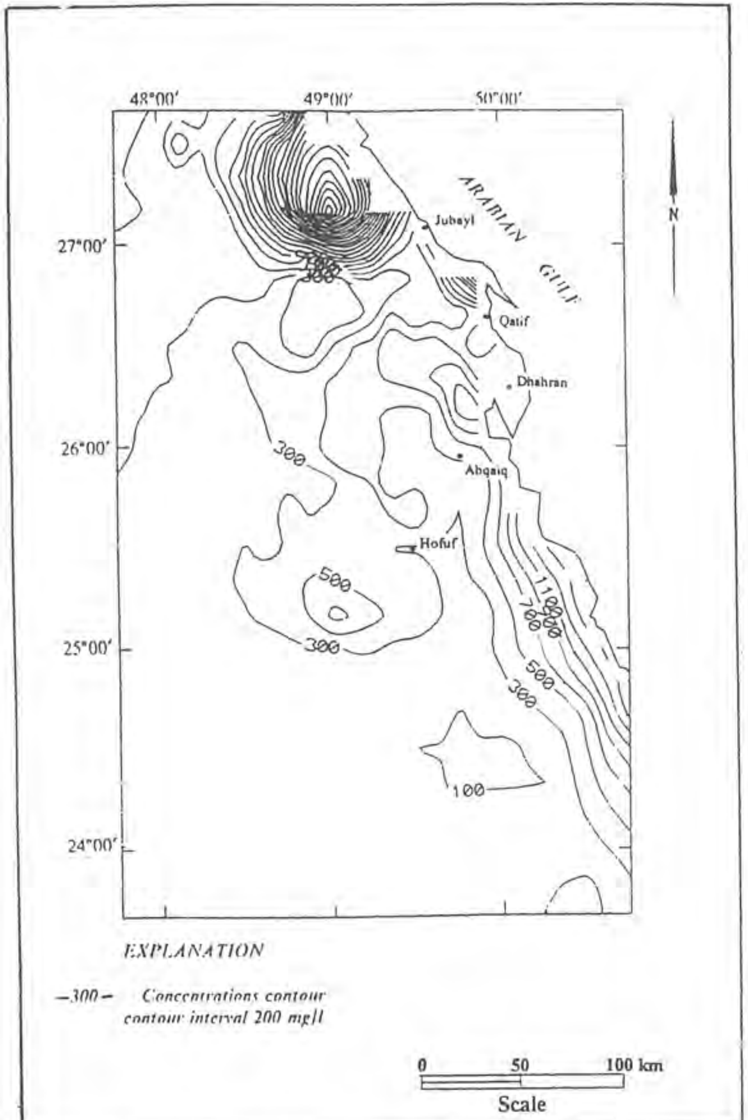

Figure 6. Areal distribution map of sodium (mg/l) in Khobar aquifer.

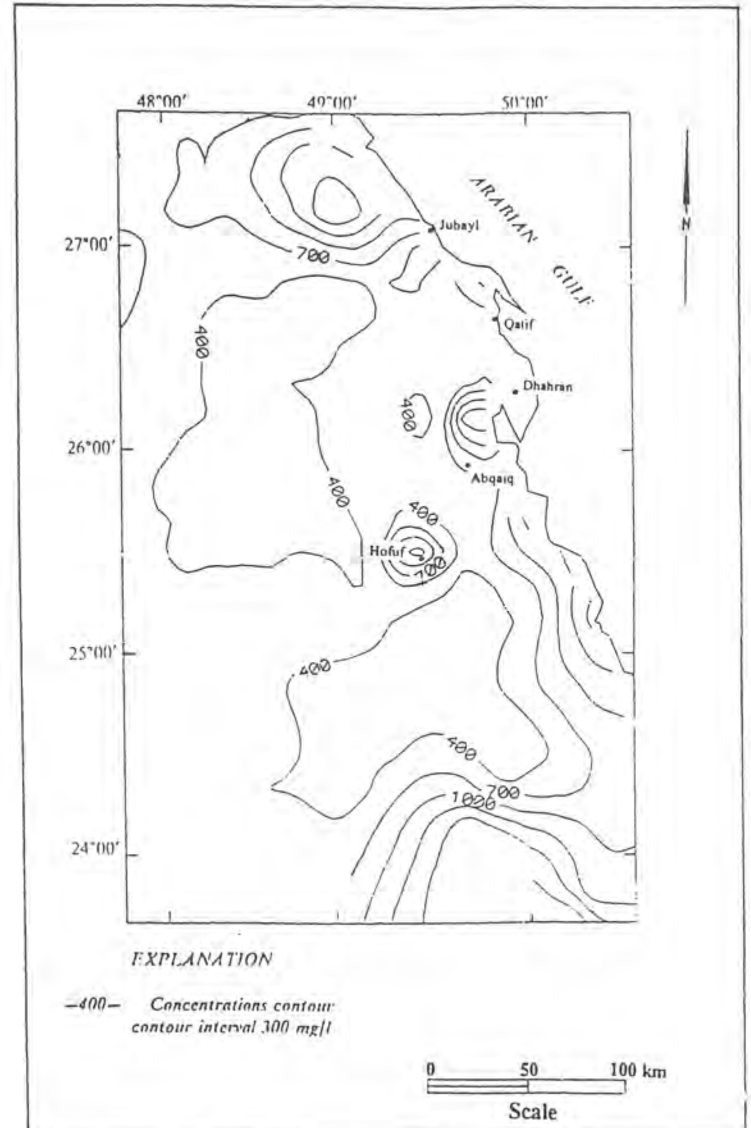

Figure 7. Areal distribution map of sulfate (mg/l) in Khobar aquifer.

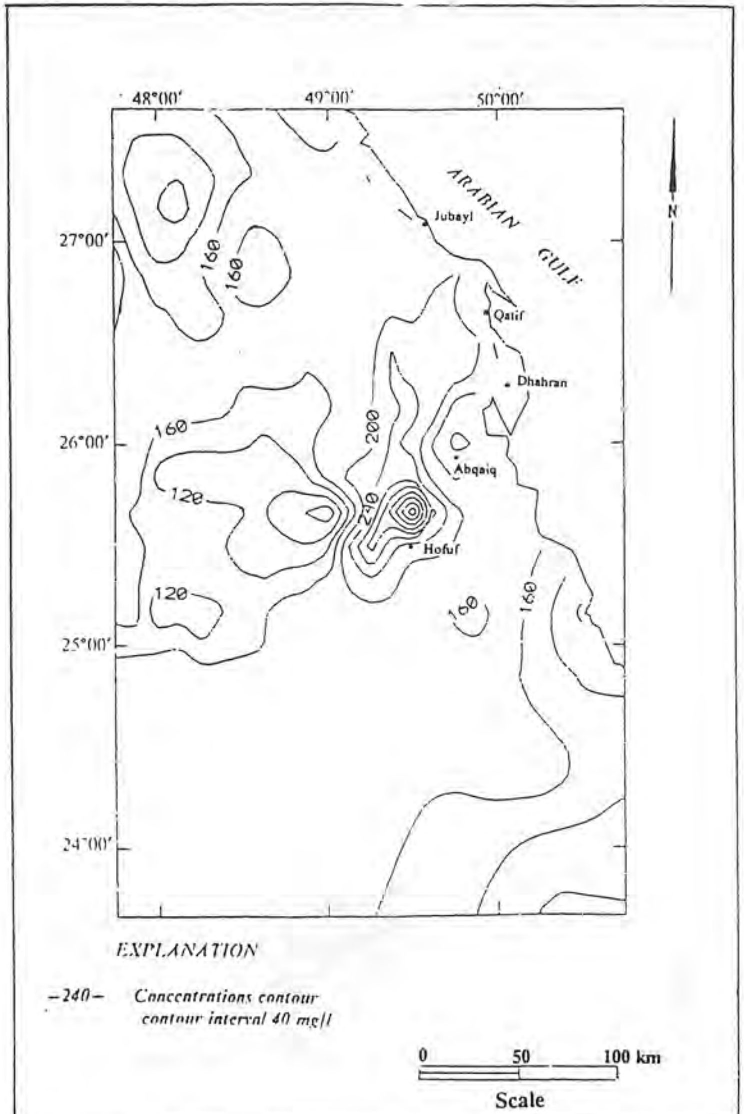

Figure 8. Areal distribution map of bicarbonate (mg/l) in Khobar aquifer.

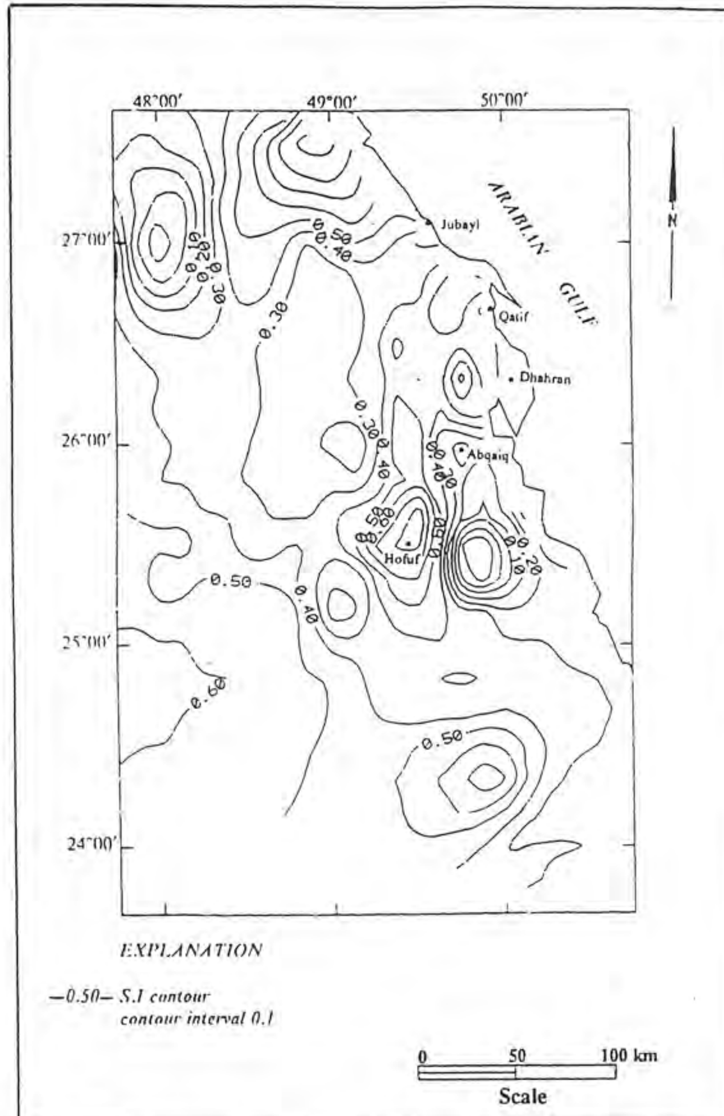

Figure 9. Calcite saturation index map of the Khobar aquifer.

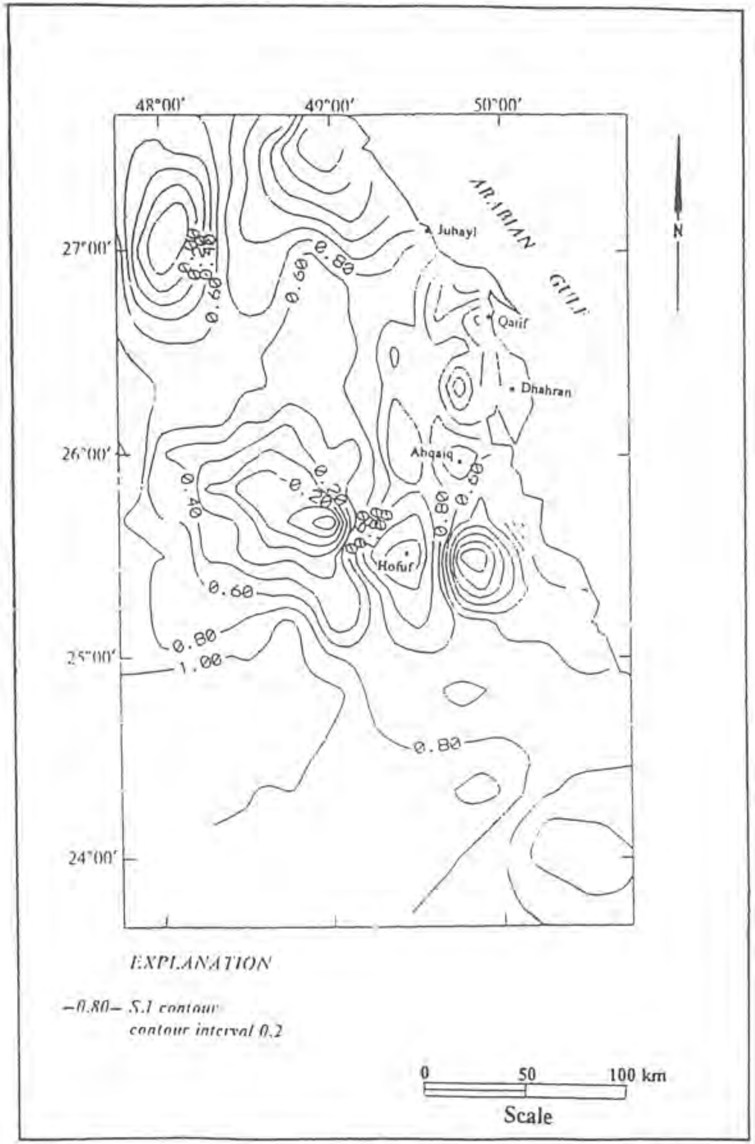

Figure 10. Dolomite saturation index map of the Khobar aquifer.

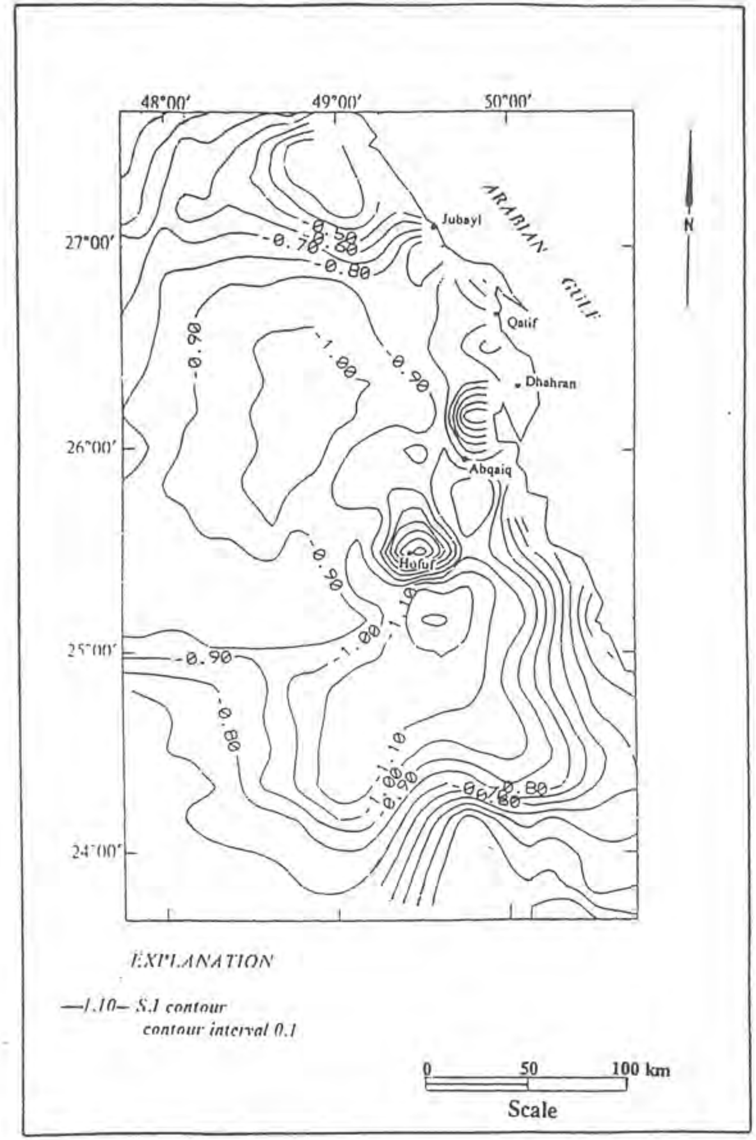

Figure 11. Gypsum saturation index map of the Khobar aquifer.

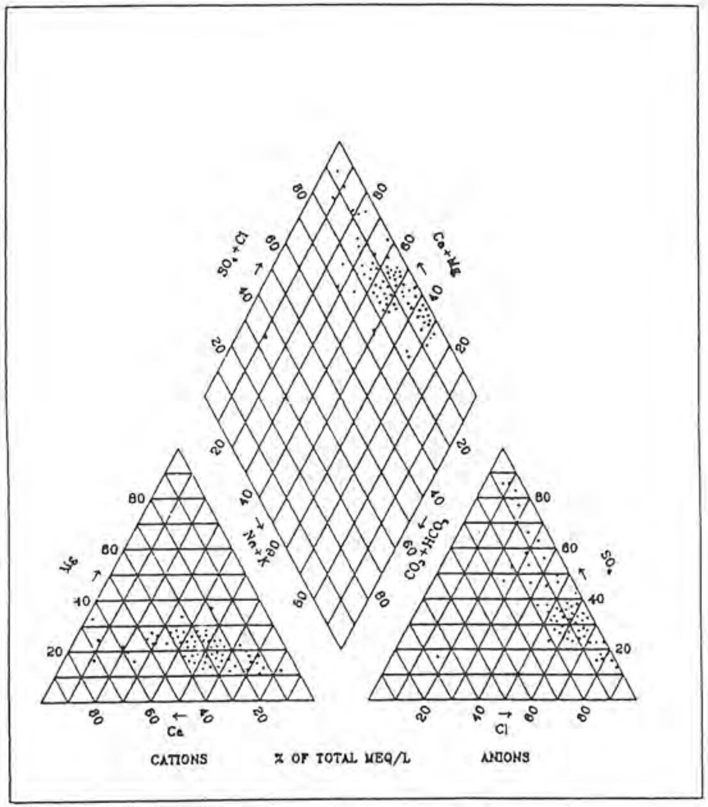

Figure 12. Trilinear plot of water samples of the Khobar aquifer.

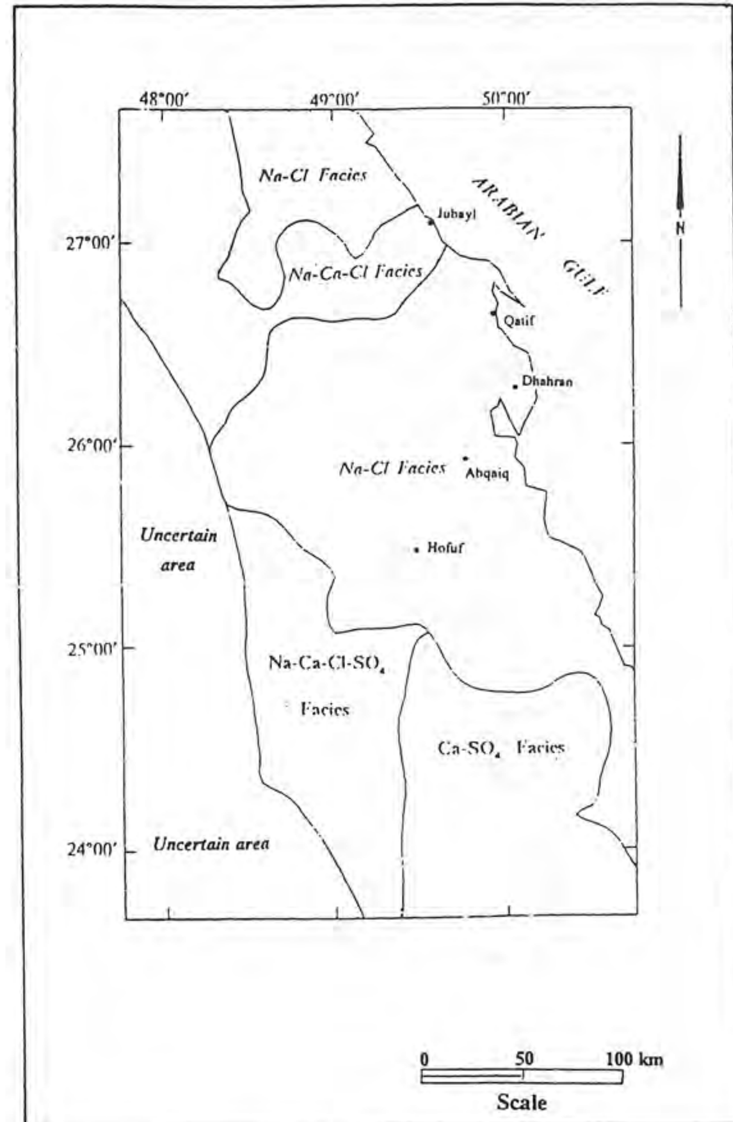

Figure 13. Hydrogeochemical facies map of the Khobar aquifer .

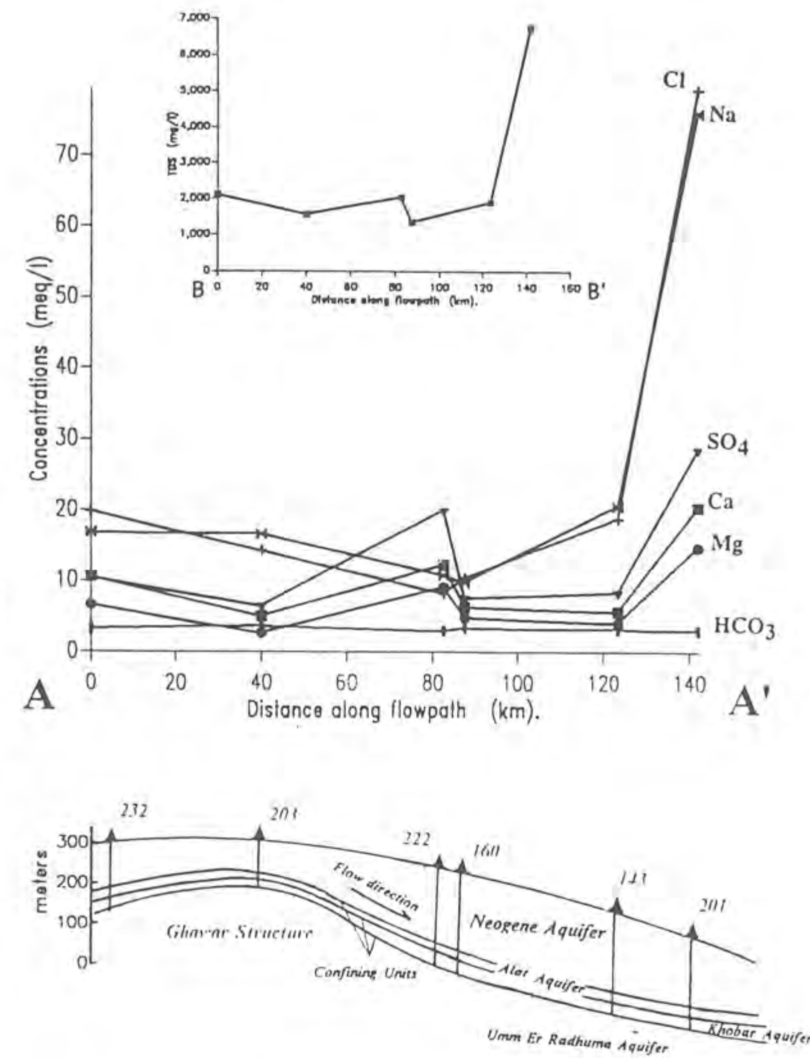

Figure 14 Hydrogeochemical parameters along Hofuf flowpath in Khobar aquifer (schematic profile, see Fig. 1 for the profile location)

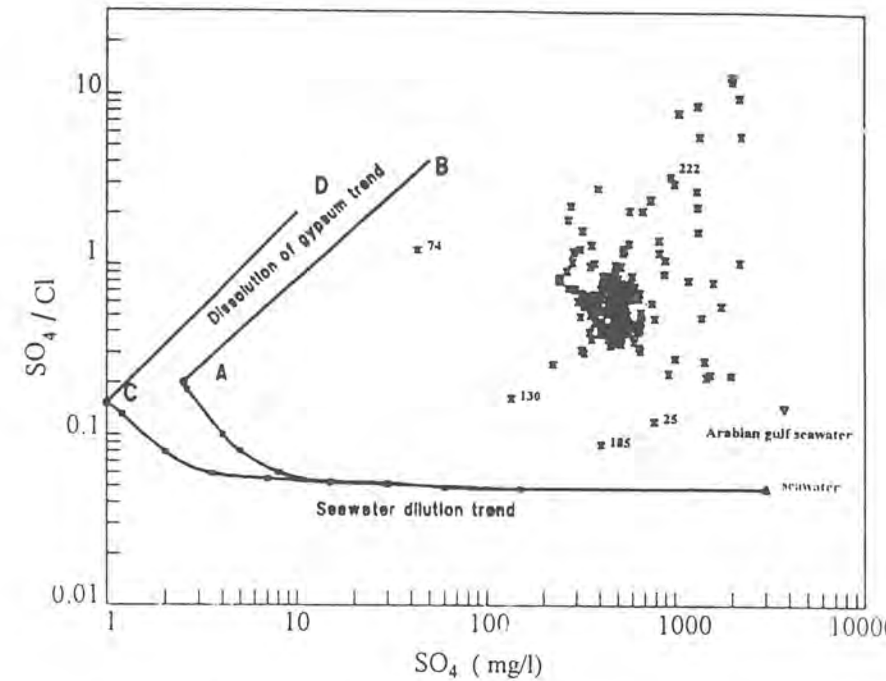

Figure 15 Relation of sulfate/chloride ratio to sulfate (mg/l) in the Khobar aquifer (the base diagram is modified from Sprinkle, 1989; Rightmire and others, 1974). Points A and C represent assumed SO_4 and C l concentrations in dilute ground water.

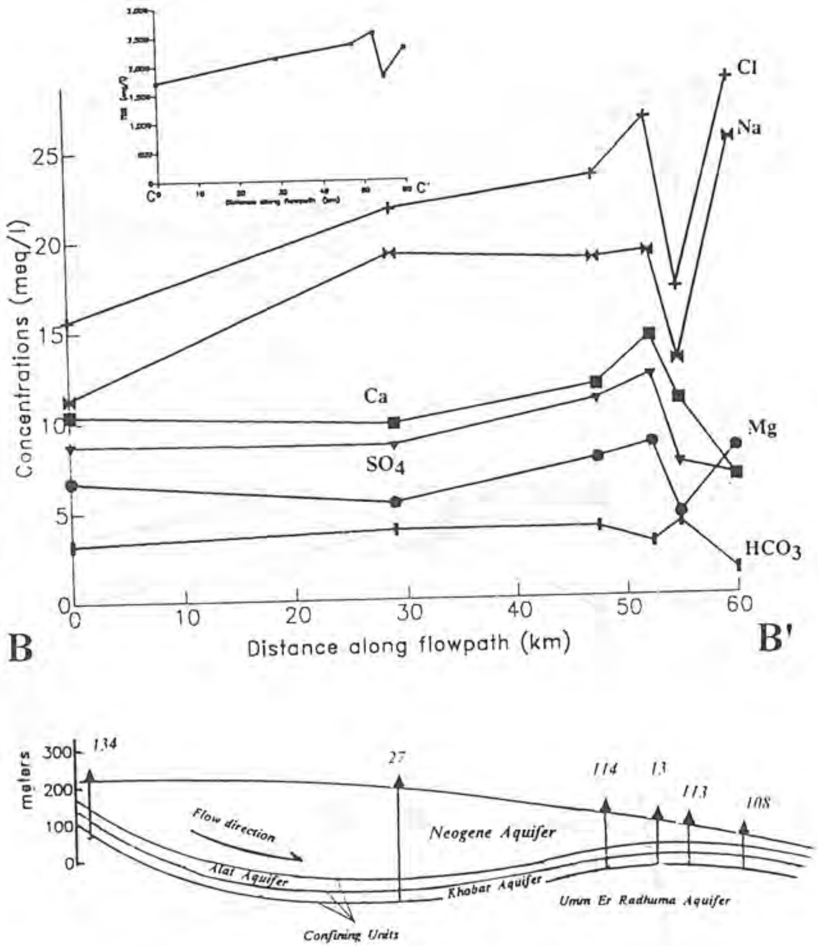

Figure 16 Hydrogeochemical profiles along Abqaiq flowpath in Khobar aquifer (schematic profile, see Fig. 1 for the profile location)

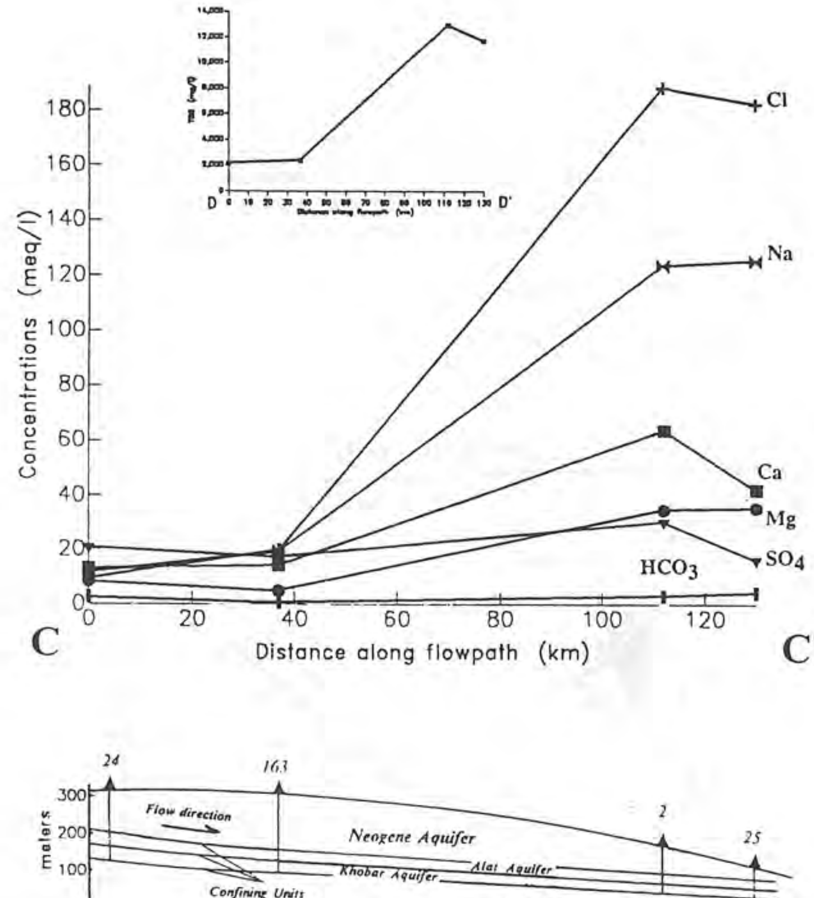

Figure 17 Hydrogeochemical profiles along a flowpath at north of Jubayl in Khobar aquifer (schematic profile, see Fig. 1 for the profile location)

Temperature Effect in Discharge Tests of Deep WaterWells

M.W. Kawechi

TEMPERATURE EFFECT IN DISCHARGE TESTS ON DEEP WATER WELLS

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Abstract

Discharge tests on water wells usually make use of measurements of the water level or wellhead pressure. A problem in deep wells in Saudi Arabia and elsewhere is that such measurements cannot be interpreted directly because they are affected by temperature changes of the water within the well. The problem can be avoided by monitoring the bottom-hole pressure or using a non-discharging observation well, but these options are not always available. Therefore this paper tests a method of correcting for the temperature effect which is based on a simple model for the cooling of the water in the well during recovery. It is shown that the method allows the aquifer transmissivity to be calculated from wellhead recovery data in temperature-affected wells provided the temperature effect is not much greater than the residual hydraulic drawdown.

Keywords: temperature, recovery, well tests, transmissivity.

Introduction

Discharge tests on water wells usually rely on measurements of the water level in pumped wells or wellhead pressure in flowing wells. A problem with deep wells in Saudi Arabia and elsewhere is that such measurements cannot be interpreted directly because they are affected by temperature changes of the water column within the well during the test. One way of avoiding the problem is to monitor the bottom-hole pressure in the test well (which is not affected by temperature changes of the water column) instead of the water level or wellhead pressure. In aquifer tests another option is to use an observation well because temperature changes in non-discharging observation wells are usually negligible. However both these options, especially deep observation wells, are expensive and consequently not always available.

Recognising the problem, Kawecki (1994) proposed a method of correcting for the temperature effect in wells when bottom-hole monitoring facilities or observation wells are not available. The method is based on an assumed exponential model for the cooling of the water column during recovery and it requires only the initial and stabilised discharge temperatures to be measured during the test in addition to the usual measurements. Provided that the assumptions are valid, application of the method to temperature-affected recovery data allows the aquifer transmissivity and the average temperature of the water column in the well at rest to be determined. Using the method, Kawecki (1994) obtained apparently realistic results in a variety of situations. However the validity of the assumed cooling model was not tested.

The purpose of this paper is to examine the validity of the proposed temperature correction method by presenting results from a specially designed test carried out on a single flowing well. To separate the temperature effect from the hydraulic drawdown, both wellhead pressure and bottom-hole pressure were monitored in the test. Temperature logs of the well at rest and at the end of the monitored recovery period were also made.

Theory and Background

Because the test was carried out on a flowing well, the equations presented below are for the case of a flowing well only. Corresponding equations for a pumped well can be similarly derived (Kawecki, 1994).

In a well at rest the average temperature of the water standing in the casing is generally lower than the temperature of the water in the aquifer because of cooling by the surrounding shallower formations. When the well is flowed the water in the casing is displaced by warmer water from the aquifer. During subsequent recovery the water in the casing cools again as it loses heat to the shallower formations, and conditions gradually return to hydraulic and thermal equilibrium. The relationship between wellhead pressure and down-hole pressure at any time during recovery is:

$$P_s(t'') = P_a(t'') - L\,\bar{\rho}(t'') \tag{1}$$

where P_s is the shut-in well head pressure

P_a is the pressure in the aquifer at a depth L below the wellhead

$\bar{\rho}$ is the average density of the water column from the wellhead to depth L

t" is the time since the start of recovery

In the following analysis L will be defined as the length of the water column from the wellhead to the top of the open interval of the well. Note also that the average density, $\bar{\rho}$, is a function of time because it depends on the time variant temperature profile in the water column.

When the shut-in well is in hydraulic and thermal equilibrium, equation (1) becomes:

$$P_{so} = P_{ao} - L\,\bar{\rho}_o \tag{2}$$

The residual drawdown at the wellhead (in units of pressure) is:

$$\Delta P_s = P_{so} - P_s \tag{3}$$

and in the aquifer at depth L below the wellhead:

$$\Delta P_a = P_{ao} - P_a \tag{4}$$

Combining equations (1) to (4) leads to:

$$\Delta P_a - \Delta P_s = L\,(\bar{\rho}_o - \bar{\rho}) \qquad (5)$$

in which the term on the right hand side is the temperature correction which needs to be added to the wellhead residual drawdown to obtain the true aquifer residual drawdown.

To derive a specific formula for the temperature correction, Kawecki (1994) assumed that the cooling of the water column in the well during recovery is according to Newton's law of cooling (for a description of the law see, for example, Chapple, 1979). This leads to the following equation for the average temperature of the water column:

$$\Theta(t'') = \Theta_s + (\Theta_p - \Theta_s)\,e^{-mt''} \qquad (6)$$

where Θ is the average temperature of the water column at time t" (averaged over the length of the water column)

Θ_s is the average steady temperature of the water column in the well at rest

Θ_p is the average temperature of the water column in the well at the end of flowing

m is a time constant describing how quickly the water column loses heat to the surrounding formations

Kawecki (1994) also assumed that within the relatively narrow range of temperatures typically occuring in a well the relationship between water temperature and water density (Figure 1) can be approximated by a linear function of the form:

$$\rho(\theta) \approx a - b\theta \qquad (7)$$

where $\rho(\theta)$ is the density of water at temperature θ and a and b are constants. From this it can be shown that for any temperature profile in the water column:

$$\bar{\rho} \approx \rho(\Theta) \qquad (8)$$

Combining (6), (7) and (8) leads to:

$$\bar{\rho}(t'') = \rho(\Theta_s) + \left\{ \rho(\Theta_p) - \rho(\Theta_s) \right\} e^{-mt''} \qquad (9)$$

Substituting (9) in (5) and noting that $\bar{\rho}_o = \rho(\Theta_s)$ gives:

$$\Delta P_a - \Delta P_s = L \left\{ \rho(\Theta_s) - \rho(\Theta_p) \right\} e^{-mt''} \qquad (10)$$

The expression for the temperature correction on the right hand side of equation (10) contains three unknown terms, Θ_p, Θ_s and m. Kawecki (1994) explained that the term Θ_p can usually be estimated by measuring the stabilised discharge temperature and showed how the terms Θ_s and m can then be determined uniquely by trial and error in the recovery analysis. Once the temperature correction is fully specified it is possible to calculate the aquifer transmissivity from the corrected residual welhead drawdowns. Knowing Θ_p and Θ_s it is also possible to correct wellhead drawdowns observed during the flowing part of the test. The temperature correction of equation (10) is therefore very useful, but it does depend on the validity of the assumed cooling model of equation (6).

The Flow Test

The flow test to examine the validity of the assumed temperature correction in equation (10) was carried out on a well tapping a sandstone aquifer in eastern Saudi Arabia. Before flowing the well a down-hole pressure and temperature recording gauge was lowered to the bottom, stopping every 61m to determine the temperature profile of the water column at rest and in thermal equilibrium. The gauge was then set at the top of the open interval at depth 1183 m and the well was flowed at maximum rate for 440 minutes. This was followed by 2237 minutes of recovery during which the pressure at the wellhead was monitored by a mechanical pressure gauge with resolution to 0.01 psi (0.07 kPa or about 7 mm of head of water). Simulataneously the down-hole gauge recorded the pressure at 1183m depth. At the end of the monitored recovery period the down-hole gauge was retrieved stopping every 122 m to record the temperature profile. The temperature profiles at rest and at the end of the test are shown in Figure 2 and selected test data are presented in Table 1.

Table 1 : Summary of Test Data

Initial shut-in wellhead pressure, P_{so} = 681.2 kPa
Length of water column, L = 1183 m
Flow rate = 0.7 m^3/min rising gradually to 0.9 m^3/min.
Flowing wellhead pressure ≈ 20 KPa
Flow period = 440 minutes
Recovery period = 2237 minutes
Average temperature of water column at rest, Θ_s (from temperature log) = 50.3 °C
Initial flowing temperature = 29°C
Final flowing temperature = 61°C
Temperature at depth 1183 m (aquifer temperature) = 69.1°C
Average temperature of water column at end of monitored recovery period (from temperature log) = 51.7°C

Figure 3 shows the observed wellhead and down-hole residual drawdowns plotted against time. The wellhead drawdown is usually large and negative because it mainly represents the significant temperature effect in this well. At the start of recovery it is -81.43 kPa which is about -8.3 m of water. In contrast the down-hole drawdown, which represents the hydraulic residual drawdown in the aquifer, is relatively small and positive. At the start of recovery the down-hole drawdown is 1.93 kPa or about 0.2 m of water and complete hydraulic recovery is reached after only 70 minutes. The reason for the small residual aquifer drawdown is a high aquifer transmissivity combined with low flow rates during the test (the flow was limited by high well losses). Approximation of the observed flow pattern by a step function and analysis of the residual aquifer drawdowns by the stepped discharge recovery method (Kawecki, 1993) gave an aquifer transmissivity of 2260 m^2/day.

The wellhead data plotted in Figure 3 depart from the the general trend of the curve between t"=550 mins and t"=1150 mins, and again from t"=1950 mins to the end of the monitoring period. The departures correspond to daylight hours and there are no simultaneous changes in the down-hole data. They are therefore thought to be partly the effect of the sun heating the water column (there were 3 m of flowpipe between the wellhead and the shut-in valve which could act as a heat collector) and partly the effect of the sun on the pressure gauge. Note also that the wellhead drawdown becomes positive towards the end of the monitored period. Since there is no hydraulic drawdown in the aquifer by this time and the temperature log shows that the water column is warmer than at rest (Table 1 and Figure 2), the observed wellhead drawdown should be negative. The positive values suggest that the measured wellhead static pressure, P_{so}, is too high, possibly because of the effect of the sun noted above.

Analysis of the Results

If the assumed cooling model is valid then, by rewriting equation (10), the sum of the wellhead pressure and down-hole drawdown should be an exponential function of the form:

$$P_s + \Delta P_a = A + B\, e^{-mt''} \qquad (11)$$

where $A=P_{so}$ and $B = L\{\rho(\Theta_s - \rho(\Theta_p)\}$. In the test well P_{so} was measured as 681.2 kPa. For calculation of B, the values of Θ_s and L are in Table 1 whilst the average temperature of the water column at the start of recovery, Θ_p, can be assumed to be the mean of the final flowing temperature and aquifer temperature (Figure 2 and Table 1). Therefore $\Theta_p = 65.1°C$. Using the temperature-density data for water reported by Anderson (1984) and reproduced in Figure 1 leads to B = 93.28 kPa. Since Anderson's data are presumably for fresh water, the densities were multiplied by the specific gravity of the water in the test well which was mersured as 1.0039 at 18°C (however the effect of this on the calculated value of B was not significant).

To ascertain whether the temperature effect does indeed follow the assumed relationship, least squares analysis was used to fit the best curve described by equation (11) to data for the entire monitored recovery period, the first 500 minutes of recovery and the first 125 minutes of recovery. Because the measured value of P_{so} was thought to be in error, the least squares analysis included the term A of equation (11) as an unknown. In each fitting a value of A was first assumed and B and m were determined. The value of A was then adjusted and the procedure repeated until a combination of A, B and m was found which gave the lowest sum of squared errors.

The derived exponential curves are plotted in Figure 4 and the corresponding values of A, B and m are presented in Table 2. Values of Θ_S calculated by substituting Θ_P = 65.1°C in the expression for B are also included in Table 2. The fit of the curves ranges from excellent for the first 125 minutes to fair for the entire monitored recovery period with a good fit for the first 500 minutes. The reasonable fit for the entire period has been achieved despite the anomalies in the field data ascribed to the effect of the sun. However the data in Table 2 show that all the curves are different from each other and from the assumed model. The cooling of the water column can therefore be closely approximated by an exponential relationship, but not necessarily the assumed exponential relationship of equation (6).

Table 2 : Parameters of equation (11) from least squares fitting to test data

Data used	A (kPa)	B (kPa)	m (min^{-1})	Θ_S (°C)
First 125 mins	711.2	54.16	0.00784	56.6
First 500 min	697.0	66.35	0.00502	54.7
All	679.9	76.67	0.00250	53.0
Assumed model	681.2*	93.28*	-	50.3*

* From measurements in the test well

Discussion

It is reasonable to assume that the possibility of approximating the cooling by an exponential relationship applies not only to the test well but also in many other cases. It is therefore not surprising that Kawecki (1994) found that his proposed method, which essentially aims to determine an exponential function which 'straightens out' curvature due to temperature effect in a recovery plot, did appear to produce successful results. However it must be stressed that the exponential relationship is likely to be empirical. By removing the curvature of the recovery plot caused by the temperature effect, the empirical correction does allow the aquifer transmissivity to be calculated, but the value of Θ_S determined by the method of Kawecki (1994) is likely to be different from the true average temperature of the water column at rest (as demonstrated in Table 2). Temperature correction of drawdowns during the discharging part of a test will therefore generally not be accurate unless the true average temperature of the water column at rest can be estimated by other means.

It should also be noted that relatively small departures from an exponential relationship will be significant if the temperature effect is much greater than the hydraulic drawdown. In such cases the method of Kawecki (1994) cannot be applied reliably and downhole tools must be used to measure the temperature profile and to monitor bottom-hole pressure. This is certainly the case in the test well where the transmissivity was successfully determined from bottom-hole pressure (from residual drawdowns which were no greater than 0.2 m) but could not be determined from temperature-corrected wellhead pressure measurements.

To examine the results which can be obtained by the method of Kawecki (1994) in cases where the wellhead measurements are not dominated by the temperature effect, the temperature effect measured in the test well was converted from pressure units to metres of water and superimposed on theoretical residual drawdowns calculated for a hypothetical case by the Theis (1935) recovery equation:

$$s'' = \frac{Q}{4\pi T} \ln\left(\frac{t}{t''}\right) \tag{12}$$

where s" is the residual drawdown
Q is the steady discharge
T is the aquifer transmissivity
t is the time since the start of the discharge test
t" is the time since the start of recovery (as already defined above)

The input data for the hypothetical case were T=100 m^2/day, Q=3270 m^3/day and (t-t")=480 mins which gave residual drawdowns of a similar order to the temperature effect measured in the test well.

Figure 5 shows the Theis recovery analysis with temperature correction by the method of Kawecki (1994) for the entire recovery period, the first 500 mins and the first 125 mins. Relevant values of Θ_s and m from Table 2 were used for the temperature correction in each case, but similar values would be determined if the temperature correction procedure in Kawecki (1994) were followed. The data for the three cases plot on approximately parallel straight lines, the slope of which gives a good determination of the transmissivity. Note that the straight lines, when extrapolated, do not pass through the origin (t/t"=1, s"=0). The displacement from the origin is caused by the difference between the empirical and assumed cooling models and by the error in measuring the static wellhead pressure, P_{so}. A displacement can therefore generally be expected in the application of the method.

If there are no temperature logs, the method of Kawecki (1994) uses the initial discharge temperature to estimate the temperature at the top of the water column at rest and the stabilised discharge temperature to estimate Θ_p and the aquifer temperature. The latter assumes that there is no significant cooling of the water as it flows up the casing. While the results presented here demonstrate that this assumption was not valid in the test well, in the author's experience of other wells there is no significant cooling for higher discharge rates of 2 - 3 m^3/min.

Kawecki (1994) also demonstrates how unreserved application of the method can produce misleading results if there are hydraulic effects (eg. aquifer boundaries) which, in addition to the temperature effect, cause the recovery plot to depart from a straight line. The chances of identifying such hydraulic effects are increased if a reasonable range for the determined value of Θ_s is known. Table 2 above suggests that the value will not be lower than the true average temperature of the water column at rest, Θ_{st}. It is therefore important to estimate Θ_{st} in order to obtain a lower bound for Θ_s. Provided the discharge rate is sufficiently high to make cooling of the flow up the well insignificant, a first order estimate of Θ_{st} can be made as the mean of the initial and stabilised discharge temperatures. Failure to obtain a straight line recovery plot with $\Theta_s \geq \Theta_{st}$ in the temperature-corrected recovery analysis would then indicate possible hydraulic effects.

Conclusions

The results of the test show that aquifer transmissivity can be determined reliably from recovery data in a temperature-affected well by applying the temperature correction method proposed by Kawecki (1994). However the method is only applicable when the true residual aquifer drawdown is comparable to or greater than the temperature effect. Where the temperature effect is dominant, down-hole recovery measurements must be made.

It has also been shown that the estimate of the average temperature in the borehole at rest obtained by the method of Kawecki (1994) is likely to be too high and that the straight line in the temperature-corrected recovery analysis can generally be expected not to pass through the origin. To correct the drawdowns in the discharging part of the test for temperature, the average temperature in the borehole at rest will generally need to be estimated by other means.

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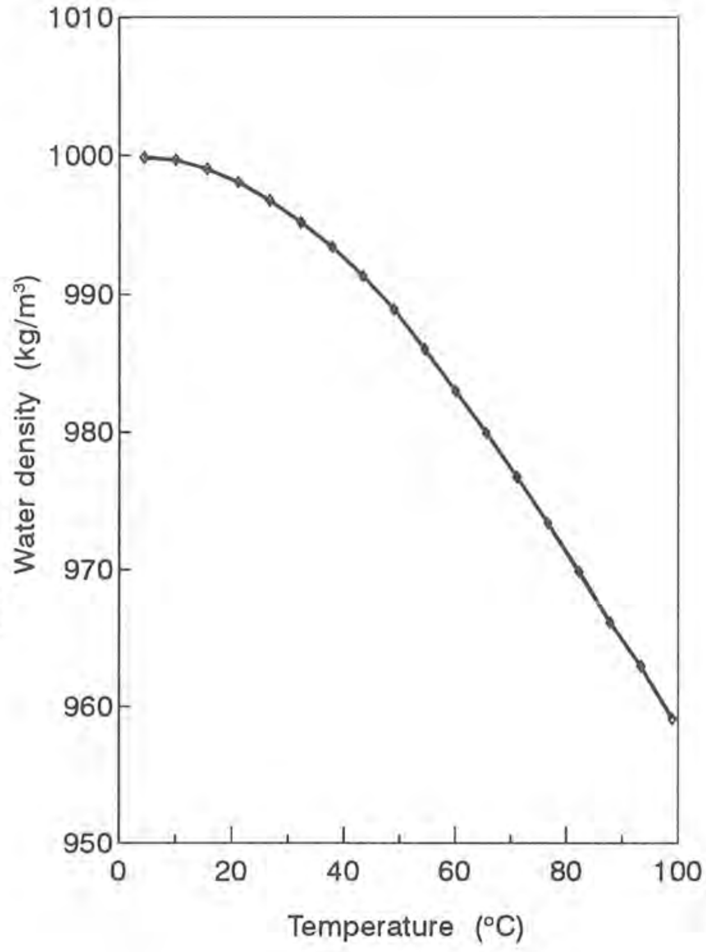

Figure 1 : Temperature-density data for water (source: Anderson, 1984)

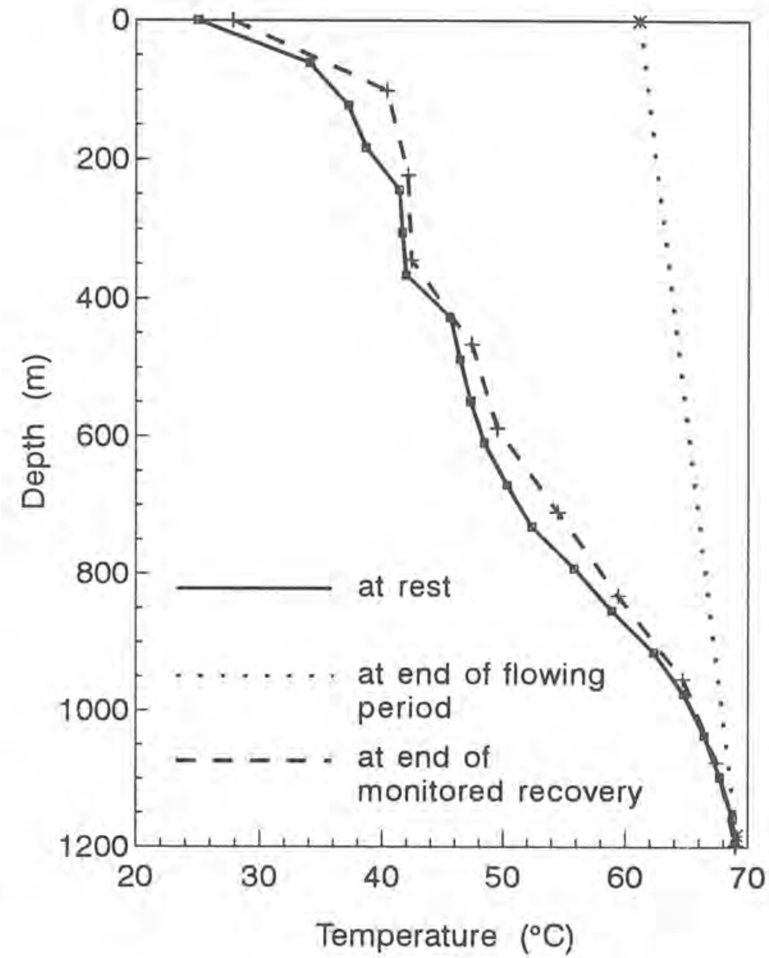

Figure 2 : Temperature profiles measured in the test well (note: flowing profile assumed from wellhead and down-hole measurements only)

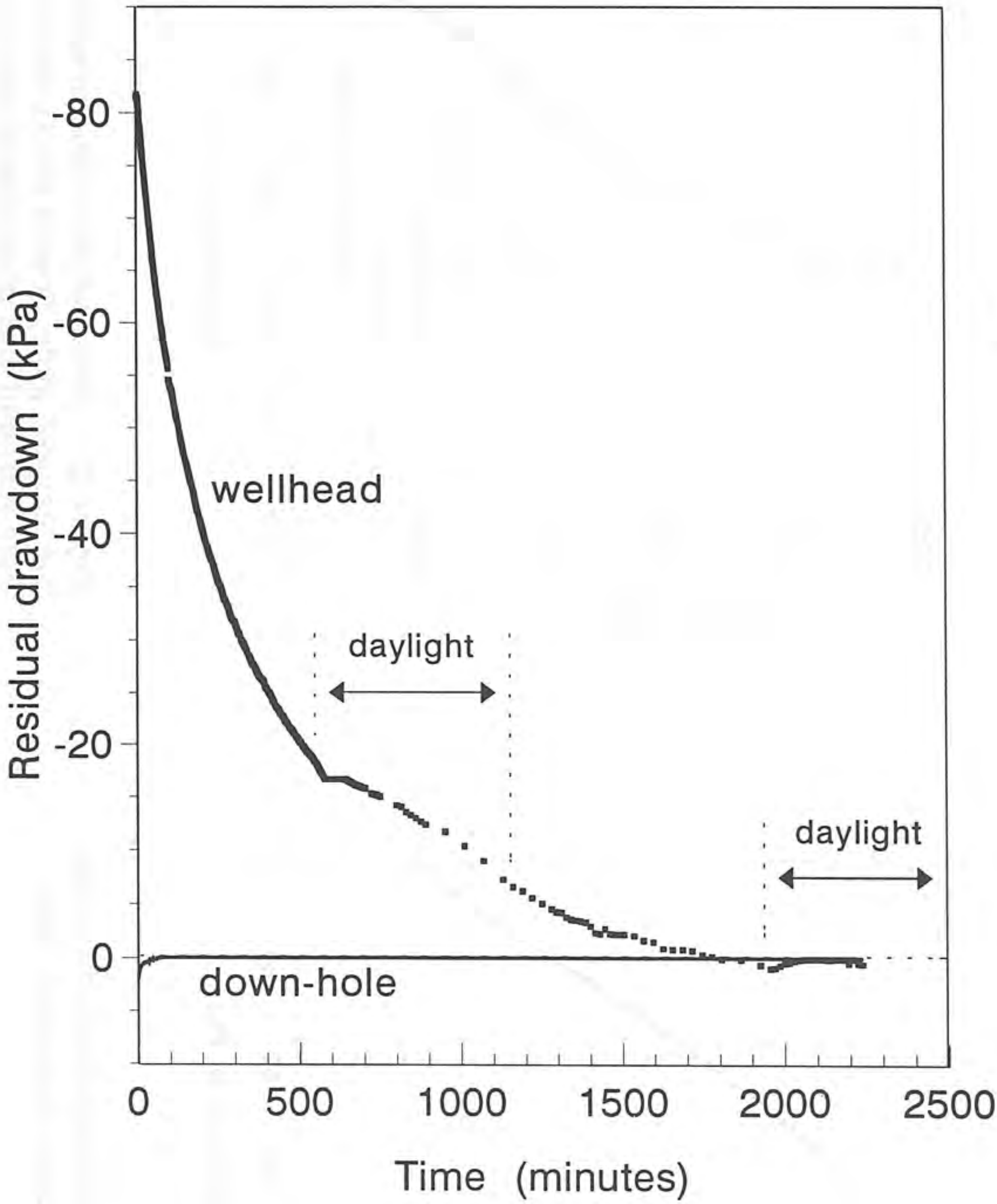

Figure 3 : Observed residual drawdowns in the test well at the wellhead and down-hole

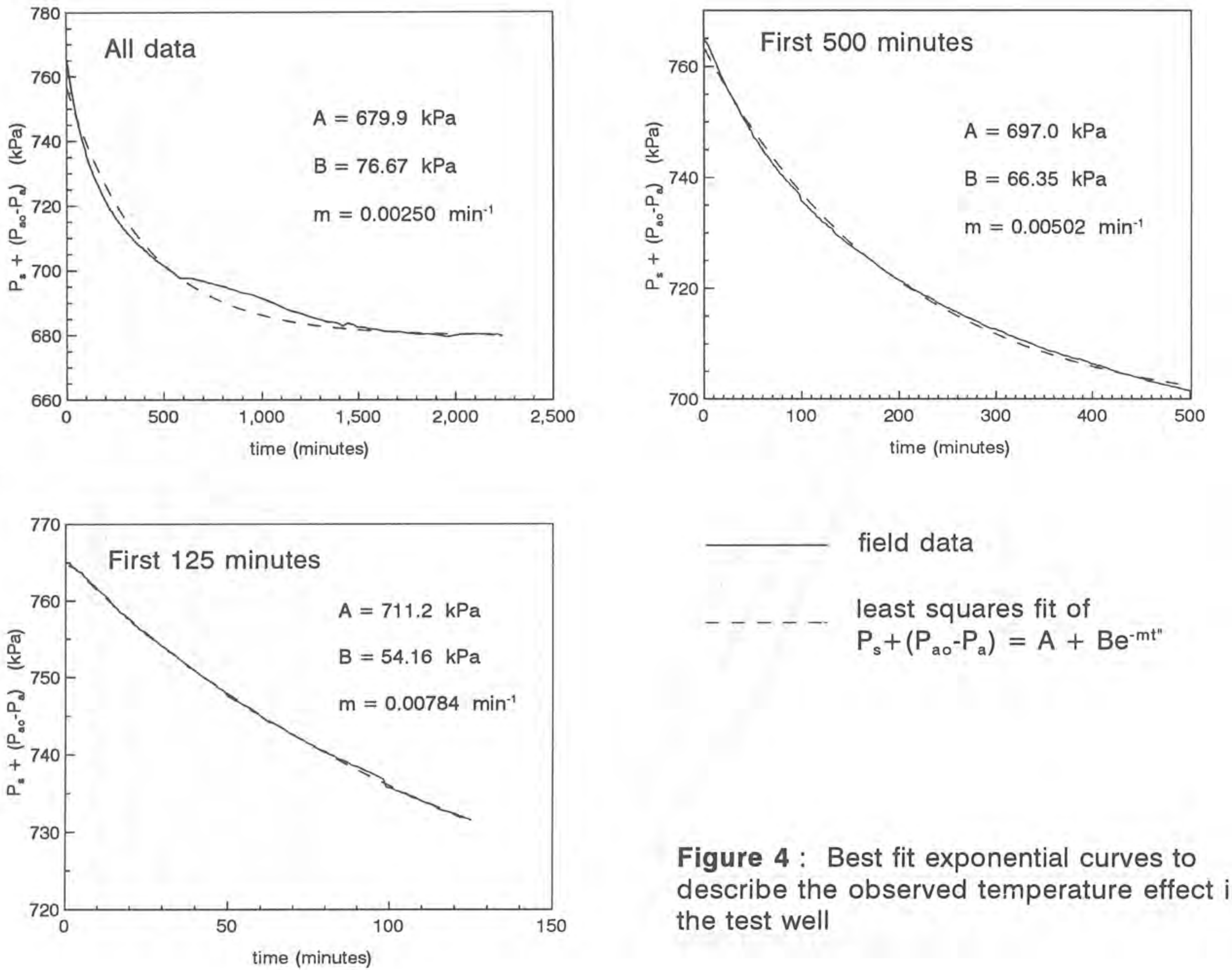

Figure 4 : Best fit exponential curves to describe the observed temperature effect in the test well

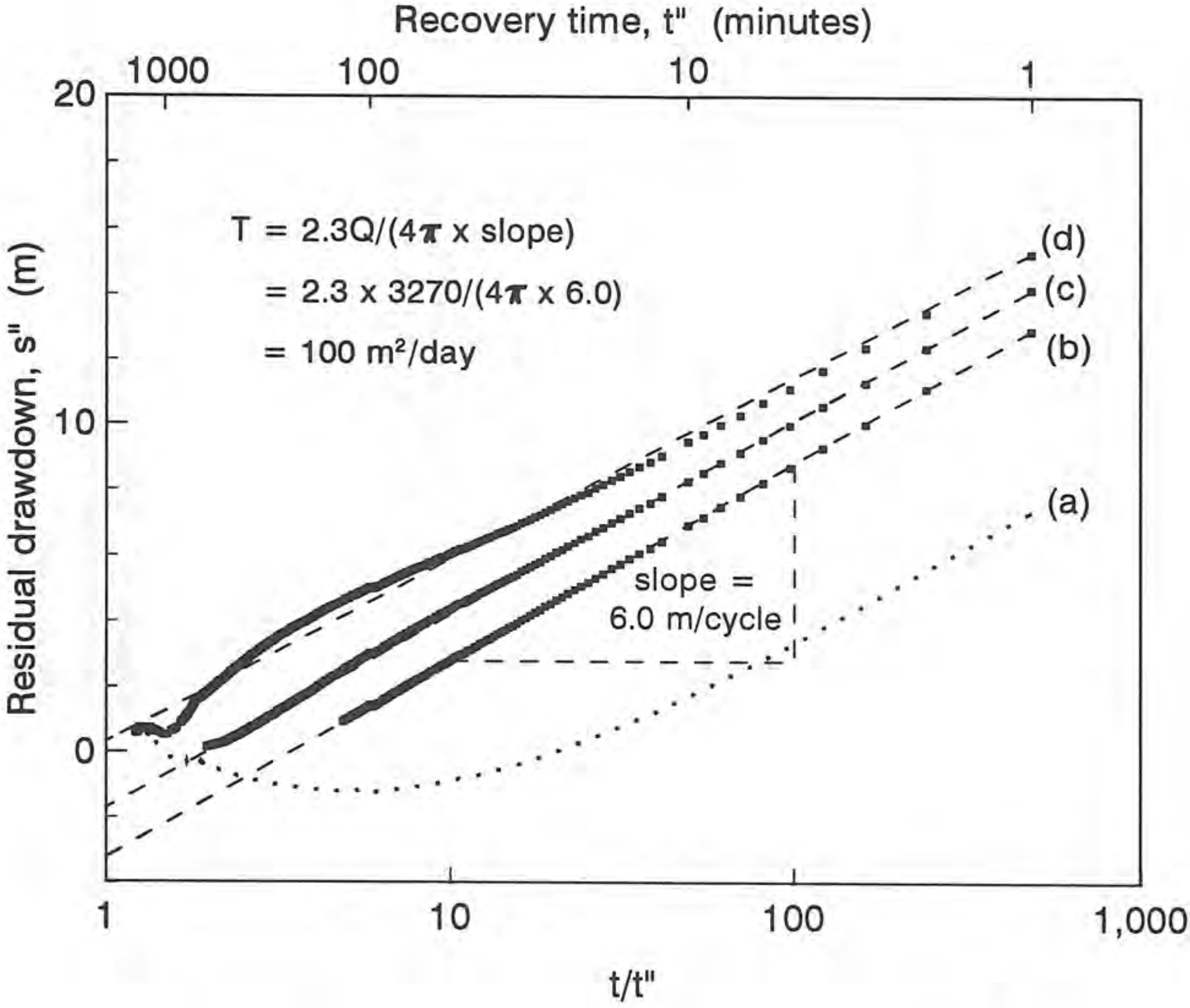

Figure 5 : Theis recovery analysis for hypothetical hydraulic drawdowns combined with temperature effect from the test well and corrected by the method of Kawecki (1994): (a) uncorrected for temperature; (b) data from first 125 minutes with Θ_s = 56.6 °C, m = 0.00784 min^{-1} ; (c) data from first 500 minutes with Θ_s = 54.7 °C, m = 0.00502 min^{-1} ; (d) all data with Θ_s = 53.0 °C, m = 0.00250 min^{-1}

Artificial Recharge Study, Northern Qatar – a Case History

Abdul Rahman Mahmoud, Dr. Samir Kotoub
and Ahmed Abdul Malek

ARTIFICIAL RECHARGE STUDY, NORTHERN QATAR
A CASE HISTORY

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The groundwater system in Qatar is heavily exploited by the agricultural sector using pumped groundwater for irrigation purposes. Since 1967 the number of farm wells has increased significantly resulting in a fourfold increase of agricultural demand to over 160 Mm3 per annum. The present situation gives a calculated water deficit from 1972 to 1993 of 30% of the freshwater reserves as estimated by the United Nations FAO in the early 1980's. The result of the over abstraction is a deterioration of water quality due saline intrusion from the coast and up coning of more saline water from the deeper aquifers. As part of the national plan to safe guard freshwater reserves the Ministry of Municipal Affairs and Agriculture commissioned a study to investigate the feasibility of Artificial Recharge into the Rus and Umm er Rhaduma aquifers, this paper details the methods and equipment employed by Entec Hydrotechnica and DAWR in undertaking the study.

Background

Agriculture in Qatar is based almost entirely upon irrigation using pumped groundwater, since 1967 the number of farm wells has increased from 350 to more than 2,000 [1]. The result of this has been an increase of agricultural demand from 42 Mm^3 to over 160 Mm^3 in 1993, (figure 1) giving a calculated water deficit from 1972 to 1993 of 850 Mm^3 [2] or 30% of the freshwater reserves as estimated by the United Nations in the early 1980's (figure 2) . At present recharge to the underlying aquifer is dependent on intense rainfall events and a limited amount of return from agricultural irrigation. This has been estimated by the Department experts as 12% of the long-term mean average rainfall or an average of 30 Mm^3 per annum, which means that abstraction is currently running at five times in excess of natural recharge from rainfall.

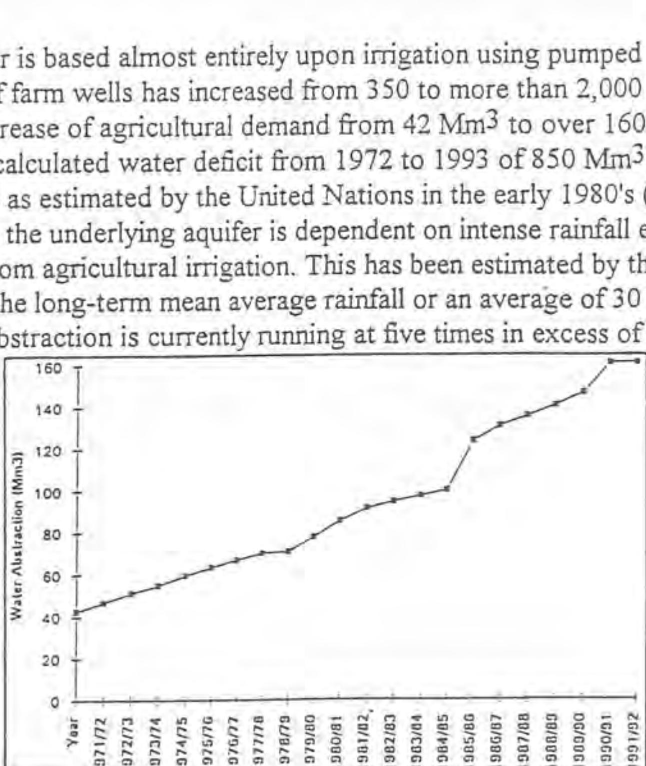

Figure 1 Total Abstraction of Groundwater for Agricultural Development

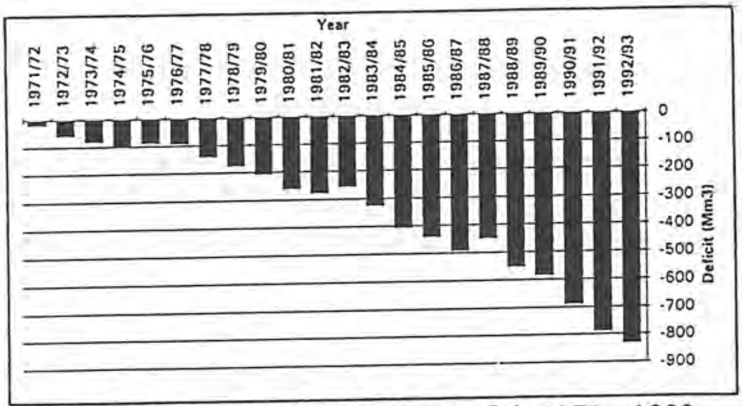

Figure 2 Accumulated Water Deficit 1971 - 1993

The consequences of the present abstraction regime are a drop in water levels of between 0.5m to 1.0m per year [3], and a deterioration of water quality due to saline intrusion from the coast and up coning of deeper saline water in the Umm er Rhaduma in some places. This is now manifesting itself by the abandonment of farms in coastal areas and the rapid salinisation of soils due to higher chloride concentrations in the pumped groundwater [4].

General Geology and Hydrogeology

There have been several reports on the geology of Qatar, each contributing more to the understanding of the stratigraphic sequence of the peninsular and its' regional setting. The sequence defined for Qatar by Cavalier and Seltrust [5] is as shown in Table 1:

Table 1 General Geological Succession of Qatar

Period/Series	Formation	Member
Neogene Recent Pleistocene	 Superficial Deposits Superficial Deposits	
Paleogene Miocene/Pliocene Miocene	 Hofuf Formation Dam Formation	 Upper Lower
Eocene	Upper Damman Formation Lower Damman Formation Rus Formation	Abarug Member Simsima Member Alveaolina Limestone Midra Shales Fhaihil Velates Limestone
Palaeocene	Umm er Rhaduma Formation	
Upper Cretaceous	Aruma Formation	

For the purposes of the study only the Damman, Rus, and Umm er Rhaduma Formations are relevant, being the strata having an immediate bearing on the freshwater aquifers being considered.

The Umm er Rhaduma (UER) of Qatar has a type lithology of grey-brown, vesicular dolomite / dolomitic Limestone with varying amounts of quartz, up to 40% near the top of the formation. A chert layer is recorded 20m below the top of the unit in the east of Qatar. The UER is overlain by the Rus Formation, a contact that in the type area in Saudi Arabia is abrupt with a possibility of a hiatus in deposition. However, in Qatar it is difficult to identify the exact contact due to facies similarity and post-depositional processes. Due to a period of gentle uplift at the time of deposition the Rus varies in thickness and facies over the geographical area of Qatar, the two facies being : a gypsiferous, argillaceous facies - Depositional Sulphate Facies (DSF) and a calcareous facies - Depositional Carbonate Facies (DCF). Post depositional dissolution of the gypsum within the DSF has given rise to another facies - Residual Sulphate Facies (RSF). During the drilling and logging work for the study, the contact between the UER and the Rus was found to be more gradational than had been indicated in past papers. This led to a two strata 'Transition Zone' nomenclature being derived to give a UER Transition Zone and a Rus Transition Zone, with each of these strata having the essential properties of the main lithological unit, either Rus or UER, while posessing characteristics attributable to the immediately adjacent stratum. The Rus Formation is overlain conformably by the Damman Formation, which consists of a series of shallow water limestones, marls and shales. Where the shales are present the contact between the two formations is clear, elsewhere the change between the sparry, hard Limestone of the Damman and the chalky, soft Limestone of the Rus has been used.

The main productive aquifers in northern Qatar are the DCF and RSF parts of the Rus Formation and the upper parts of the UER Formation. Historically the freshwater lens was developed by abstraction from wellfields with a large number of widely spaced wells designed to skim off the freshwater without causing up coning of saline water. However, most of the groundwater abstraction now takes place from farm wells, usually completed to be open in both the Rus and UER in order to increase yields from the higher permeability zones of the UER.

Permeability in the Rus appears to be controlled by both its depositional and post depositonal history, thus the DCF appears to have a relatively high primary permeability due to its basic lithology whilst the DSF is relatively impermeable. Where the DSF has undergone extensive dissolution to RSF it may be quite permeable, this permeability being controlled by both lithology and the fractures created by the collapse of the roofs of solution features. The permeability of the UER appears to be dominated by the presence of a karstic horizon near the top of the unit which may have formed during sub-aerial exposure prior to the deposition of the Rus, in some places the voids are infilled by clay and silt leading to a reduced permeability.

The hydraulic continuity between the Rus and UER seems to vary from location to location. In the south the RSF has a low permeability and seems to confine the UER whereas in the north there is an apparent high degree of hydraulic continuity between the DCF and the UER. Given the sub-horizontal nature of the many fissure systems that largely control the overall transmissivity , it appears that the degree of hydraulic continuity is dependent on the vertical permeability of the intervening rock which may be distinctly different to the horizontal permeability.

The Damman Formation is largely unsaturated over the centre of the peninsula. However, around the coast it is folded down to below sea level and is therefore of hydrogeological significance on the regional scale. The Lower Damman contains the Midra Shale member which is relatively impermeable, although in the centre of Qatar this has been eroded prior to the deposition of the Upper Damman Limestones creating a window exposing the Rus to fresh recharge water. This has enhanced the dissolution processes and hence increased the permeability in the Rus in this area. [6]

Study Area Selection

The general study area was initially identified by DAWR as being within the 1971 3m above sea level water level contour and the 1000 mg/l dissolved solids concentration contour (Figure 3). Further refining of four specific test site locations was then carried out with consideration being given firstly to the geology and hydrogeology to define the general area for each site so that the locations were split as follows:

Site 1 - situated in the RSF with the UER chert layer present.
Site 2 - situated in the RSF with the chert layer absent.
Site 3 - situated on the DCF / RSF boundary with the chert layer present.
Site 4 - situated in the DCF with the chert layer absent. (Figure 4). [7]

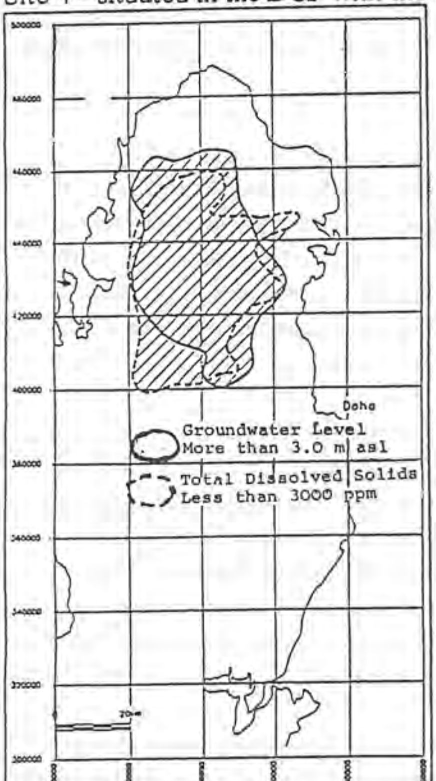

Figure 3 Study Area Location

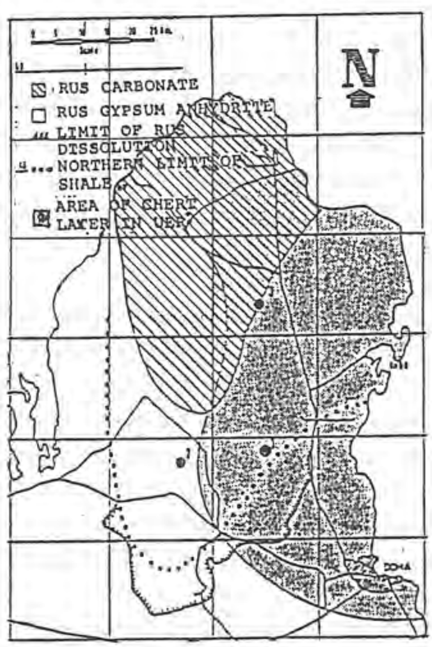

Figure 4 Test Site Locations

Once the areas had been defined, each site was then considered in more detail in order to optimise the test location. Logistical consideration was given to the availability of water for each site to allow the injection tests to be carried out, and the requirement for access to the sites for drill rigs and test equipment to get to the area without excessive ground preparation works. As sensitive monitoring equipment was to be used in the testing, further consideration was given to the possibility of interference effects from local abstractions, particularly farm wells, so using data on farm abstractions collected by the Irrigation Section of DAWR radii of influence were drawn and the site was then located at a point where less than 50mm drawdown would be predicted.

Site and Test Design

The aim of the field testing was to provide the basic parameters required for modelling a large scale recharge scheme. Among the most important parameters to be defined were Transmissivity, Storativity, Effective Porosity, Aquifer Thickness and Dispersivity. Transmissivity and Storativity were determined by normal pumping test analysis, Aquifer thickness was determined from the geological and geophysical logging carried out on site, Effective Porosity and Dispersivity were examined by the monitoring of levels of tracer in monitoring wells and re-abstracted water. Although Effective porosity is difficult to measure directly, in unconfined aquifers, like the Rus in this instance, it approximates to the Specific Yield, which can be determined from pumping test analysis. In confined aquifers it is usually considered to be a standard ratio of total porosity, which in this study was measured from the sonic and normal resistivity geophysical logs. The modelling carried out on completion of the field work was then used to finalise the accurate figures.

Prior to the start of site work a limited amount of preliminary computer modelling was carried out using SWIFT, a solute transport model. A number of runs were carried out using aquifer parameters thought to be representative of the UER following a historical data collection exercise. These were used to show the position and degree of dispersion of the injected water under the proposed testing programme. The results were then used to decide on an observation well spacing of 30m and 80m from the pump well. With the Rus having a much lower Transmissivity this spacing was reduced to 10m and 30m for the observation wells. After further discussions between Entec Hydrotechnica, DAWR and the UNDP the two observation well lines were moved closer to each other to enable a measure of hydraulic connection to be made by taking reading readings in both sets of wells during the pump testing. (Figure 5).

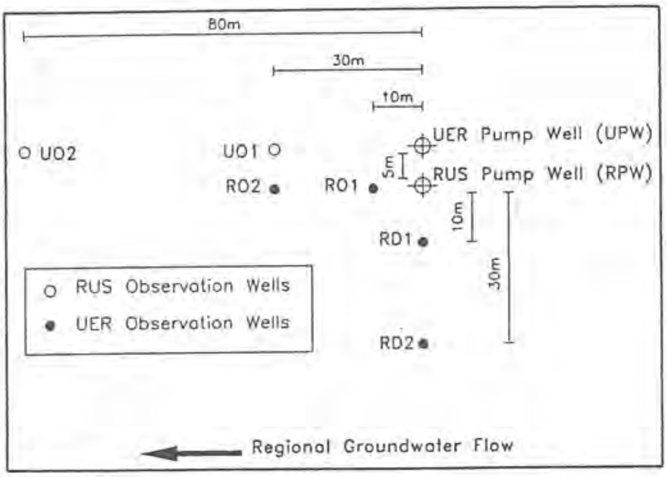

Figure 5 Test Site Borehole Configuration

A standard test methodology was adopted for testing each aquifer at all four sites. A preliminary step test was carried out to assess the pumping well efficiency. This was followed by a seven day constant rate test in order to determine the standard aquifer parameters, Transmissivity, Storativity and leakage. Using the results from these tests a suitable injection rate was selected and a fourteen day injection test carried out, the injection water being labelled with a fluorescent dye. Consideration had to be given to a variety of potential problems with the injection phase of testing, in order to prevent air entrainment into the well the injection pipe had to be installed at a minimum of 10m below standing water level, a further precaution being the design and fitting of an orifice plate to the end of the pipe to maintain a positive pressure at the wellhead. With the standing water levels being between 17m and 40m below ground level at the four sites and the aquifer properties encountered, there was no requirement to introduce the injected water under pressure into any of the test wells.

Immediately after the end of injection a re-abstraction test was carried out for 28 days in order to determine the recovery of the injected water. A final step test was then carried out to see if any changes in borehole efficiency had occurred over the full testing period. [8]

Monitoring of the water levels in the pumping and observation wells was undertaken continuously prior to, and during, the testing using transducers and dataloggers, these being set to take readings at intervals suitable to the test being undertaken. The water levels were monitored in both aquifers during testing in order to determine the degree of hydraulic connection between the aquifers. The quality of the injected and abstracted water was monitored continuously using a water quality data logger with sondes to monitor temperature, electrical conductivity (EC), pH, dissolved oxygen and turbidity.

During the injection and re-abstraction tests it was important to quantify the breakthrough of injected water at the observation boreholes and the proportion of the injected water in the total water re-abstracted. A natural or artificial tracer was therefore required to label the injected water. At a few of the test sites the natural contrast in EC between the injected water and the native groundwater was sufficient to allow this parameter to be used as a tracer. However, at most sites the EC contrast was small and Rhodamine WT, a fluorescent dye, was added to label the injected water.

Rhodamine WT was selected as a tracer after consideration of a variety of factors; its safe use in waters of potable or agricultural supply quality, its degree of interaction with the aquifer and native groundwater, background and detection levels, ease of measurement and cost. At concentrations of less than 100ug/l Rhodamine WT is approved for use in potable water supplies by the US EPA and the UK NRA. In general, Rhodamine WT does not react with the aquifer or groundwater, although some sorption may occur in aquifers containing large amounts of organic or clay materials. These conditions were not expected in the carbonate Rus and UER aquifers. In the Rus and UER groundwaters, small amounts of natural organic material give a background fluorescence equivalent to less than 0.2ug/l Rhodamine WT. Using a Turner Designs 10-AU-005 Fluorometer, Rhodamine WT detection levels are better than 0.1ug/l and sample concentrations may be measured with ease using either a continuous flow through cell linked to a datalogger, or a discrete sample facility. The low background and detection levels permitted a dosing concentration of 50ug/l Rhodamine WT in the injected water and proportions of down to 2% injected water to be measured confidently, this in turn meant that only small amounts of tracer concentrate had to be used thereby keeping the costs low.

Site Works

In order to be able to test each of the aquifers separately it was important to carry out careful well construction. The boreholes were drilled using a Hans England Rotary drill rig utilising a combination of down the hole hammer and tri-cone rock bit using air flush, foam flush was permitted in areas of unstable ground during the drilling. The precise depths of drilling were determined by the site conditions as the borehole progressed. At each site the first hole to be drilled was an UER observation well to allow an assessment of the geology and hydrogeology to be made through logging of the drill flush samples taken every metre and field EC's taken every 3m once the first water strike was encountered. Using the information gathered on site, the boundary between the Rus and UER aquifers was assessed and the casing depth for the UER boreholes decided on. Durawell thickwalled UPVC casing was then inserted, grouted into place and allowed to set prior to the well being continued to full depth. In general the construction was similar at each site : the Rus wells were cased to 3.5m from the surface to prevent surface material dropping into the hole, then drilled to approximately 5m above the base of the Rus aquifer to prevent direct connection with the UER. The UER wells were cased and grouted through the entire Rus aquifer and then drilled on until a sufficient thickness of UER was penetrated, generally a minimum of 20m. All wells were developed by air lift techniques until the water flows became clean.

Further information from the wells was collected by running a full geophysical survey down each borehole on site. The following sondes were run :

Caliper - Measuring borehole diameter, locating fissures and other secondary porosity features, checking on casing depths.
Gamma - Measures natural radioactivity of the aquifer, used for locating shale and clay layers,which in turn is useful for correlation between boreholes.
E-Log (16" and 64" normal resistivity) - Measures the combined resistivity of the aquifer and groundwater. This is useful for determining groundwater quality and zones of high secondary porosity, the 64" sonde measures deeper into the aquifer than the 16" sonde.
Temperature / Conductivity - Measures water quality and geothermal profile. This may be used for identifying a saline interface, and can be used for determining the principal fissure inflows/discharges from the well.
Sonic Log - Determines the velocity of sonic waves in a section of the aquifer which can then be used to calculate porosity.
Flow Log - Determines the vertical velocity of water in a borehole, this is especially useful when the borehole is being pumped as the contributions from various horizons to the total flow can be determined.

All the information collected was then used in the analysis and development of the computer models for each site and the regional water management predictive model to enable the DAWR to predict the effect of different injection / re-abstraction scenarios.

Pump and Injection Testing

Schematic diagrams of the general site set up for pumping and injection tests are shown in figures 6 and 7. Three electric submersible pumps were used during the testing programme : Grunfos SP5A-8 for testing the low flows in the Rus, Caprari 100mm pump for testing the medium flows in the Rus, and a BBC 8T60 for the high flow UER testing. A 150mm Sykes diesel powered pump was then used to pump water from the test sites to nearby depressions, 800m to 1200m away, in order to prevent recirculation and interference effects.

For the Rus pumping and injection tests 75mm GI pipe was used for all connecting pipework and the rising main / injection pipe, flow being controlled by 75mm gate and ball valves. For the UER pumping and injection tests 150mm GI pipe was used for all connecting pipework and rising main, however, 75 mm GI pipe was used for all the vertical section of the injection pipe in order to increase back pressure to assist in preventing air entrainment, flow was controlled by a 150mm gate valve. 150mm 1.5 bar layflat delivery hose was used for the discharge pipe to the depressions.

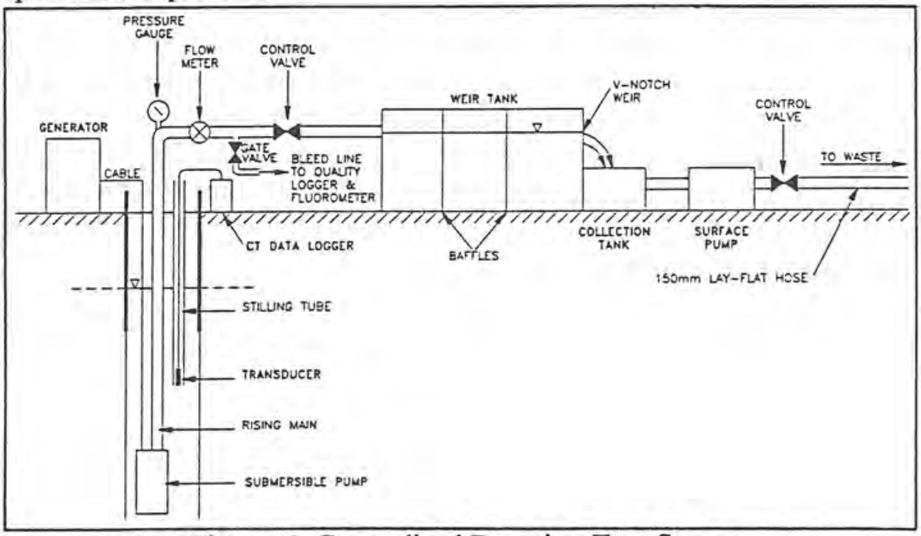

Figure 6 Generalised Pumping Test Set-up

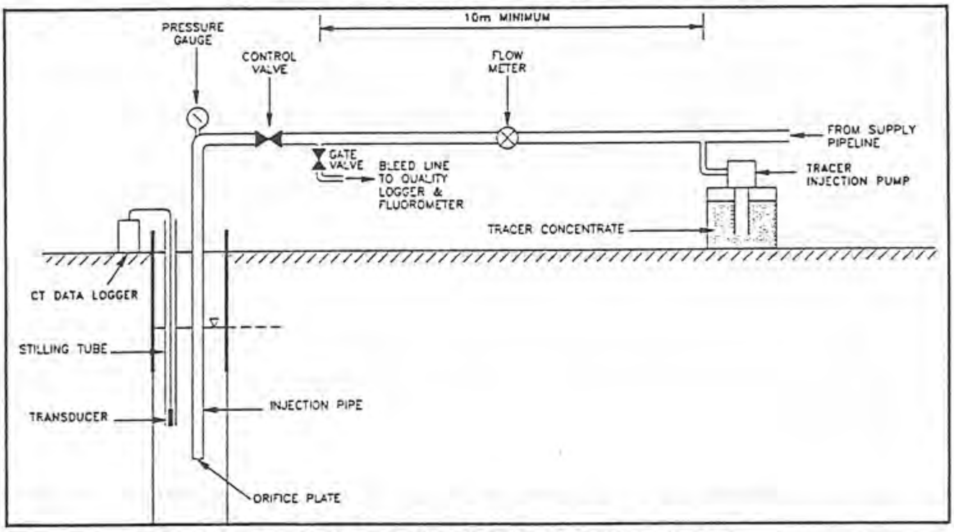

Figure 7 Generalised Injection Test Set-up

During the injection the Rhodamine WT fluorescent dye was added as a tracer having been prepared as a concentrate and added to a 200 litre drum, from this it was injected into the supply pipeline about 10m upstream of the wellhead. Originally a Wallace and Tiernan G0V 2MD chlorine injection pump was used, then as an aid to finer control this was replaced by a LM1 P135 water dosing pump.

Results

The aquifer test data were analysed in the field using manual methods for distance drawdown analysis further refined by the use of a computer curve fitting programme to obtain the basic aquifer parameters. These results are summarised in Tables 2 and 3:

Table 2
Comparison of Injection / Re-abstraction Results

Aquifer	Site	Q/S		T (m2/d)	
		Injection	Re-abstraction	Injection	Re-abstraction
	1	9.17	9.04	10	10
Rus	2	13.08	15.01	22	14
	3	11.19	8.74	50	50
UER	1	2200	1414	1500	1400
	2	2000	1248	1000	680
	3	731	603		800
	4	472	451		560

Table 3
Comparison of Step Test Data

Aquifer	Site	B		C	
		Pre-test	Post-test	Pre-test	Post-test
	1		0.0618		7.29E-05
Rus	2		0.0160		1.60E-04
	3	0.0570	0.0650	1.00E-07	4.27E-04
UER	1	0.000316	0.000097	2.77E-07	2.62E-07
	2	0.000638	0.000638	1.25E-07	1.25E-07
	3	0.000941	0.000911	9.11E-07	6.31E-07
	4	0.000400	0.000820	1.16E-06	8.05E-07

As can be seen by the spread of results and the differences between values calculated from injection and re-abstraction, both aquifers showed complex responses to pumping. This was caused by interaction of hydraulic effects caused by flow in a dual porosity, fractured medium and vertical leakage between the aquifers. As a result standard methods of analysis could only be applied as guidelines. To overcome this finite difference modelling was used to simulate the field data.

Taking the site 3 data as an example [9], the transmissivity of the UER test section, as indicated by the specific capacity, was more than an order of magnitude greater than that of the Rus. (Figure 8). The specific capacity of both aquifers during injection were similar to, or greater than, those measured during abstraction and did not vary significantly as injection progressed. The specific capacities measured during the final step test were higher than those measured before testing began. This indicates that, rather than any aquifer clogging occurring during the testing, a problem that has occurred in other recharge attempts, the aquifer was developed by the injection / abstraction process.

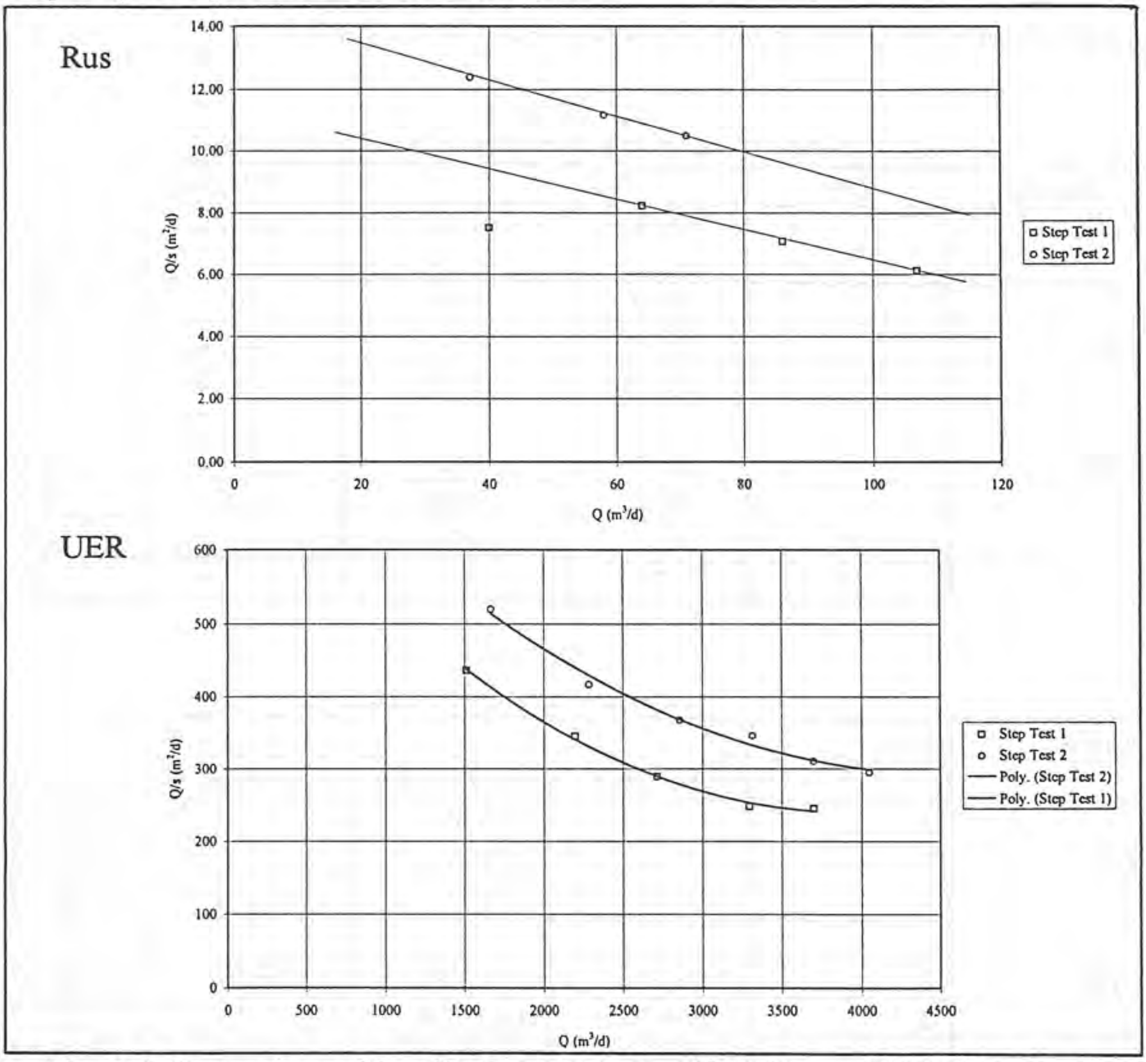

Figure 8 Comparison of Specific Capacity Data - Site 3 Rus and UER Step Tests

An important concept in the viability of an artificial recharge scheme is that of recovery efficiency. Merritt [10] suggested that recovery efficiency is defined as the volume of water recovered before the quality deteriorates below a required threshold. This is a useful measure on a site specific basis but can create difficulties when comparing the performance of aquifers with different background water qualities. An alternative approach is to define the overall recovery efficiency as the percentage of injected water recovered after the same volume has been abstracted as was injected. The recovery of injected water from the Rus and UER at site 3 are shown in Figure 9. These indicate that the Rus has a much lower dispersivity than the UER, and as a consequence, has a much higher overall recovery efficiency (70%) compared with the UER (40%). The effect of the dispersivity is being assessed quantitatively in more detail using the SWIFT finite difference solute transport model.

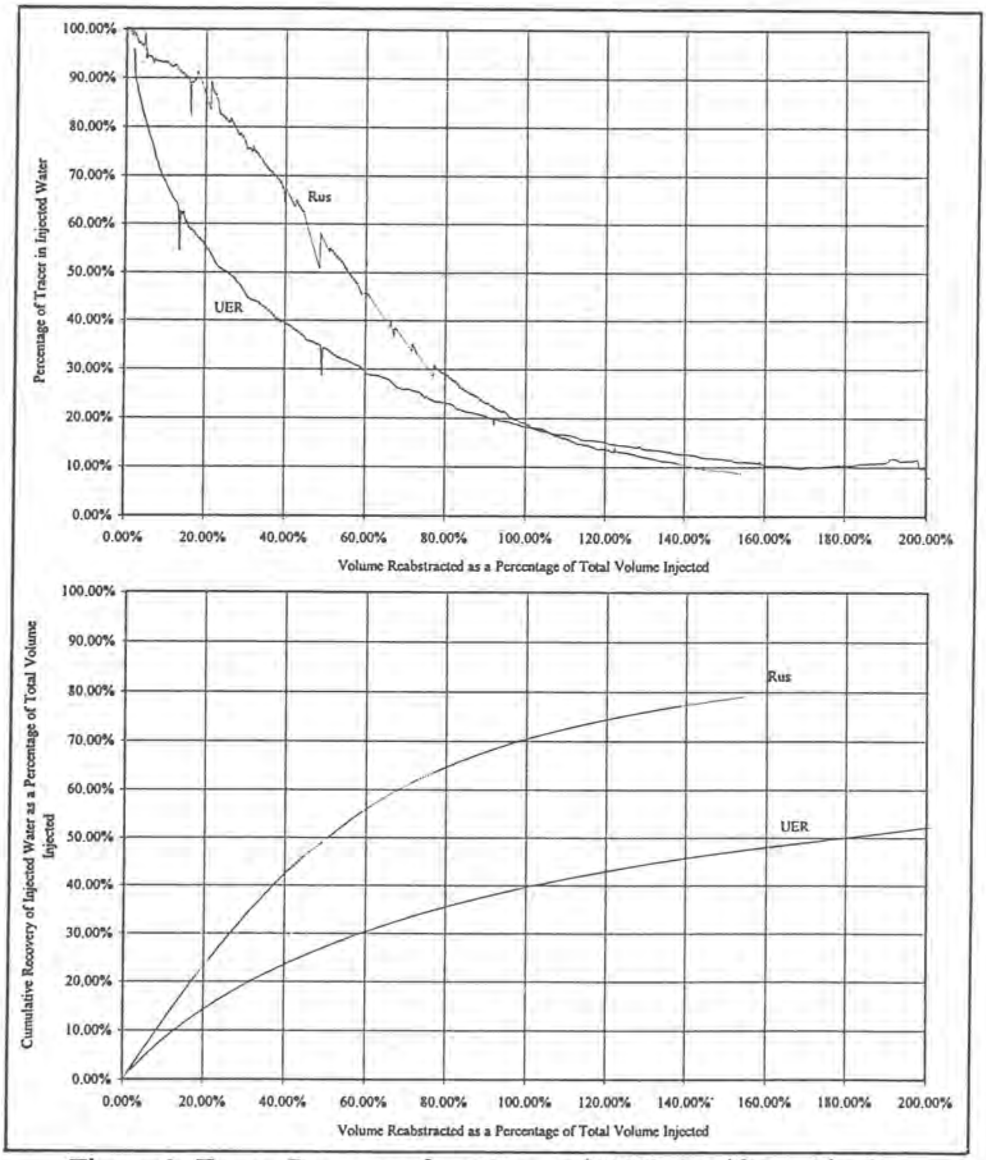

Figure 9 Tracer Recovery from Rus and UER Aquifers - Site 3

While the site tests have proved that the aquifers in Qatar are capable of accepting water for recharge, further work continued to provide the Qatari Government with useful management information for both immediate use and for the future. The results of the modelling of the site data are being applied to larger models in order to simulate a range of aquifer management scenarios. Different options can therefore be assessed objectively in order to determine the most efficient use of artificial recharge to improve groundwater quality. In Qatar there is an important management decision to be made as to the most suitable aquifer for artificial recharge. While the Rus has higher recovery efficiencies its' overall capacity is much lower than the UER due to lower transmissivity. In this case computer models are being used to assess the number and configuration of wells required to inject a given quantity of water into each aquifer and the resultant recovery efficiencies under a range of scenarios. The lower number of wells required for injection into the UER needs to be balanced against a lower recovery efficiency in a financial assessment of the relative merits of each aquifer for artificial recharge.

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Dams in the United Arab Emirates and Their Role In the Groundwater Recharge

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DAMS IN THE UNITED ARAB EMIRATES AND THEIR ROLE IN THE GROUNDWATER RECHARGE

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Like *other regions of the Arabian Gulf United Arab Emirates is also suffering from the shortages of fresh water supplies. Rainfall is erratic and meager and in the absence of perennial surface water resources all the agricultural supplies are met through groundwater. The annual recharge of groundwater from the rains is estimated to be 120 million cubic meters. To enhance the groundwater recharge, dams have been built at a number of wadies in the UAE.There are about 35 small and large dams having a total designed storage capacity of about 70 million cubic meters. The results of the monitoring wells indicate that the dams are functioning effectively and contributing to the groundwater recharge.*

Wadi Ham dam which was built in the year 1982 is used as study case to demonstrate the role of dams in the groundwater recharge.

Since 1982 there have been six main rainy seasons with eighteen flood events have been recorded at the dam site. Apart from providing protection against flood damages the dam has retained and recharged to aquifers about 18 million cubic meters of water.

GENERAL :-

The United Arab Emirates lies in the eastern corner of the Arabian Peninsula.It is situated between latitudes 22 degrees and 26 degrees North and between longitudes 51 and 56 degrees East. The UAE is bordered in the north by the Arabian Gulf, in the east by Oman and the Gulf of Oman and in the west and south by Saudi Arabia. Its mainland surface area is 77,700 square km.

CLIMATE:-

The UAE has a hot, dry climate typical to the hot arid desert zone. The air temperature is high through out the year. The summer months are very hot and winter is pleasantly warm.The average yearly air temperature varies between 25 and 30 degrees Celsius. In summer the maximum day temperature may rise to about 50 degrees Celsius. The average rainfall in the UAE IS 105 mm/year.The average rainfall varies and shows an increase from the coast (Sharjah 90 mm/year) towards the mountain range(140 mm/yr.)The western desert portion of the UAE receives the lowest rainfall, about 40 mm/yr.

WATER RESOURCES.

There are no perennial surface water resources in the UAE and the country depends on groundwater for all its agricultural water supplies.The municipal water supplies in the major cities are met through desalinated water while in rural areas the domestic supplies are met through groundwater. Treated sewage water is used in big cities for landscaping and gardens.The total recharge by precipitation is estimated to be 120 million cubic meters/year.

In order to enhance the groudwater recharge a number of dams have been built for controlling the flood water and recharging it into the aquifers.There are about 35 large and small dams in the UAE the total storage capacity of these structures is about 70 million cubic meters.Figure (1) shows the location of some major dams.

The dams do not only save precious fresh water but also provide protection against losses caused by flash floods.

The results of monitoring boreholes downstream of dams have proved that dams are functioning effectively and recharge is taking place ,see table(1) which gives the maximum rise of water levels recorded in the observation wells at some of the dams .Table (2) shows the flood volumes stored and the average rainfall for the period 1982-1993. The total volume of the water stored was about 50 million cubic meters in ten years.The number of flood events during this period is shown in the table (3).

Wadi Ham dam is used as study case in this paper to demonstrate the role of dams in the groundwater recharge.

STUDY CASE WADI HAM DAM

Introduction :-

- The wadi Ham dam is located at wadi Ham approximately 7 km upstream of Fujairah city about 8 km from the coast.

The dam was constructed in order to:-

a. Recharge the groundwater into the underlying aquifers
b. Hindering the possible discharge of fresh water to the sea,as a consequence in case the infiltration capacity of the gravel plains in the lower wadi area is exceeded during the high runoff resulting from major floods.
c. Provide protection to the downstream area from the flash floods.

The construction of the dam was completed in 1982 It comprises of four main structures :--

- Gravel filled Main dam with a core of fine material, combined with North dam totaling about 2050 mt.long and upto 16 meters high
- A South dam about 750 mt. long and upto 11 mt.in height.
- Concrete spillway no.1 which is 60 mt.long and 3 mt high.
- Spillway no. 2 .which is 118 mt.long and 3 mt.high.

Present capacity of the dams is about 6 million cubic meters.
Figure (2) shows the project layout and the fig.(3) shows the typical cross sections of the dam and spillways.
Table (4) gives wadi Ham project statistics.

GEOLOGY. -:

The dam commands a catchment area of 190 square kilometers mainly comprising of the wadi Ham.
The entire catchment is underlain by rocks of the Semail complex. This complex is an Ophiolite suite of basic igneous rocks but also includes extensive exposures of metasediments.
The Semail complex was placed as a thrust sheet from the east.The rock material found in the hills of the area consists of medium grained gabbro alternating with fine- to medium grained diorite.
The surface of the hills is generally covered by tallus material and weathered rock.

The surface weathering is quite obvious due to closely spaced bedding with many joints and minor faults.The climate is liable for a generally deep reaching weathering. The rock outcrops in the dam axis belongs to a subsurface ridge which was well proven by the geoghysical soundings. The rock surface at the ridge is only about 20 mt. deep while further downstream it was found to be at about 70 mt. depth.

The alluvial deposits may be subdivided according to their age as follows,

- Recent :- Sandy gravel, slightly silty, loose,sound ,rounded components of diameter maximum 30-50 mm. with some pebbles and cobbles.These channel gravel occurs along the braided active channels
- Young :- Sandy gravel,slightly, with many cobbles and boulders . Some rounded to subrounded and angular components.
- Old gravels :- Sandy gravels, mostly silty, with many cobbles and boulders. Weathered, rounded to sub angular components.Typically cemented layers.
- Conglomerate Unit :- This unit lies directly over the Semail bedrock.It is highly cemented and hard and compact .

The hills typically are covered by tallus material consisting of angular rock components with silt and sand .The components are of medium sized gravels (dia.15-30 mm.). Within the group of hills at the right abutment there is an area of fine grained surface matrial well protected from strong current of occasional floods.

GROUNDWATER OCCURENCE.

The main aquifer comprises of sand and gravel of the Recent,Young and Old Gravel units within the wadi and the alluvial fan downstream of the wadi The bedrock-gabro-has also been found to contain some groundwater locally by the virtue of secondary porosity due to fractures.

The depth to water level decreases from 30--50 mt near the dam site to about 5-7 mt.near the coast.

The infiltration of flood waters along the active channels and the direct infiltration of the rainfall provides the major source of recharge to the aquifers down stream of the Wadi Ham dam.

The groundwater resources are exploited mainly by,

- Irrigation wells near the coast and upstream.
- The Ministry of Electricity and Water's well fields at Fujairah and Sha'ara , for domestic water supplies.

The total production of the two wellfields operated by the Ministry of Electricty and Water is approximately 3.5 million cubic meter annually.The annual groundwater abstraction along the wadi Ham upstream of the irrgated coastal zone is estimated to be about one million cubic meter annually.

WATER QUALITY.

The water quality near the coast has been detioriated considerably due to excessive pumping and sea intrusion . Salinity in the wells near the coast has reached upto 6000 ppm.and even more.The situation in the upstream areas and at the Sha'ara wellfield is still good and the salinity is about 800 ppm.Salinity values in this area in 1982 were about 700 ppm,the little increase in the salinity could be attributed to the recharge from the dam.

GROUNDWATER LEVELS :

A network of obsevation wells for the monitoring of groundwater levels downstream of the dam is maintained by the Ministry since 1987. The observation wells, BHF.15,BHF.1, BHF.4,BHF.9 and BHF.12 lie downstream of the dam in the wadi (fig. 4.), and are least affected by the pumping effect of the irrigation wells. The wells,BHF.15and BHF.12 are equipped with automatic recorders.Daily fluctuations are recorded on the charts which are changed monthly.The readings at other wells are normally taken at monthly interval.The hydrographs of these wells are presented in the figure(5).

The effects of major flood events are apparent on the hydrographs of the wells.

In the hydrograph of the well No. BHF.15 which is the nearest to the dam, about 900 mt.downstream , water level started to rise almost immediately after the floods. While in the hydrographs of the other wells there is an elapsed time for the same flood event for water level to rise.The variation in the values of the rise of water levels in individual wells also depends on the lithology of the well .

The pattern of water level changes after the flood include the following features:-

- Downstream decrease in water level change.
- Downstream increase in time to reach the peak.

From the results of the observation wells it is evident that the dam is functioning well and serving the purpose for which it has been built for.

See table no.(2) which gives the flood volumes in the dams and table (3) which gives the number of flood events at the dam sites.Figure(6) shows the volumes of the flood water stored in the reservoir at wadi Ham dam for the period 1982-1993.

The rise in the water level in the wells downstream of the wadi Ham dam can be corelated with the flood volumes in the reservoir and the consequent infiltration of the water from the storage area.

In 1988 the total flood volume during the two flood events at the dam site was about 7.5 million cubic meters (MCM) and the maximum rise of the water level in the well No.15 , downstream of the dam was 23.65 mt.

Table (1) shows the maximum rise in the water levels in the monitoring wells downstream of the Ham dam and some of the other dams in the UAE.
From the fluctuaions of the water levels in the wells it is evident that the recharge is happening and this depends on the amount of the rainfall and the flood volume of the water stored at the dam .

RECHARGE TO GROUNDWATER

To evaluate the role of wadi Ham dam in recharge of the groundwater and its effect on the aquifer , the flood of 1987-1988 is considered here.
The flood occured on18-19 Feb.1988 from the storm over the wadi Ham catchment area.The storm is the largest recorded since the constrution of the dam.The dam was filled upto the maximum flood level mark ,84.5 mt above the sea level and the spillways over flew for the first time. The dam stored about 6.0 million cubic meter water resulting from the flood within twelve hours of the start of the rains.
The infiltration into the aquifer has been calculated by using the'*change in volume*' method.
The groundwater hydrograph records show a maximum rise of 23.6 mt.over a period of 40 days,at well No.BHF 15,about 900 mt. down stream of the dam.At well No. BHF.4,about 6.8 km. from the dam the maximum rise recorded 1.4 mt. over a preriod of some 200 days.
Calculation of the change in the water levels(between the beginning February 88,and at the end of March'88) show a change of volume of 52,000 sqaure meters along a line of section between BHF 15 and BHF 9 (see Fig.7). Assuming an aquifer width of1,000 meters the total change in the volume of the aquifer section was 53 million cubic meters.
The change in aquifer volume upstream of well No BHF 15 and below the reservoir can be estimated by assuming a flood reservoir area of 400,000 sq.mt. and a change of 30 meters in the water level.This volume is estimated to be about 12 million cubic meters.
The conversion of aquifer volume to water volume is made by using a value of 0.1 for the storage coefficient. The total change in the volume of water is therefore, worked out to be about 6.5 million cubic meters.
As mentioned earlier,some of the early water level rise must be atributed to the direct rainfall infiltration.The monitoring wells downstream of the dam in the wadi plain show an initial rise of about 1.5 mt.of water level over an area of 2 km. by 1 km. between wells BHF 15 and BHF 9 ,this infiltration totalled to 0.3 MCM.

The total infiltration from the reservoir is therefore estimated to be about 6.0 million cubic meters.This figure is comparable to the 6.0 MCM water stored at the dam and an estimated 0.5 MCM overflow from the spillways.

Within the limits of accuracy of the estimations it is concluded that there was almost complete infiltration of the stored water and the recharge effect has been noted in the monitoring wells.

CONCLUSION.

Since its construction in 1982 the wadi Ham dam is functioning effectively and contributing to the groundwater recharge.The total volume of the flood water stored at the dam has been estimated to be 18 million cubic meters Such quantity of the water has positively affected the groundwater in the aquifers as an average annual infiltration of about 1.64 million cubic meters.

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3. Sir M.Mac Donald and Partners,
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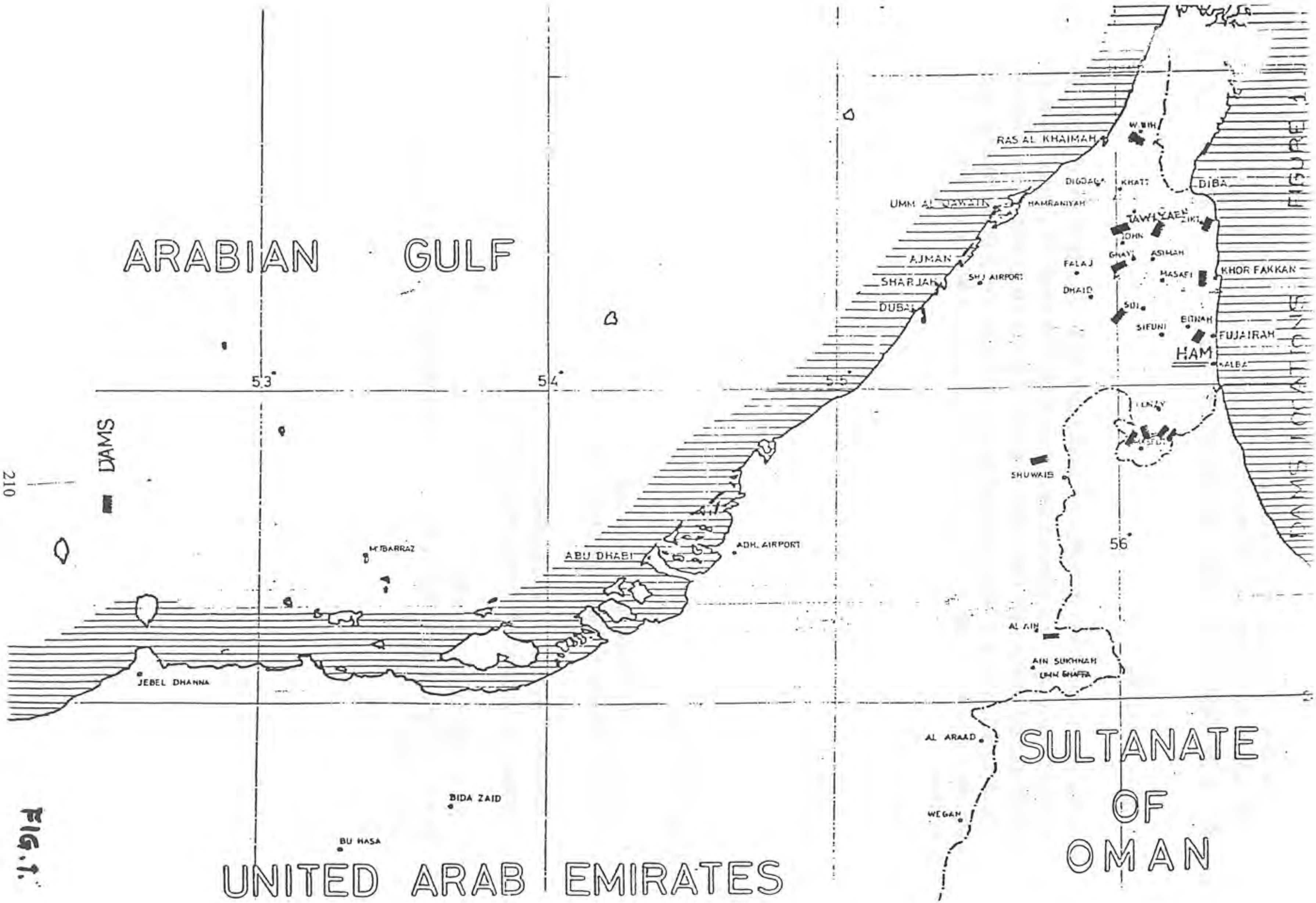

ARABIAN GULF
UNITED ARAB EMIRATES
SULTANATE OF OMAN
DAMS
53
54
55
56
RAS AL KHAIMAH
UMM AL QAWAIN
AJMAN
SHARJAH
DUBAI
ABU DHABI
DIBA
KHOR FAKKAN
FUJAIRAH
HAM
KALBA
DIGDAGA
KHATT
HAMRANIYAH
TAWI ZAEB
ZIKT
JOHN
GHAYL
ASIMAH
FALAJ
MASAFI
DHAID
SUI
SIFUNI
BITNAH
SHJ AIRPORT
SHUWAIB
AL AIN
AIN SUKHNAH
UMM GHAFFA
AL ARAAD
WEGAN
ADH. AIRPORT
M'BARRAZ
JEBEL DHANNA
BIDA ZAID
BU HASA

FIG. 1.

Wadi Ham: General Layout

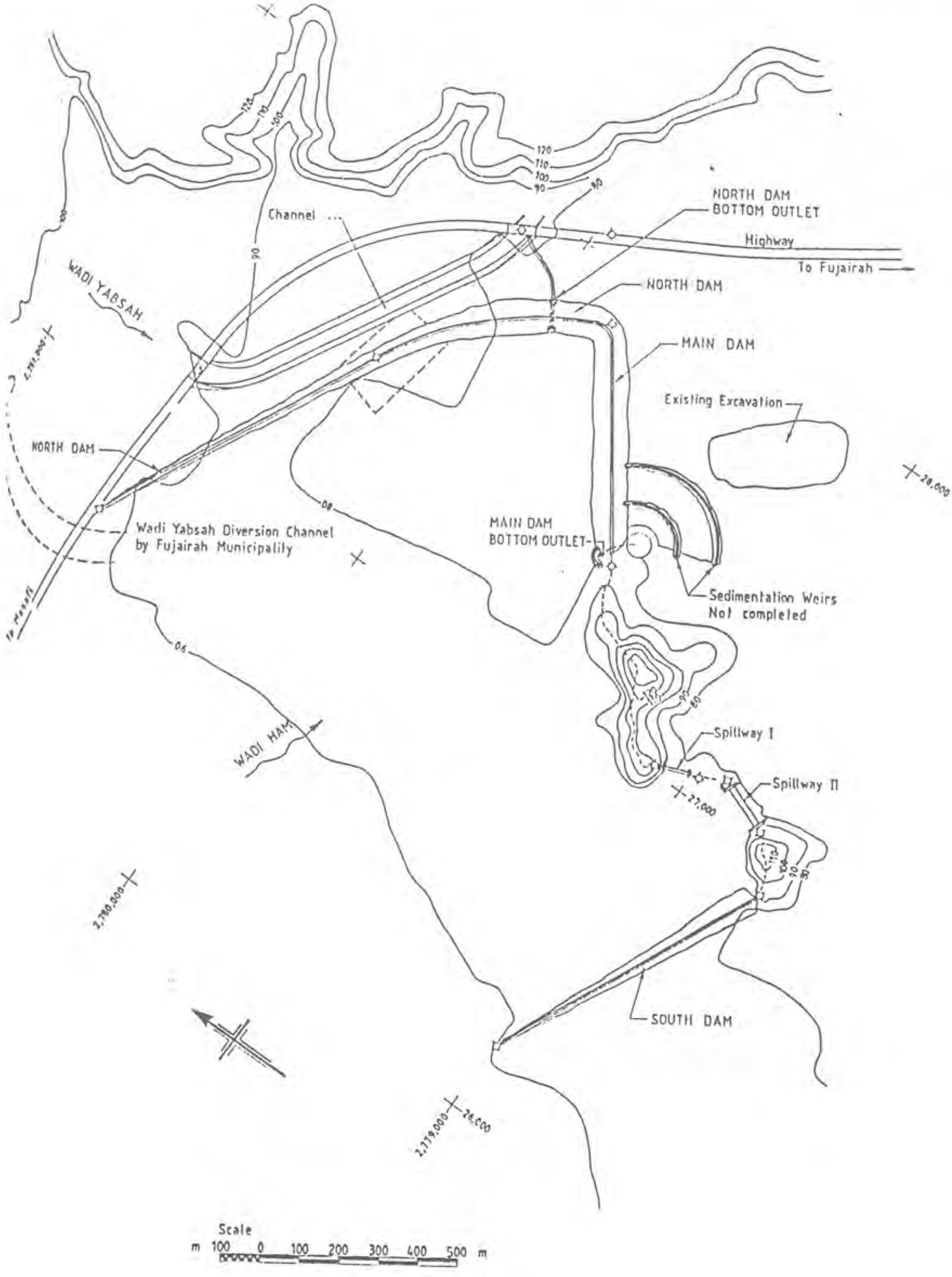
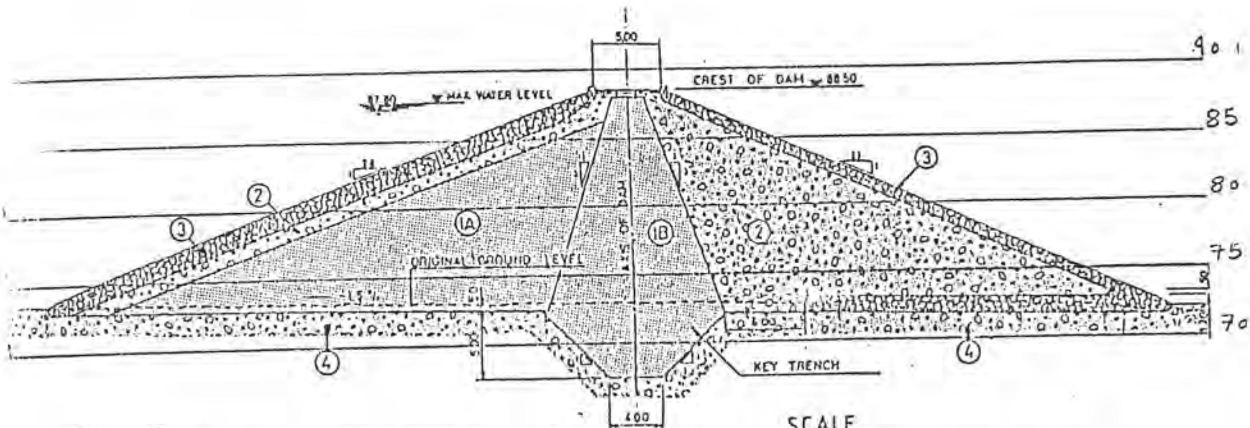


Figure 2

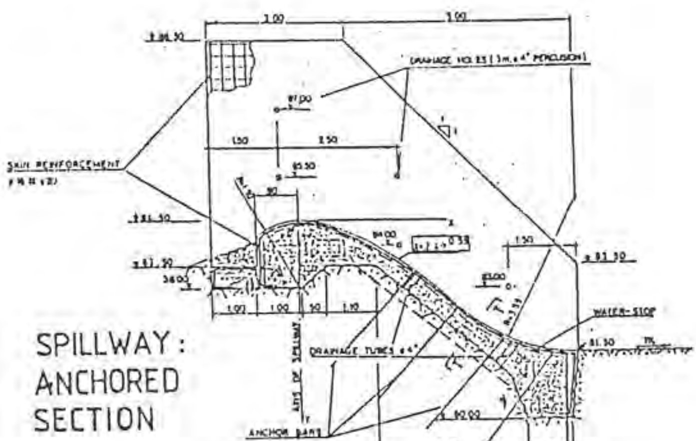
Wadi Ham: Typical Sections



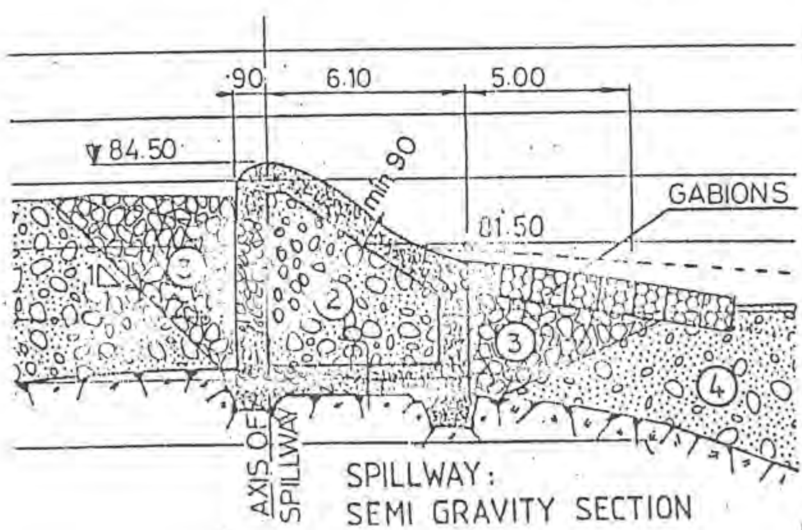
SCALE
0 5 10 15 20 m

(1A) CORE { PROCESSED ALLUVIUM (FINE) Φ < 50mm
(1B) { 70% 1A + 30% COLLUVIUM
(2) SHELL – NATURAL ALLUVIUM COMPACTED
(3) RIP-RAP DRAINAGE – PROCESSED ALLUVIUM (COARSE) Φ > 50mm
(4) FOUNDATION – NATURAL ALLUVIUM

MAIN DAM



SPILLWAY: ANCHORED SECTION



SPILLWAY: SEMI GRAVITY SECTION

Figure 3

Wadi Ham : Location of Monitoring Boreholes

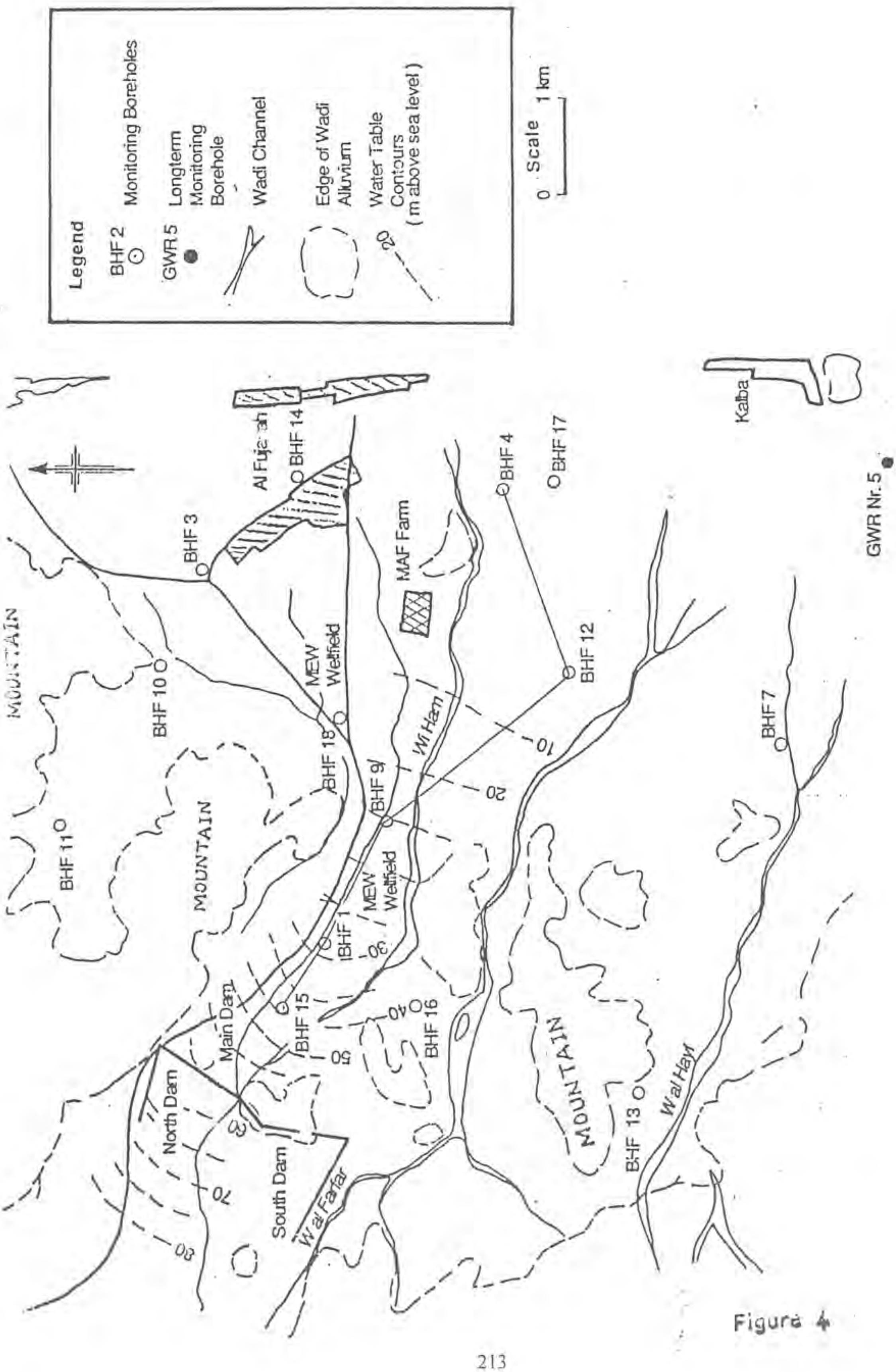
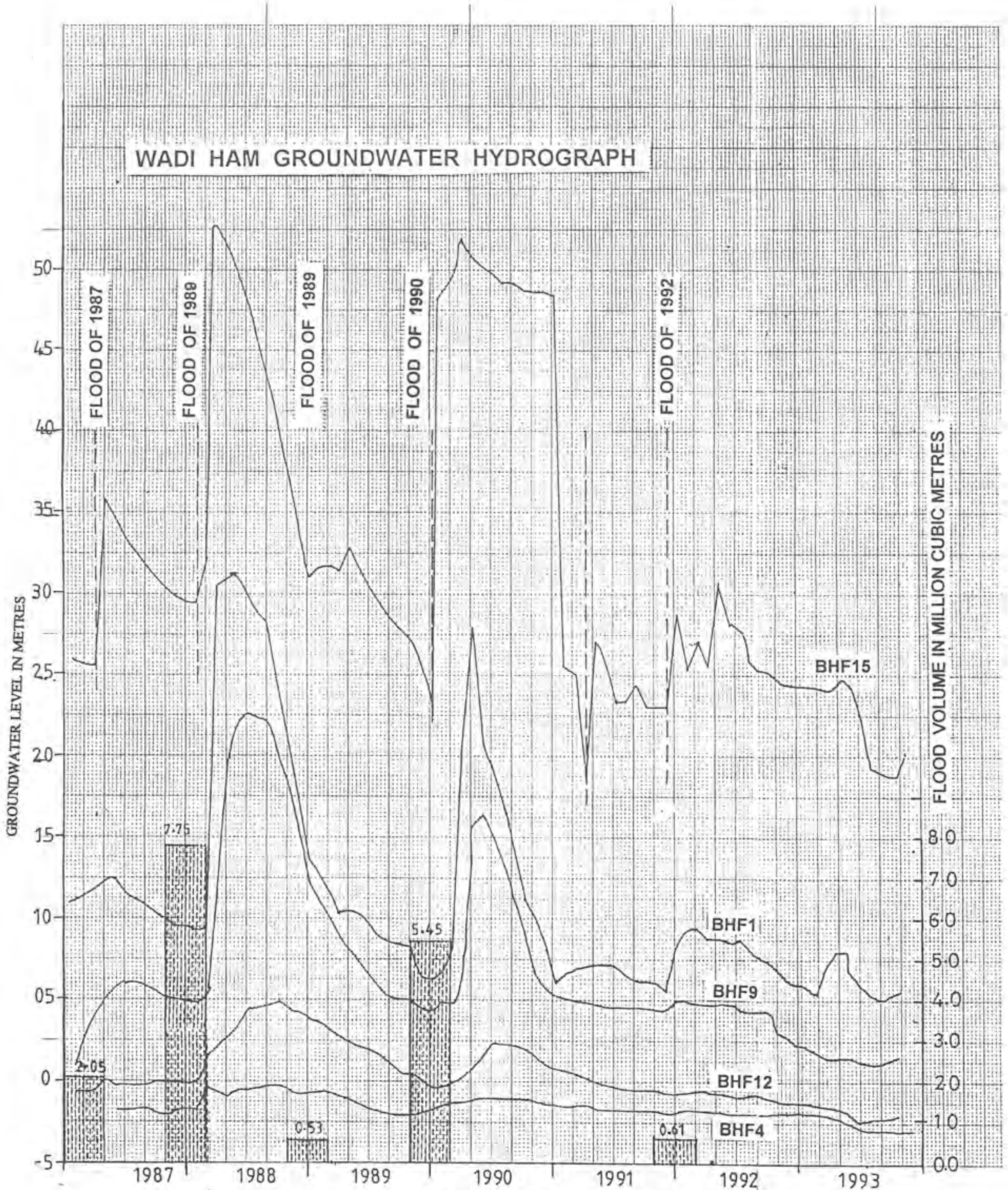


Figure 4



WADI HAM GROUNDWATER HYDROGRAPH
FLOOD OF 1987
FLOOD OF 1988
FLOOD OF 1989
FLOOD OF 1990
FLOOD OF 1992
GROUNDWATER LEVEL IN METRES
FLOOD VOLUME IN MILLION CUBIC METRES
BHF15
BHF1
BHF9
BHF12
BHF4
2.05
7.75
0.53
5.45
0.61
50
45
40
35
30
25
20
15
10
05
0
-5
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1987
1988
1989
1990
1991
1992
1993

FIG.5 & FIG.6

Wadi Ham : Change in Aquifer Section

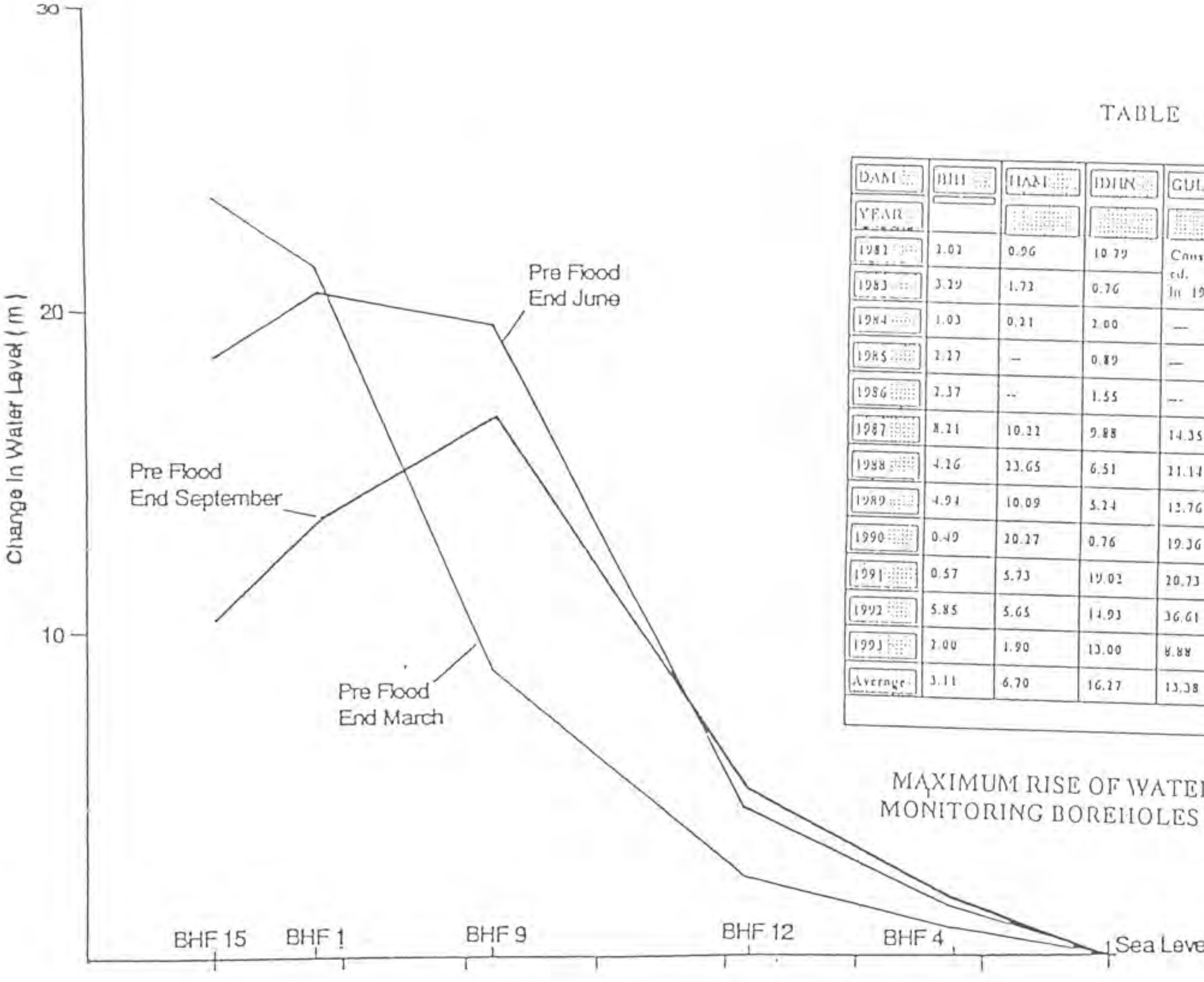


TABLE 1

DAM	BIH	HAM	DHN	GULFA	HADF	ZIKT	TAWIAYN
YEAR							
1982	2.02	0.96	10.79	Constructed In 1984			
1983	3.29	1.72	0.76				
1984	1.03	0.21	2.00	—			
1985	2.27	—	0.89	—			
1986	2.37	--	1.55	---			
1987	8.21	10.22	9.88	14.35			
1988	4.26	23.65	6.51	21.14			
1989	4.94	10.09	5.24	12.76			
1990	0.49	20.27	0.76	19.36	CONSTRUCTED IN		
1991	0.57	5.73	19.02	20.73	1991	1991	1991
1992	5.85	5.65	14.93	36.61	8.47	18.52	0.42
1993	2.00	1.90	13.00	8.88	10.84	1.5	9.64
Average	3.11	6.70	16.27	13.38	9.65	10.01	5.03

MAXIMUM RISE OF WATER LEVELS IN METRES IN MONITORING BOREHOLES DOWNSTREAM OF DAMS

FIG. 7

TABLE 2

YEAR / DAMS	82-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93
BIH	0.926	--	0.08	--	3.00	9.48	0.008	1.20	--	4.28	2.20
HAM	1.55	0.08 --	--	--	2.05	7.75	0.53	4.45	--	0.61	--
IDHN	0.103	--	--	--	0.15	0.85	0.20	0.50	--	0.57	0.70
GULFA	--	--	--	--	0.281	0.50	0.03	0.15	0.07	0.19	0.31
HADF	--	--	--	--	--	--	--	--	--	0.54	1.07
ZIKT	--	--	--	--	--	--	--	--	--	0.50	0.43
TAWIAYN	--	--	--	--	--	--	--	--	--	2.32	1.32
TOTAL	2.58	0.08	0.08	--	5.481	18.58	0.76	7.30	0.071	9.02	6.03
AVERAGE RAINFALL in mm	260	28	23	60	160	227	76	178	70	145	145

FLOOD VOLUMES IN THE DAMS IN MILLION CUBIC METRES AND AVERAGE RAINFALL FOR THE PERIOD 1982 - 1993

TABLE 3

YEAR	82-83	83-84	84-85	85-86	86-87	87-88	88-89	89-90	90-91	91-92	92-93
DAMS											
BIH	2	--	--	--	3	5	1	2	--	4	1
HAM	4	--	--	--	4	2	2	3	--	3	--
IDHN	1	--	--	--	3	1	1	1	--	3	2
GULFA	--	--	--	--	4	1	1	4	3	4	1
HADF	--	--	--	--	--	--	--	--	--	3	1
ZIKT	--	--	--	--	--	--	--	--	--	2	2
TAWI-AYN	--	--	--	--	--	--	--	--	--	3	2

NO. OF FLOOD EVENTS IN THE DAMS

TABLE – 4

MAIN DAM and NORTH DAM:

Type	gravel
Length	2050 m
Crest level	88.5 m
Original ground level(lowest)	73.0 m
Height above original ground level	15.5 m
Height above lowest foundation level	20.5 m
Upstream slope	1:2.8
Downstream slope	1:2.3
Volume (including South Dam)	1.25 $m^3 10^6$
Crest width	5.0 m

SOUTH DAM:

Type	gravel
Length	750 m
Crest level	88.5 m
Original ground level(lowest)	76.0 m
Height above original ground level	12.5 m
Height above lowest foundation level	14.5 m
Upstream slope	1:2.8
Downstream slope	1:2.3
Volume	see above
Crest width	5.0 m

SPILLWAY NUMBER I:

Type	concrete ogee (partially reinforced)
Crest length	62.00 m
Crest level	84.50 m
Design flow (EW value, see text)	890 m^3/s

SPILLWAY NUMBER II:

Type	concrete ogee (partially reinforced)
Crest length	118.00 m
Crest level	84.50 m
Design flow (EW value, see text)	1 700 m^3/s

PROJECT PURPOSE:

- flood attenuation
- flood retention
- recharge of downstream aquifer

Telemetry Based Water Supply Management System at Water Transmission Directorate of State of Bahrain

Subhash S. Natu

TELEMETRY BASED WATER SUPPLY MANAGEMENT SYSTEM AT WATER TRANSMISSION DIRECTORATE, STATE OF BAHRAIN.

SUBHASH S. NATU
Electronics Engineer.

PREAMBLE :

Water Transmission Directorate is responsible for producing potable water for consumers in the State of Bahrain. The process involves extraction of ground water, blending of ground water and desalinated water , chlorination and its quality control. This potable water after blending and quality control is stored in the elevated tanks. The net work involves distillate forwarding stations, ground water stations and blending stations which are located all over Bahrain .

For optimized monitoring of the complete water network it is necessary to have centralized process control system . The system that is employed is a combination of electronic instrumentation for the measurement of process parameters, telemetry system for transmission of process data and centralized computer control system for actual control function as also for management information .

This paper is intended to give an insight in to the technical aspects of water supply management system.

1.0 BLENDING PROCESS :

Fig . 1 shows the process and instrumentation (P & I) diagram of a typical blending station .

The station receives desalinated water from distillate forwarding station . (Umm Al Hassam for urban area and West Rifa'a for rural area) . The blending station also extracts its ground water with the help of bore hole or submersible pumps . Desalinated water and ground water are stored in separate ground storage tanks .

Based on various factors of water quality , ground water and desalinated water are mixed with predetermined ratio and blended water is produced . This blended water is stored in separate blended water tanks .

Blended water is then subjected to chlorination and chemical/bacteriological quality control as it enters in to the elevated tanks .

Aforementioned process at each station is monitored automatically with the help of telemetry based control system located at centralized place called West Rifa'a and at Umm Al Hassam .

2.0 ELECTRONIC INSTRUMENTATION :

The base level of any control system is the generation of electronic signals which are directly related to process parameters . Present technology provides variety of state - of - the - art instruments to measure different parameters and convert them in to proportional electronic signals . The important measurements in a typical blending station are as under :

- Flow of incoming and out going water mains .

- Level of water in storage tanks .

- Pressure in distribution mains .

- Temperature of blended water .

- Conductivity of ground water , blended water and distillate .

- Residual chlorine levels in blended water .

- Conductivity of ground water , distillate and blended water .

- pH of ground water , distillate and blended water .

- Speed of pumps .

These parameters are aquistioned in a standard form of 4 to 2o milli ampers and are termed as analog signals .

Over and above there are digital signals which are available in the form of open / close contacts Here is a list of digital inputs for the operation of blending station :

- Pump running / failed contacts .

- Pump protection contacts . (e.g. . earth fault , overload , phase imbalance etc. .)

- Plant safety signals . (e.g. . High level in the tanks , chlorine leak , fire alarm etc. .)

- Valve open / close contacts .

3.0 TELEMETRY TECHNIQUE :

3.1 TELEMETRY OUT-STATION :

The plant data available in different form is conditioned and digitized with the means of analog to digital conversion . This data is then ready for transmission to the centralized computer system at remote location . This job is carried out in a unit called telemetry outstations .

These unit are microprocessor based and have fixed inputs / outputs capabilities. One telemetry out station handles up to 32 input / outputs . They can also be configured to act as master station . The telemetry outstation software is contained in pre - programmed read only memory (PROM)

3.2 DATA TRANSMISSION :

Data highway technique based on co - axial cable , twisted pair telemetry links or microwave radio links are used for transmission of data . The selection of these techniques is based on transmission distance and speed with which data is transmitted .

In Bahrain water supply system , the data transmission is done with a combination of telemetry links and microwave network .

To enable messages from a number of sources to be transmitted over a common link some form of multiplexing is required , Time Division Multiplexing (TDM) is a well known technique used here . In this method different variables are transmitted sequentially using a single frequency band .

The data is represented by frequency rather than amplitude and this is less susceptible to corruption by attenuation , phase distortion ands noise . The modulation and demodulation is carried out at either end of the link by a device called Modem . The communication protocol used for message transmission is based on industrial standard and is limited to 64 bites or less to minimise error effects .

The messages can be structured to provide functions such as plant data input , plant data output with or with out revertive checking and system status transfers .

Fig . 2 Gives different techniques of transmission of data through telemetry links .

4.0 CENTRALIZED COMPUTER SYSTEM :

The schematic diagram of a central system is shown in fig . 3 . This scheme provides the rural and urban area distribution network with the centralized control from main control center at West Rifa'a or from control center at Umm Al Hassam .

There are three process computers utilized for this purpose viz.

- Two computers at West Rifa'a which are in " Hot Standby " mode i.e. failure in one computer will enable automatic changeover to other computer .

- One computer at Umm Al Hassam is in " Warm Standby Mode " i.e. This computer has same capabilities but has time lag as far as update of database of control center . This provides back-up and geographical separation of the control centers .

FOLLOWING FACILITIES ARE AVAILABLE IN BOTH CONTROL CENTERS :

- Intelligent operator consoles for plant operation .

- Hard copy color printer for serene dumps and management report .

- Mosaic MIMIC panel with dedicated instruments as back - up .

- Unintruptable power supply in case of power failure .

- Dedicated voice communication network .

5.0 SOFTWARE CAPABILITIES :

The control philosophy as outlined is a accomplished with various display on the operator consoles in the following manner :

- Sequential control of each blending station based on predetermined logic .

- Display of each station with real time update of important measured variables . (plant over view)

- Pump status on real time basis .

- Display of measured variables (flow , level , temperature , pH , conductivity , residual chlorine etc. .) in the bar graph form .

-Alarm display of plant parameters .

- Display of chlorinator operation .

- Trend display of various parameters in the group form .

- Display and totalization of flow computations .

- Generation of daily, weekly and monthly reports in terms of water quality and totalised flow.

- Historical data storage .

6.0 SYSTEM ENHANCEMENTS .

6.1 Due to increase in demand of water , number of new blending stations have been commissioned in the recent past and same will be connected to the telemetry network .

6.2 Modifications will be made in the present system so that it will be able to generate .
- Plant equipment performance analysis .
- Profiles or models for optimized water quality .
- More management information on day to day basis .

6.3 Entire telemetry outstations will be also able to work on stand - alone mode in case of any communication failure .

6.4 High integrity communication methods will be used including fibre-optic cables for telemetry links.

6.5 Distributed data-base facilities will be provided.

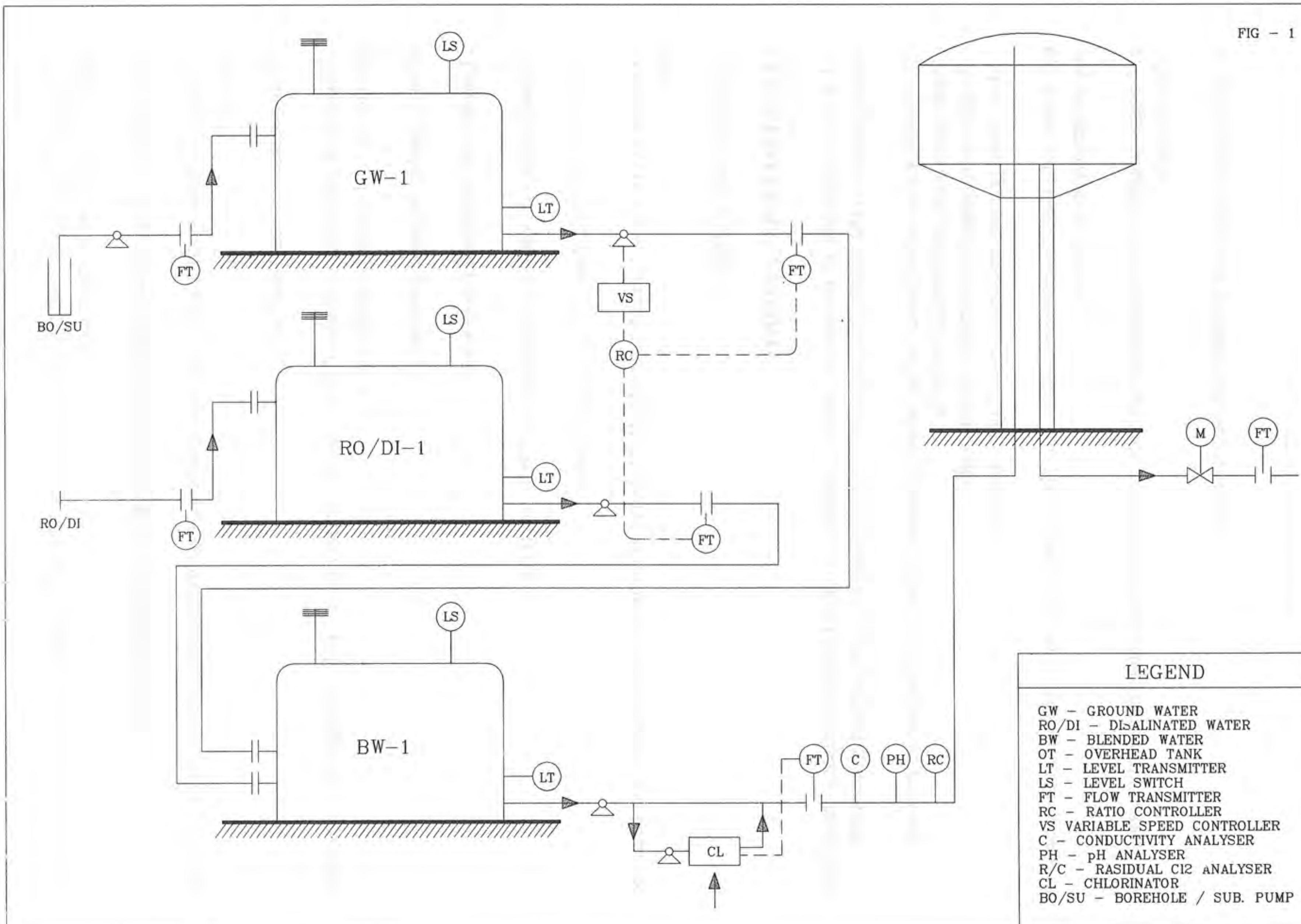

GW-1
RO/DI-1
BW-1
BO/SU
RO/DI
LS
LT
FT
VS
RC
C
PH
CL
M
LEGEND
GW – GROUND WATER
RO/DI – DISALINATED WATER
BW – BLENDED WATER
OT – OVERHEAD TANK
LT – LEVEL TRANSMITTER
LS – LEVEL SWITCH
FT – FLOW TRANSMITTER
RC – RATIO CONTROLLER
VS VARIABLE SPEED CONTROLLER
C – CONDUCTIVITY ANALYSER
PH – pH ANALYSER
R/C – RASIDUAL Cl2 ANALYSER
CL – CHLORINATOR
BO/SU – BOREHOLE / SUB. PUMP

FIG – 1

Fig – 2

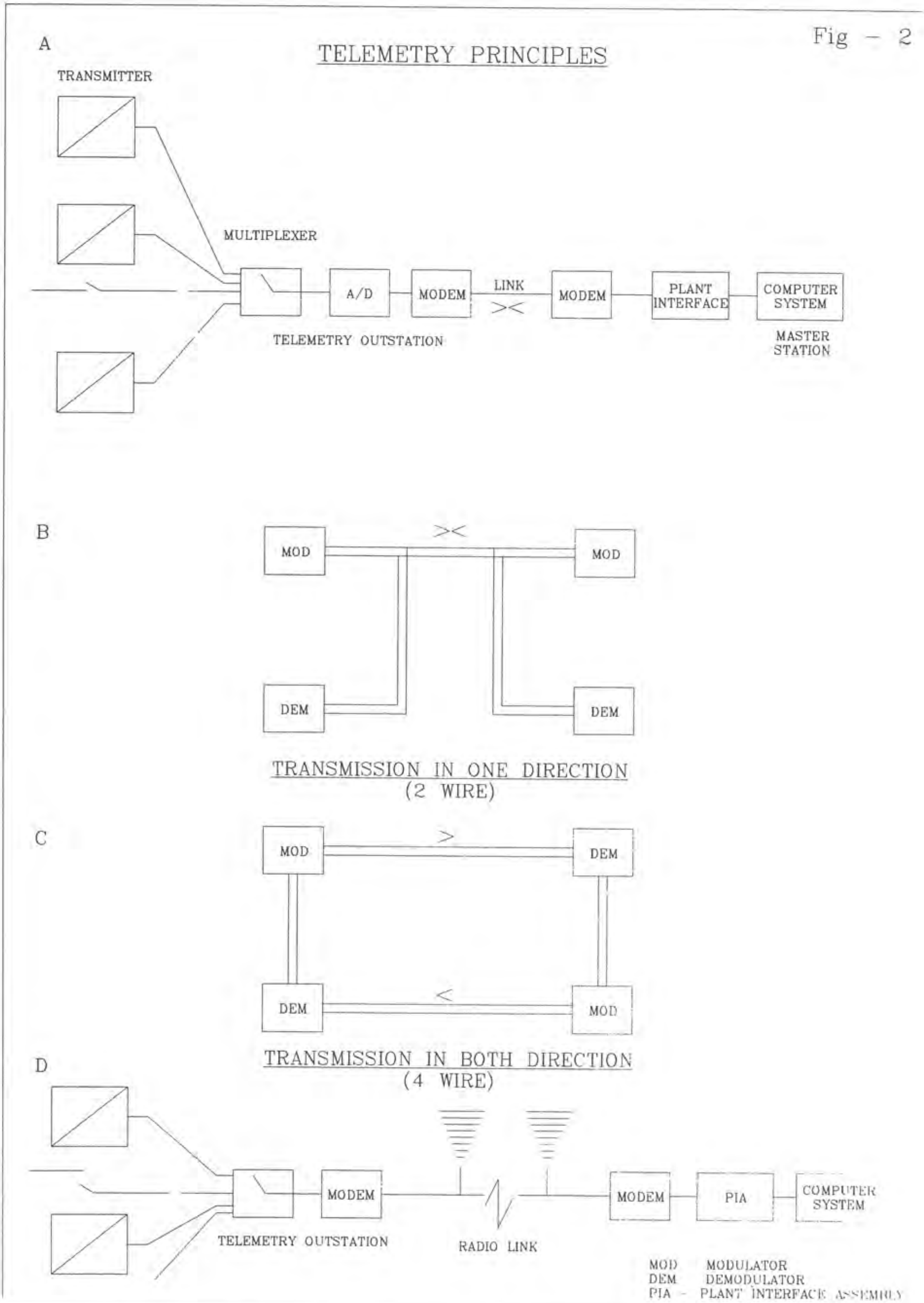
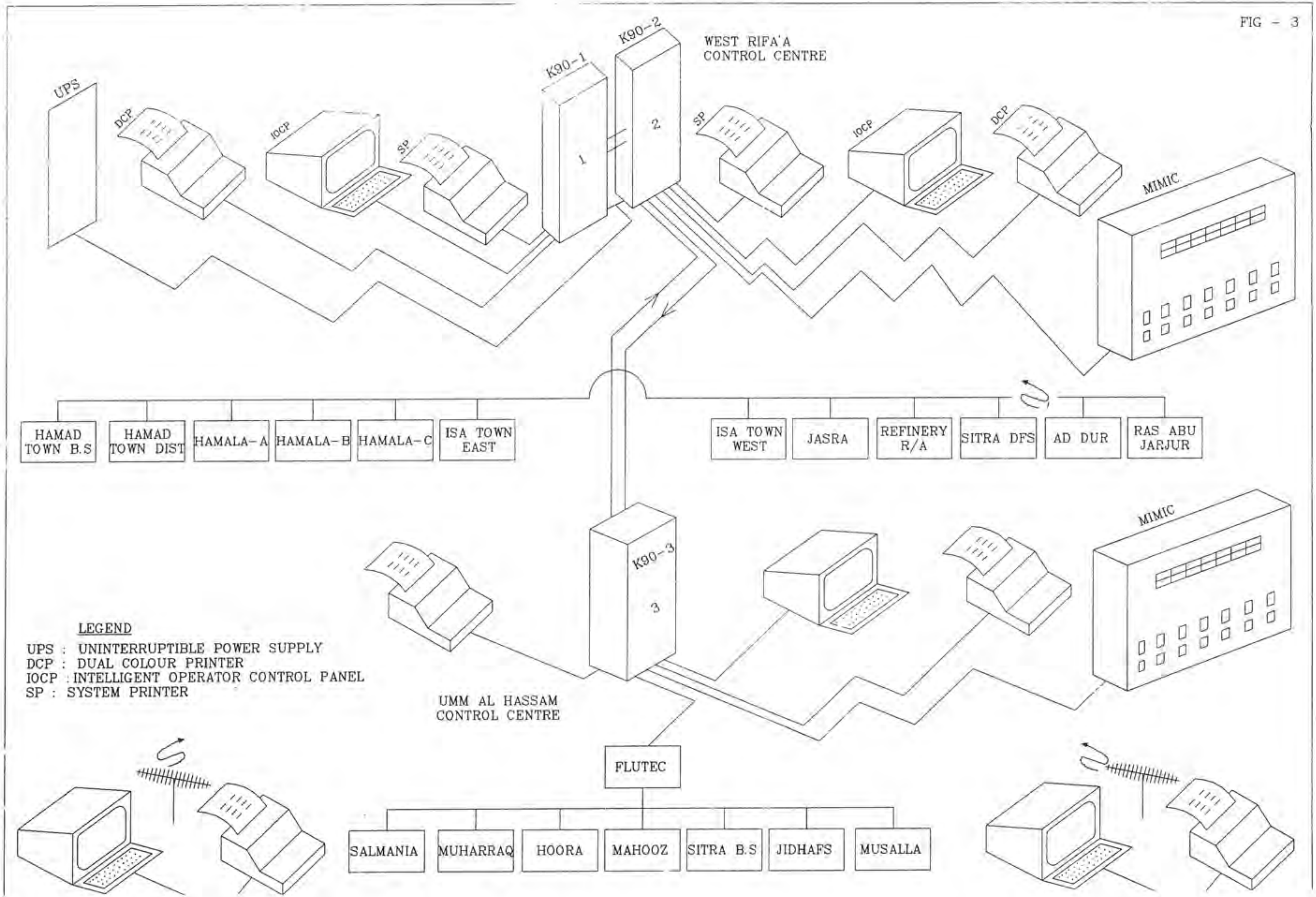

A
TELEMETRY PRINCIPLES
TRANSMITTER
MULTIPLEXER
A/D
MODEM
LINK
MODEM
PLANT INTERFACE
COMPUTER SYSTEM
TELEMETRY OUTSTATION
MASTER STATION
B
MOD
MOD
DEM
DEM
TRANSMISSION IN ONE DIRECTION
(2 WIRE)
C
MOD
DEM
DEM
MOD
TRANSMISSION IN BOTH DIRECTION
(4 WIRE)
D
MODEM
TELEMETRY OUTSTATION
RADIO LINK
MODEM
PIA
COMPUTER SYSTEM
MOD MODULATOR
DEM DEMODULATOR
PIA – PLANT INTERFACE ASSEMBLY

FIG - 3


WEST RIFA'A CONTROL CENTRE
UPS
DCP
IOCP
SP
K90-1
K90-2
1
2
SP
IOCP
DCP
MIMIC
HAMAD TOWN B.S
HAMAD TOWN DIST
HAMALA-A
HAMALA-B
HAMALA-C
ISA TOWN EAST
ISA TOWN WEST
JASRA
REFINERY R/A
SITRA DFS
AD DUR
RAS ABU JARJUR
K90-3
3
MIMIC
LEGEND
UPS : UNINTERRUPTIBLE POWER SUPPLY
DCP : DUAL COLOUR PRINTER
IOCP : INTELLIGENT OPERATOR CONTROL PANEL
SP : SYSTEM PRINTER
UMM AL HASSAM CONTROL CENTRE
FLUTEC
SALMANIA
MUHARRAQ
HOORA
MAHOOZ
SITRA B.S
JIDHAFS
MUSALLA

TRANSIENT-ELECTROMAGNETIC TECHNIQUES FOR GROUND-WATER PROSPECTING IN AL JABEEB AREA, ABU DHABI EMIRATE, U.A.E.

by M. Al Za'afarani[1] and A. Al Kamali[1]
National Drilling Company of Abu Dhabi

ABSTRACT

Transient-electromagnetic surveys were used in the study of the ground-water resources of a 300-square kilometer region known as Al Jabeeb area of Abu Dhabi Emirate. The surveys provided resistivity data which were interpreted to develop layered earth models and to construct both geo-electric and hydrogeologic cross sections. These model interpretations were used to determine: (a) site-specific predictions of the hydrogeologic framework as an aid in siting test wells; (b) target depths for subsequent drilling activities; (c) the thickness and lateral extent of Quaternary alluvium, which contains the main fresh water-bearing aquifer in the area; and (d) the presence of subjacent bedrock containing conductive clay-rich zones.

INTRODUCTION

The main objective of the Ground-Water Research Project (GWRP) is to evaluate fresh and slightly saline ground-water resources of Abu Dhabi Emirate. Surface-geophysical techniques, specifically Transient-Electromagnetic (TEM) surveys, were applied in the project to explore subsurface hydrogeologic conditions before drilling. This information was used to advise drilling staff of the locations of sites with high hydrologic potential. TEM surveys also provide required data for aquifer thickness and depth, which are necessary for ground-water modeling.

The GWRP is the first major research project to emphasize use of TEM surveys in Abu Dhabi Emirate. The TEM method was selected over vertical electrical sounding (VES) because of greater ease and speed of field operation. Also, the method avoids ground-electrode contact problems that commonly arise with resistivity surveys in arid regions. Recent studies have demonstrated the effectiveness of TEM methods in hydrologic investigations (Fitterman, 1987; McNeil, 1988).

DESCRIPTION OF THE STUDY AREA

Abu Dhabi (fig. 1) is the largest of the seven emirates that form United Arab Emirates. It is characterized by a desert climate, with an average precipitation of about 100 mm per year. Consequently, natural recharge to the ground-water system is limited.

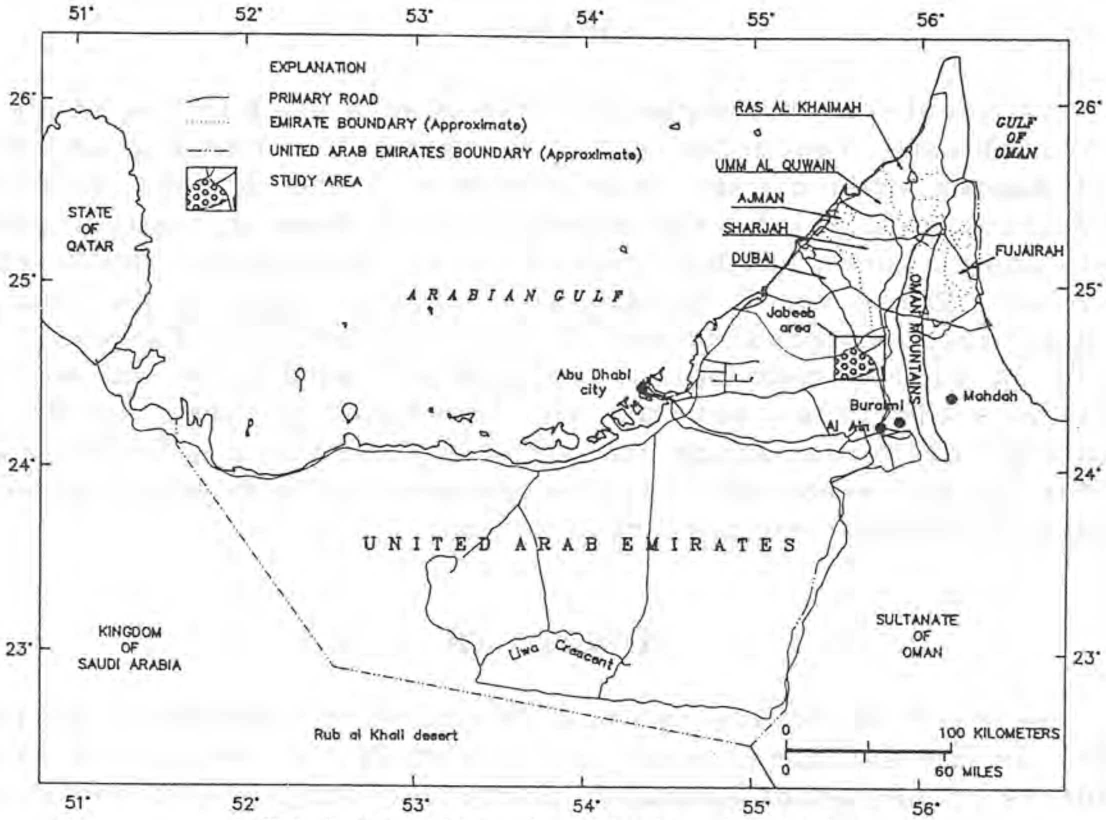


Figure 1. United Arab Emirates and location of Jabeeb study area

Fresh ground water (dissolved-solids concentration less than 1,000 mg/l) historically has been supplied from the shallow alluvial deposits along the western flank of the Oman Mountains, especially near the city of Al Ain. Results of previous drilling indicated that an additional supply of fresh ground water exists in Al Jabeeb area. Al Jabeeb area is represented roughly by a 15 by 20-km rectangle located about 40 km north of Al Ain (fig. 2). The area is situated west of Al Ain-Dubai Highway and is bounded by Sweihan Road on the south and Al Faqa Road on the north. The area encompasses the old Khadar well field and parts of the Jabeeb and Sweihan Road well fields operated by Al Ain Water and Electricity Department. Between November 1990 and February 1994, the GWRP drilled four wells in Al Jabeeb area (GWP-34, 43, 44, and 45). Based on tests of yield and water quality from these wells, it was decided that further drilling would be necessary to evaluate the ground-water system and that TEM should be used as a tool for selecting the drilling sites for wells GWP-107 to 122.

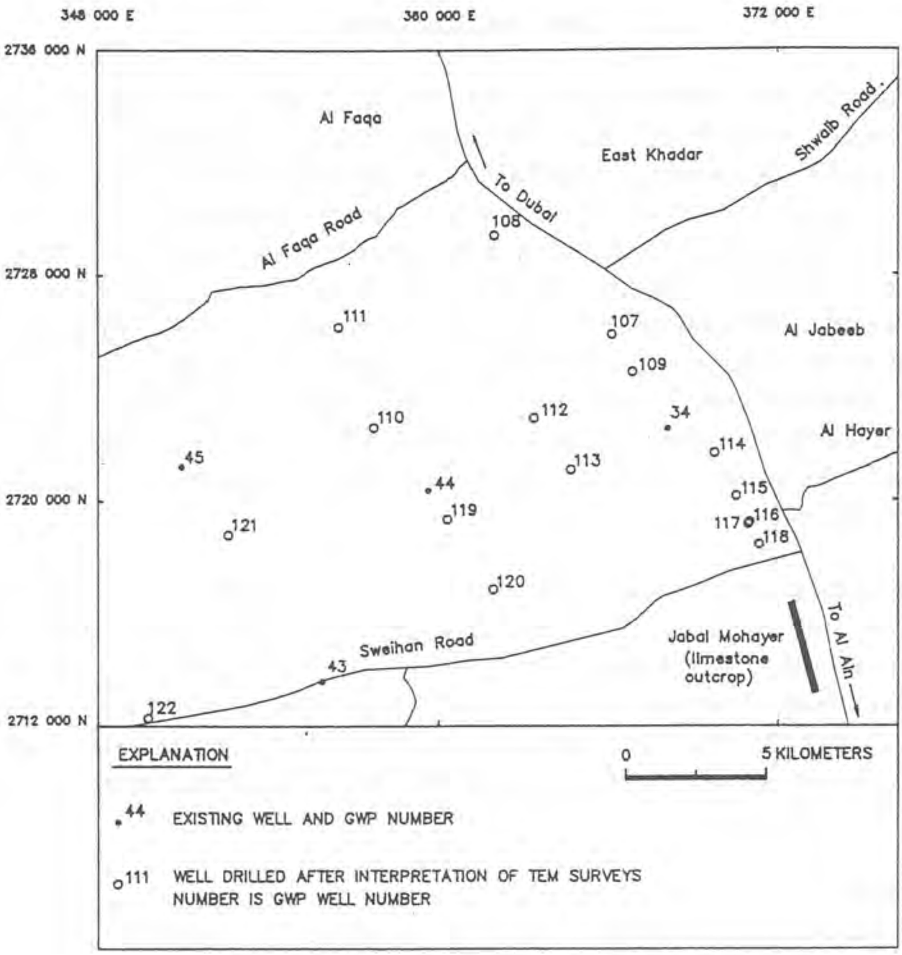


Figure 2. Location of GWP wells in Al Jabeeb area.

The surface is covered by a layer of very fine-to coarse-grained eolian sand of Holocene age. This layer varies in thickness from about zero to 30 m. The eolian sand is underlain by Quaternary alluvium, consisting primarily of clayey sand inter-bedded with gravel. The alluvial unit comprises the principal permeability within the widespread Al Ain aquifer. The aquifer in the Jabeeb area is underlain by less permeable material, mostly clays and siltstone, ranging in age from Tertiary to Cretaceous.

The base of the alluvium is recognizable on most petro-physical logs and by TEM due to the sharp contrast between the resistivity of the two units. The base of the alluvium, or, in general, the top of the clay and siltstone is recognized as the base of Al Ain aquifer.

TEM TECHNIQUES

Two sets of equipment, EM-47 and EM-37, manufactured by Geonics, were selected to conduct TEM soundings in the study area. In both systems, portable transmitters send square wave DC current with known frequency and amperage into a wire transmitter loop laid out as a square on the ground. The current in the wire induces sets of eddy currents which decay as they move downward and outward in the ground. The magnetic field associated with the eddy currents induces voltage, or transients, in a small sensitive receiver coil placed at a fixed position in or near the transmitter loop. The coil is connected by cable to a receiver (Protem) which measures and digitally records the induced transient.

The Protem unit samples the coil response to the induced transient at a series of time intervals that are delayed by a prescribed amount from each turn-off of the loop current. This time-domain sampling of induced voltage data may be interpreted in terms of depth and formation resistivity because the coil response primarily is a function of both parameters (Fitterman, 1987, Spies, 1989).

Each EM system has a specific transmitter loop receiver coil system designed to measure ground resistivities over differing time and depth ranges. Key factors in determining the minimum and maximum depth ranges of measurement are current strength, loop size, length of sampling time, resistivity of formations, and electro-magnetic noise level of the ground itself (Spies, 1989).

The EM-47 system uses a battery-driven transmitter that supplies a 3-ampere current into a square loop 40 meters on a side (fig. 3a). The maximum depth of investigation is about 100 meters. On the other hand, the EM-37 system uses a gasoline-powered portable generator as a power source to supply 120 volts, 30-ampere square wave DC current. The standard size of the loop is 100 meters, which may be increased, if needed (fig. 3b). The maximum depth of investigation by the EM-37 method is about 200 meters.

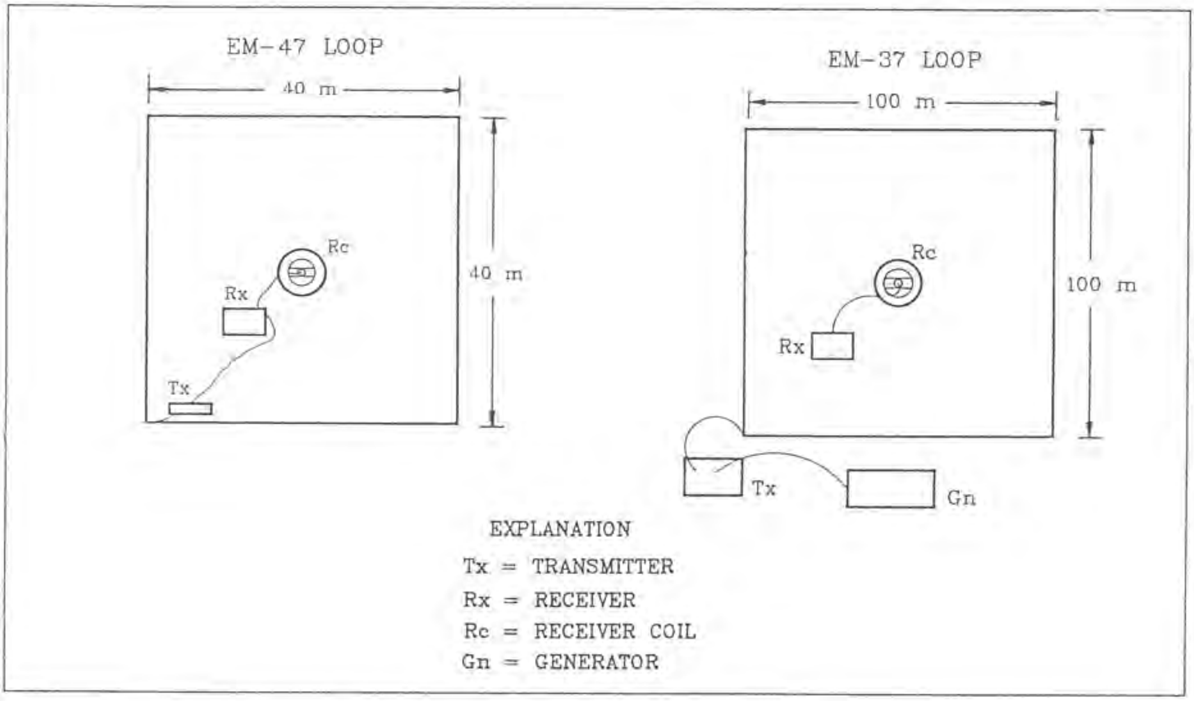


Figure 3. Configurations of EM-47 and EM-37 loops.

FIELD SURVEYS AND DATA INTERPRETATION PROCEDURE

TEM surveys are most often conducted in the form of profile lines oriented transversely across suspected water-bearing formations (Al Za'afarani, 1992). The length of each profile may range from 2.5 to 10 km. The measuring stations (loops) may be separated by equal distances or according to the geomorphic configuration of the area. The elevation and coordinates of each station are measured and plotted.

The data records for each TEM sounding station are downloaded to a personal computer. After eliminating noisy data records measured at ultra high and high frequencies, the data are averaged. The net result for each sounding is a composite set of 40 apparent resistivities that typically define a smoothly descending curve when plotted against the sample times of the associated data channels (fig. 4a). The curve actually is composed of two partially overlapping branches, each with 20 points, which correspond to the ultra high and high transmitting frequencies used in each sounding. The average data set for each TEM loop is entered and interpreted using a one-dimensional layered earth model, derived by forward and inverse algorithms using software from Interprex Ltd. (fig. 4b).

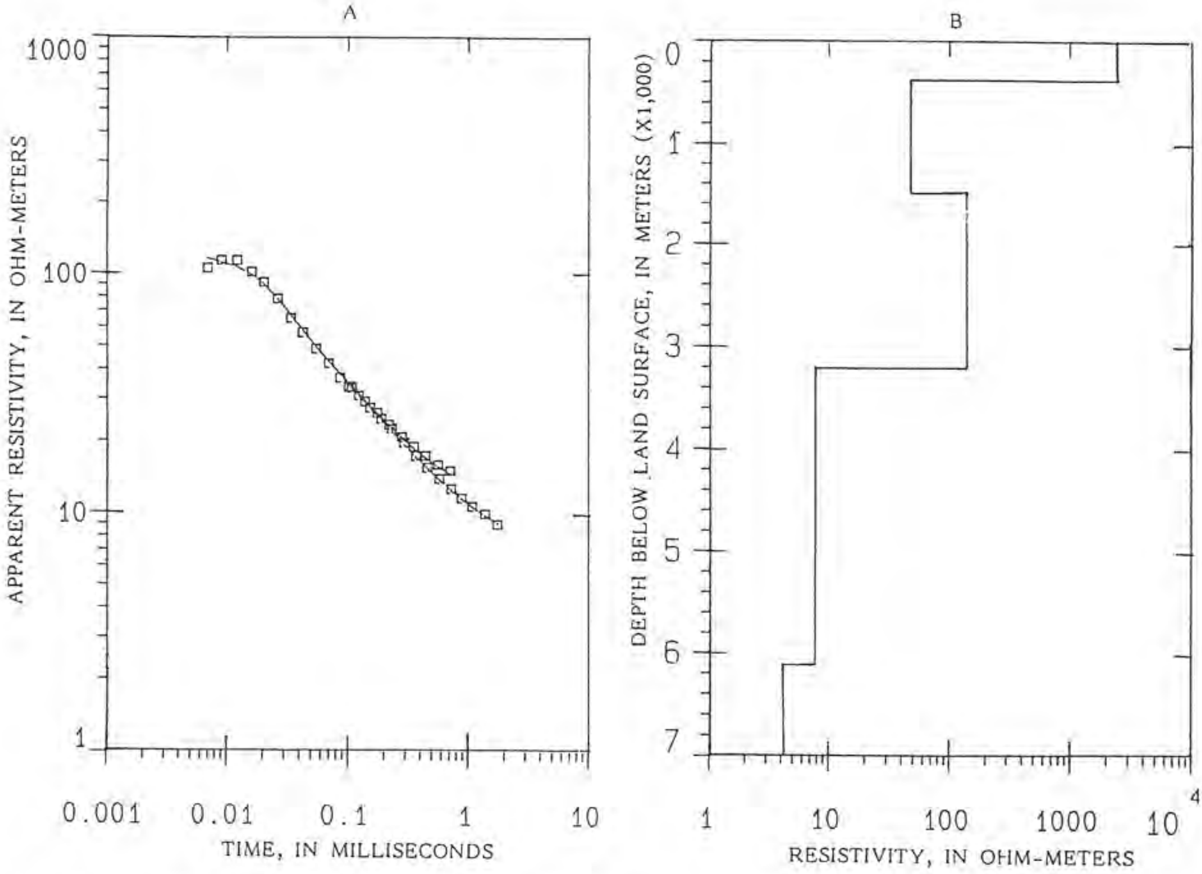


Figure 4. Data curve (A) and interpreted resistivities (B) for TEM model layers at Al Jabeeb.

RESULTS OF AL JABEEB PROFILE LINES

Before marking the profile lines for TEM surveys, an analysis was made of existing data for the area, including geologic maps, lithology from test wells, topographic maps, areal photos and satellite images. Based upon analysis of the existing data, areas of promising hydrologic potential were selected for TEM surveys. TEM surveys were conducted along seven lines with variable lengths and number of loops depending mainly upon the geomorphologic conditions (fig. 5). Sites for drilling were selected based upon the results obtained from TEM surveys.

Resistivity models were developed for the TEM loops measured along the profiles. The main layers derived from the interpretation of TEM data. Based on four and five layered models are:

(1) a highly resistive surface layer of unsaturated eolian sand;

(2) an intermediate resistive layer of unsaturated mixed sand, silt, and clay, which probably is cemented alluvium;

actual depth determined by drilling. At 10 sites, TEM predicted the depth to the base of alluvium within plus or minus 2 m, and this is equivalent to one standard deviation.

The geo-electric cross section in figure 6 illustrates the correlation of TEM loops and shows the lateral extension of the aquifer. Layer 4 of the geo-electric column represents highly permeable material, and, where saturated comprises Al Ain aquifer. Resistivity generally is below 20 ohm-meter for this layer. The resistivity contrast between the saturated and unsaturated zones in layer 4 was too weak to enable definition of the water table using TEM. The relatively low resistivity in this layer likely indicates the dampening effect of the highly conductive subjacent bedrock.

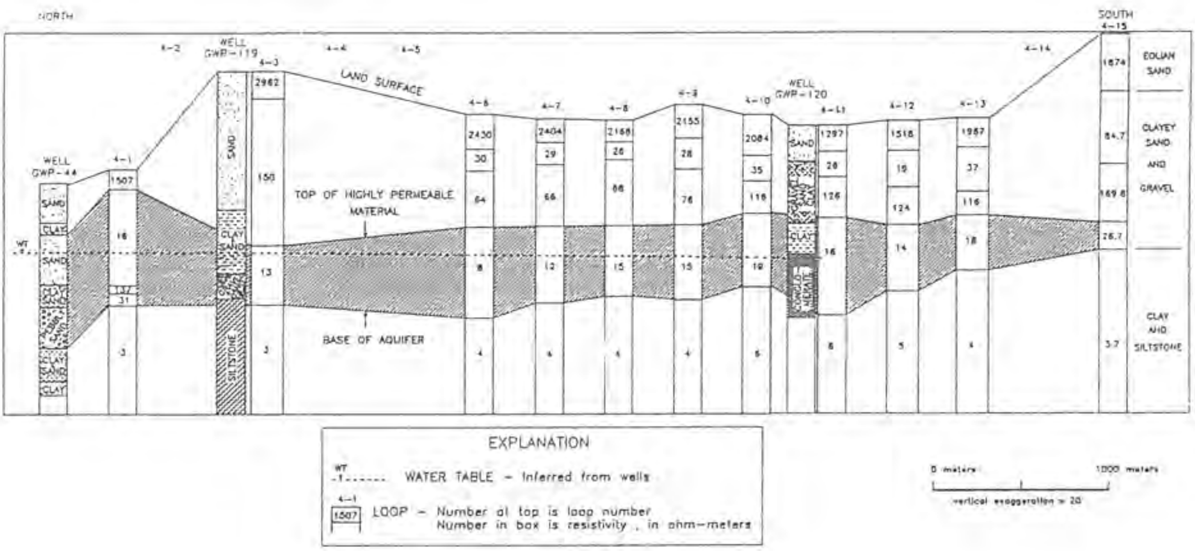


Figure 6. Five-layered geoelectric cross-section along line 4, Al Jabeeb.

SUMMARY AND CONCLUSIONS

Transient electromagnetic surveys are very useful in acquiring hydrologic data in arid regions. The surveys were implemented to define the thickness of the aquifer zone and basal contact of resistive alluvium overlying conductive bedrock. In Al Jabeeb area, TEM soundings were successful for determining the depth to the base of Al Ain aquifer at most sites, with a typical accuracy of plus or minus 2 meters. The technique also was of great use in selecting drilling sites with a relatively high hydrologic potential.

TEM could not be successfully applied to calculate the depth of the water table in most models. Models derived from TEM surveys are most reliable and give the best results when calibrated against boreholes with induction and lithologic logs.

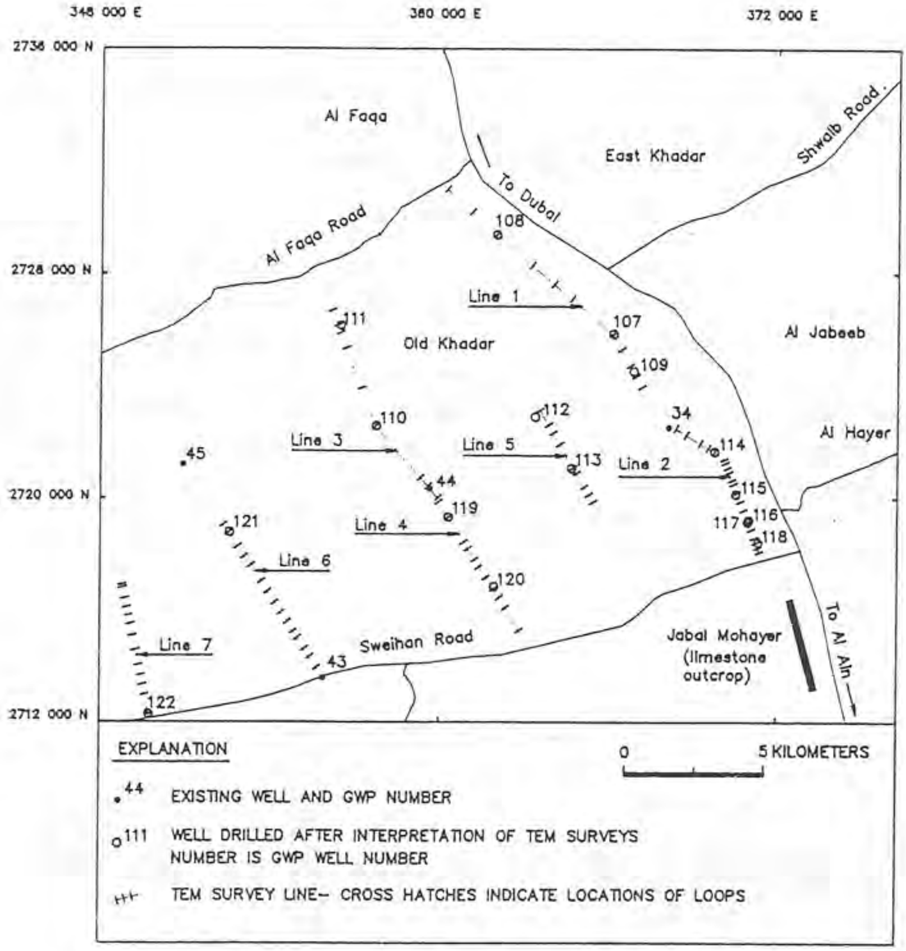


Figure 5. Transient electromagnetic survey lines in Al Jabeeb area.

(3) a resistive layer consisting of unsaturated alluvium, composed of clayey sand and gravel;

(4) an intermediate resistive layer of highly-permeable fully-to-partially-saturated alluvium, composed of pebbly sand, gravel, and conglomerate;

(5) a conductive layer generally of clay or siltstone.

The configuration of the alluvium-siltstone contact was the prime target of the TEM surveys. Depressions in this contact were suspected to represent buried wadi channels, which were likely to contain highly-permeable material. The depressions also represent low points in the ground-water reservoir and were likely to have greater saturated thickness than other areas.

Fifteen GWP borehole sites were used to compare values of depth to base of the alluvium estimated from TEM surveys with values from petrophysical and lithologic logs. The estimated depth based on TEM ranged from 12 m above to 13 m below the

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Khorsan' Ground Water Quality and Purification

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KHORASAN's GROUND WATER QUALITY AND PURIFICATION

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ABSTRACT

Ground water quality of Khorasan Province of the Islamic Republic Of Iran (I.R.Iran) has been investigated. As we expected, much variety have been observed in the Khorasan's groundwater quality. For example, the variation of the total hardness of the province has been observed to be from as low as 50 to as high as over 1000 mg/L as calcium carbonate. The amount of cations, anions, temporary and permanent hardness, alkalinity and etc. for each city have been presented and consequently the quality of the Khorasan groundwater has been investigated. All analysis have been compared with universally accepted standards. Finally, according to the results, suitable methods of purification, using "Teras" powder has been provided to improve the groundwater of the Khorasan province by about 70% reduction in the total hardness.

KEYWORDS

Calcium, magnesium and total hardness, bicarbonate, sulfate, sodium, chloride, cations, anions, groundwater, treatment, and Teras powder.

INTRODUCTION

Khorasan province with twenty major cities is located in the north part of I.R. Iran with an area of about 313000 kilometer square which is equivalent to about 1/5 of the I.R. Iran's total land area. The whether is moderate with distinct seasons. Average yearly rainfall ranges from 75 to 500 mm.

Water is a good solvent. It dissolves minerals while passing through the lagers of the ground. A large portion of these materials are: calcium, Magnesium, carbonate, bicarbonate, chloride, sulfide and nitrate. There is also a possibility of the existence of some other elements such as iron, manganese, aluminum, copper, zinc, bromide, phosphorous, silicate and fluoride in groundwater. The quality of water differs with regard to the kind and amount of dissolved minerals and chemical constituents. Therefore, it was tried to classify the groundwater in Khorasan province in the Islamic Republic of Iran (I.R. Iran) on the basis o their quality. This has been done by collecting data from over 350 samples from 17 cities of the province. The name of cities with the Greek numbers are shown in Fig. 1.

CITY CODE	CITY NAME
I	Bojnourd
II	Shirvan
III	Ghouchan
IV	Dara-gaz
V	Asfarayen
VI	Neyshabour
VII	Mash-had
VIII	Sabzevar
IX	Kashmar
X	Torbat-haydariye
XI	Torbat-jam
XII	Taybad
XIII	Tabas
VX	Ferdous
XV	Gonabad
XVI	Ghayenat
XVII	Birjand

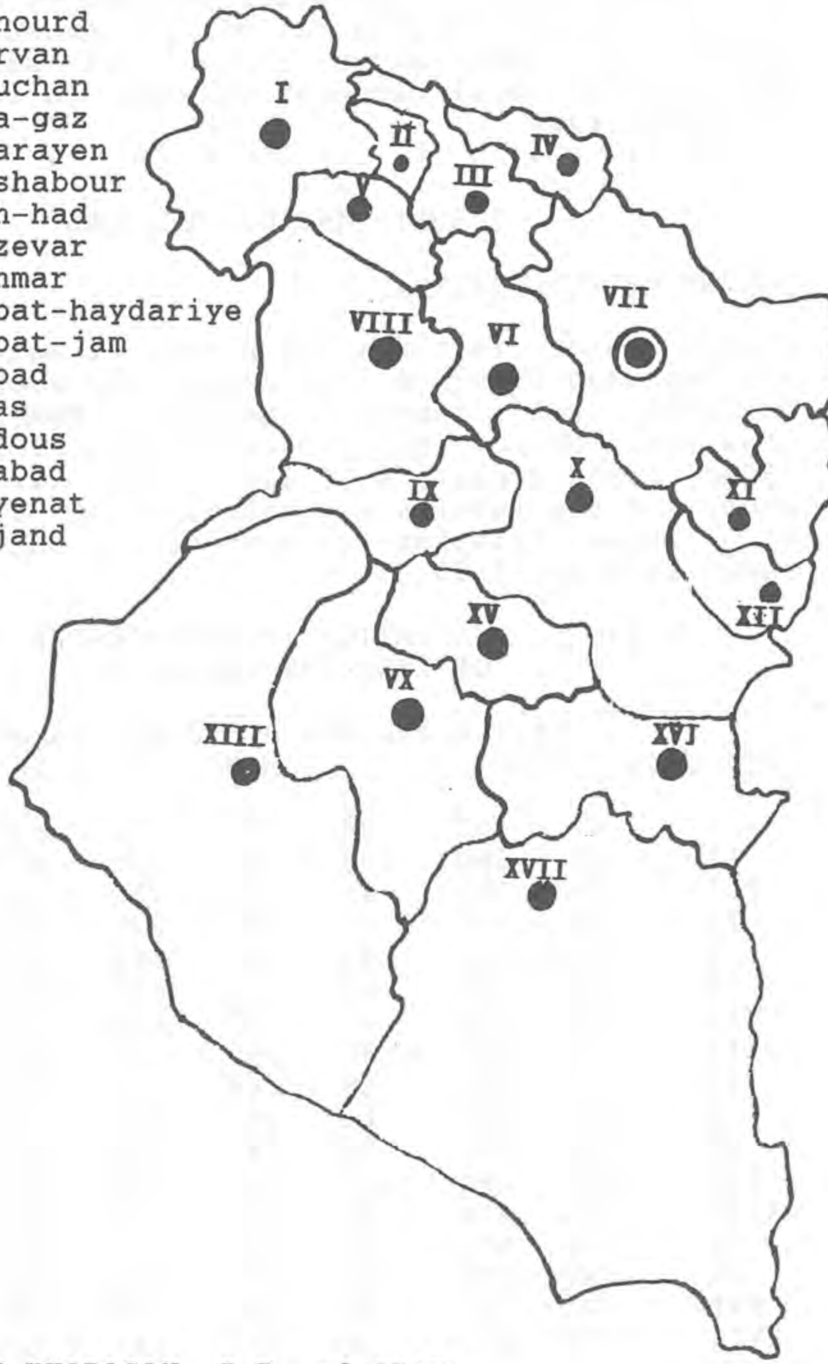

Fig. 1. MAP OF KHORASAN, I.R. of IRAN

In the later part of this paper, the city Greek numbers are used. It was the objective of this study to examine the quality of the Khorasan province and to suggest simple type of treatment.

For the first purpose of this study, ground water quality of Khorasan Province of the Islamic Republic Of Iran (I.R.Iran) was examined. It has been observed that the variation of the total hardness of the Khorasan province was from as low as 50 to as high as over 1000 mg/L as calcium carbonate. Similar variations have been observed in the amount of cations, anions, temporary and permanent hardness, alkalinity and etc. for the province. The other purpose of the study was to find a suitable methods of purification. "Teras" powder has been shown as a good sand powder to decrease the total hardness of water in less than 8-hr of mixing time.

RESULTS AND DISCUSSIONS

Ground Water Quality:

Ground water is the major source of water supply for the Khorasan Province. The parameters such as calcium hardness (CaH), magnesium hardness (MgH), total hardness (TH), sodium (Na+), bicarbonate (HCO3-), sulfate (SO4=), and chloride (Cl-) were determined in this study and the results are tabulated in Tables I and II. These variation for some of the major cities are shown in Figs. 2 TO 5.

Table I. VARIATION IN GROUNDWATER CATIONS OF KHORASAN PROVINCE

	(All units are expressed in mg/L as CaCO3)			
CITY CODE	CaH	MgH	TH	Na+
I	47 - 455	28 - 405	213 - 845	25 - 275
II	92 - 360	109 - 286	265 - 612	50 - 250
III	109 - 305	41 - 226	150 - 523	75 - 234
IV	128 - 400	62 - 430	230 - 530	40 - 250
V	50 - 320	47 - 245	112 - 560	35 - 170
VI	54 - 320	68 - 275	132 - 546	20 - 230
VII	34 - 420	49 - 378	135 - 633	25 - 990
VIII	22 - 150	31 - 495	53 - 560	25 - 562
IX	42 - 196	29 - 159	98 - 350	30 - 495
X	39 - 452	14 - 360	126 - 812	100 - 612
XI	88 350	78 - 343	224 - 652	25 - 522
XII	140 - 395	103 - 312	266 - 695	100 - 606
XIII	37 - 414	34 - 408	71 - 812	225 - 998
VX	92 - 408	81 - 302	195 - 511	52 - 509
XV	20 - 395	43 - 375	31 - 645	50 - 895
XVI	62 - 593	36 - 542	98 - 730	100 - 999
XVII	68 - 310	53 - 375	197 - 593	100 - 500

Table II. VARIATION IN GROUNDWATER ANIONS OF KHORASAN PROVINCE

	(All units are expressed in mg/L as $CaCO_3$)		
CITY CODE	HCO_3^-	$SO_4^=$	Cl^-
I	35 - 400	20 - 295	24 - 250
II	175 - 350	87 - 318	46 - 232
III	100 - 400	50 - 161	28 - 126
IV	115 - 350	75 - 736	23 - 198
V	75 - 200	11 - 195	14 - 160
VI	85 - 323	45 - 255	42 - 215
VII	80 - 410	50 - 800	14 - 704
VIII	98 - 250	15 - 239	35 - 461
IX	134 - 376	60 - 340	34 - 206
X	105 - 409	20 - 250	30 - 963
XI	125 - 310	30 - 436	21 - 508
XII	218 - 375	95 - 576	56 - 535
XIII	65 - 250	190 - 725	57 - 988
VX	135 - 200	50 - 500	52 - 476
XV	65 - 360	35 - 446	34 - 746
XVI	125 - 460	35 - 513	54 - 926
XVII	165 - 495	75 - 523	85 - 363

From these data, and according to the water quality standards, the degree of hardness in the groundwater of the Khorasan province for different states are given in Table III.

Table III. DEGREE OF HARDNESS IN GROUNDWATER OF KHORASAN PROVINCE

	(All units are expressed in %)			
TH (mg/L):	<100	100><300	300><500	>500
CITY CODE	Soft	Moderate	Hard	Psuedo-Hard
I	0.0	28.0	45.0	27.0
II	0.0	38.0	47.0	25.0
III	0.0	46.0	34.0	20.0
IV	0.0	32.0	44.5	23.5
V	0.0	27.5	51.0	21.5
VI	0.0	39.0	43.5	17.5
VII	0.0	30.8	59.0	10.2
VIII	19.5	18.7	42.3	19.5
IX	23.0	46.0	31.0	0.0
X	0.0	31.0	46.0	23.0
XI	0.0	30.5	44.5	25.5
XII	0.0	34.5	29.5	36.0
XIII	13.0	25.5	33.5	28.0
VX	0.0	32.5	42.5	25.0
XV	24.0	26.0	17.0	33.0
XVI	17.5	22.5	28.5	31.5
XVII	0.0	23.0	44.5	32.5

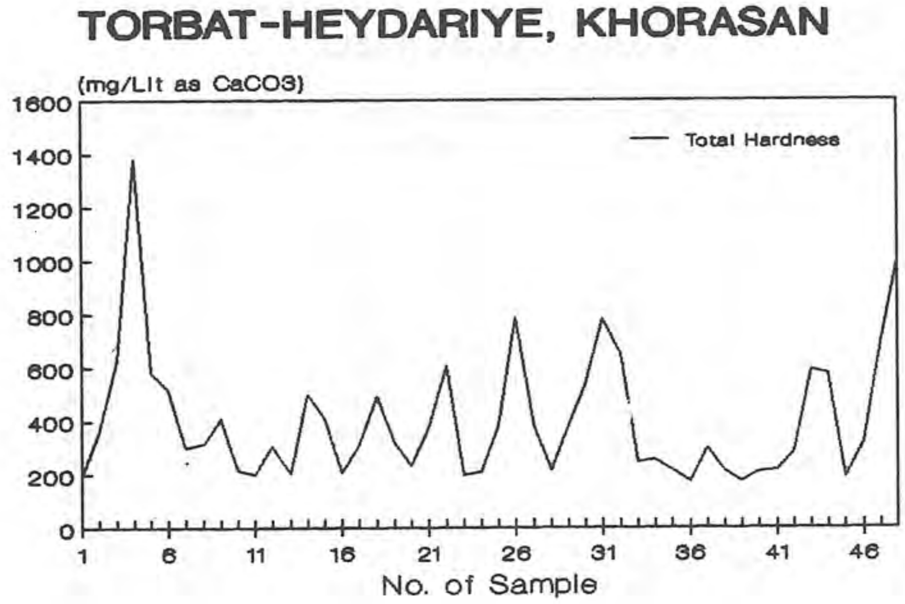

Fig. 2. TOTAL HARDNESS VARIATIONS OF TORBAT-HEYDARIYE, KHORASAN

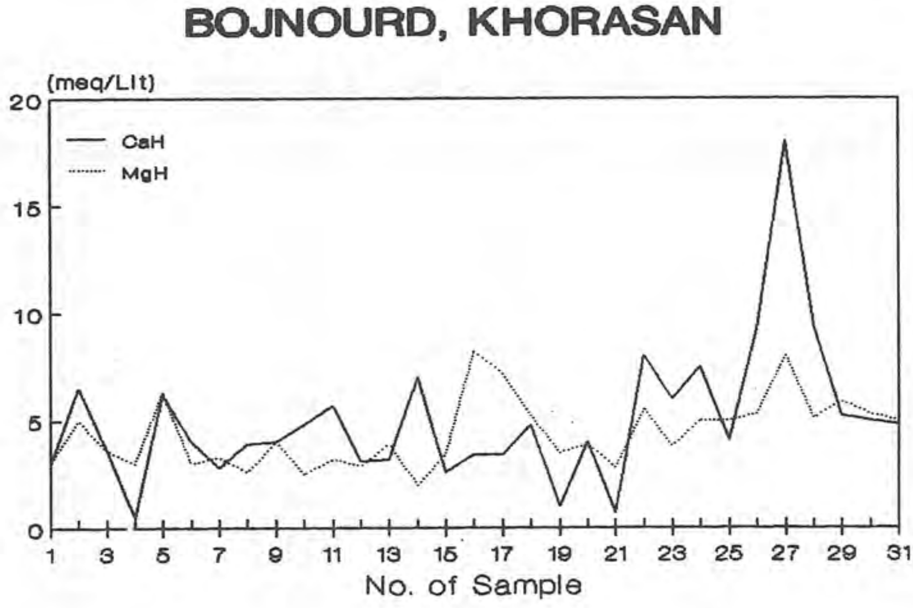

Fig. 3. CALCIUM & MAGNESIUM HARDNESS VARIATIONS OF BOJNOURD, KHORASAN

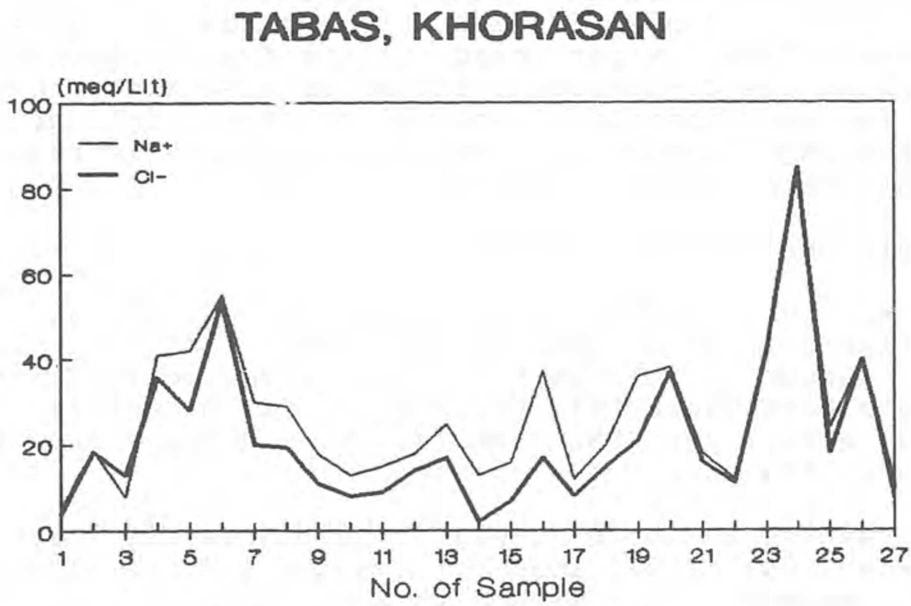

Fig. 4. SODIUM & CHLORIDE VARIATIONS OF TABAS, KHORASAN

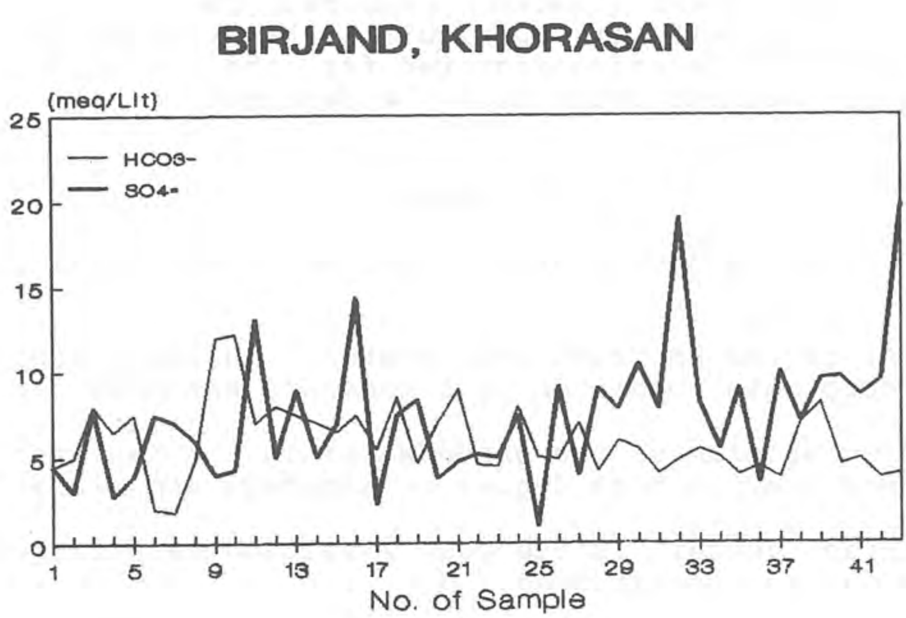

Fig. 5. CARBONATE & SULFIDE VARIATIONS OF BIRJAND, KHORASAN

The results indicated that the hardness in most of the Khorasan's groundwaters are in the range of accepted values. These water need to be disinfected before reaching the consumers as the major treatment process. But for the minority of the wells having high hardness as the major impurities, softening process is required to be included in the treatment plant.

Simple Purification Method:

Teras stone powder which is a kind of natural stone available in large quantity and cheap in I.R. Iran, was suggested for the treatment of psuedo-hard water. It has been shown that this kind of materials have good effect on the removal of hardness from hard waters (Fig. 6).

The results as shown in Fig. 6 indicates the relation between one time shaking in a day and the degree of the treatment. This method is very simple and needs low energy and it is very economical, but needs to wait four days to have about 65% hardness removed. Continuous mixing have been done in the laboratory using mixing time of 150 rpm from one to eight hours. The results are shown in Fig. 7. The results indicate that mixing time of 8-hr for Teras powder doses from 0.5 to 4.0 weight percent were the optimum.

This results also indicate that depending on the degree of hardness removal required for a special water, the cost of chemicals, the price of mixing equipment and the time required for the reaction, a type of treatment plant could be designed.

CONCLUSIONS

The following conclusions can be drawn from this study:

1. Most of the Khorasan groundwaters quality are in the acceptable condition with moderate hardness.

2. Major problem of the groundwater in the Khorasan province are the high degree of hardness and nitrate.

3. Teras powder is the most effective material used for reducing the hardness.

4. Mixing time of 8-hr is suggested for the Teras powder dosage of 0.5 to 4.0% by weight.

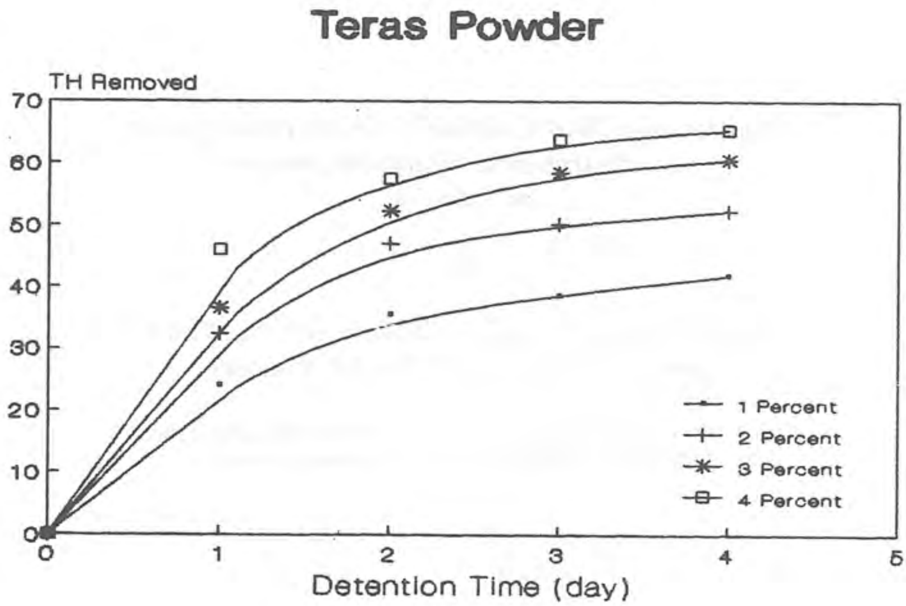

Fig. 6. ONE TIME MIXING EFFECT OF TERAS POWDER ON HARDNESS REMOVAL

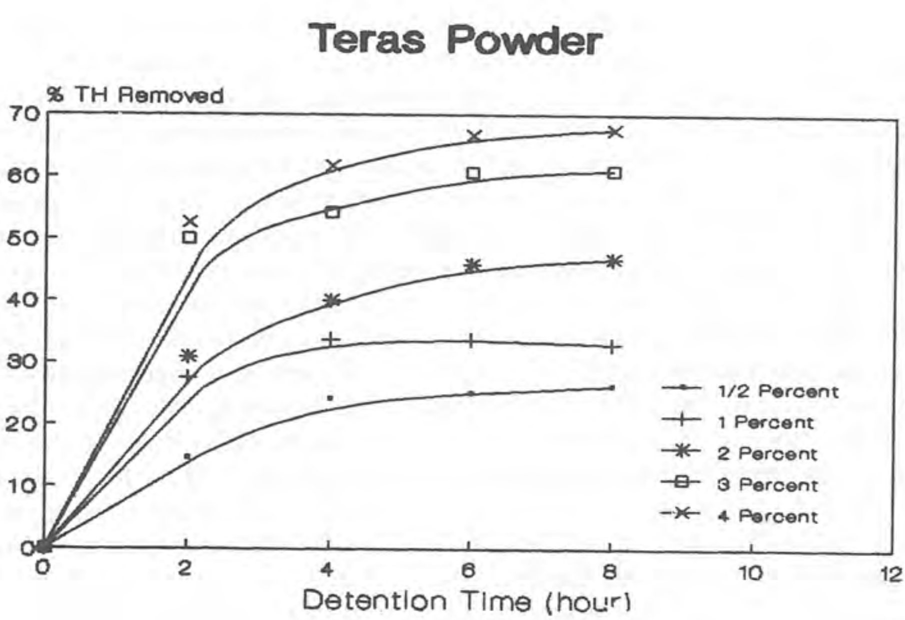

Fig. 7. CONTINUOUS MIXING EFFECT OF TERAS POWDER ON HARDNESS REMOVAL

Selection of a Pilot Catchment for the Development of a Hydrological Monitoring System in Palestine

By

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Abstract

The development of a hydrological monitoring system for the Occupied Palestinian Territories (OPT) is important for the management of water resources in the region. The efficient way to do this is to develop indigenous water resources management capabilities through the collection, storage and analysis of all hydrological data. The first step in this direction is to select a study (pilot) catchment in the OPT suitable for locating hydrological monitoring equipment. First, a list of criteria for catchment selection has been prepared by the team of University of Newcastle upon Tyne (UNUT). Subsequently a survey of water resources related data has been carried out in the UK and OPT. The data were manipulated and analyzed by UNUT and PHG using Geographical Information System (GIS) and tabular analysis procedures. Field verification of the final catchment selection was then undertaken. Zeimar catchment in North West Bank (NWB) was the choice.

1. Background and Objectives

The 6-day war of 1967 was a major crisis to the Palestinian nation, not only because the Israelis have raped the Palestinian lands and forced hundreds of thousands to flee their homes, but also because the Israelis placed all the Palestinian water resources under the strict control of the military authorities. Moreover, the Israelis have systematically drilled huge numbers of deep wells in the Palestinian lands to supply Jewish settlements and at the same time the Israeli military authorities imposed severe limitations on Palestinian water use. Benvenisiti (1986) stated that the Israelis exploited 95.5% of the West Bank (see Figure 1) water. There is no example better than the experience of Rowley (1990) who witnessed (during his residency in Balata camp (a refugee camp of about 30,000 population) in 1986) that the water supply to the camp was strictly limited to 2 hours per week, while any adjacent Israeli settlement had water running in the taps 24 hours per day. Subsequently, skilled Palestinians found it extremely difficult to return to the Occupied Palestinian Territories (OPT) under the affliction of Occupation. The lack of skilled Palestinians (in the field of water resources) has received less attention than it deserves (Grey, 1992). It was the British Government through its Overseas Development Administration (ODA) who started preparing for a project (in 1991) which would help to develop Palestinian capabilities in water resources management (Gowing and Parsons, 1991). The ODA project is in place now and has been integrated with another project (funded by the EU) aimed at selecting a pilot catchment where a network of hydrological monitoring equipment can be installed and operated.

The development of indigenous water resources management capabilities should include (Parsons, 1992) collection, storage and analysis of all basic hydrological and hydrogeological data. This will be achieved by providing experience of the operation of the hydrological monitoring instruments, the recording, processing and analysis of the data they generate.

The objective of this paper is the presentation of the general approach used in identifying potential catchments for the installation of hydrological monitoring system.

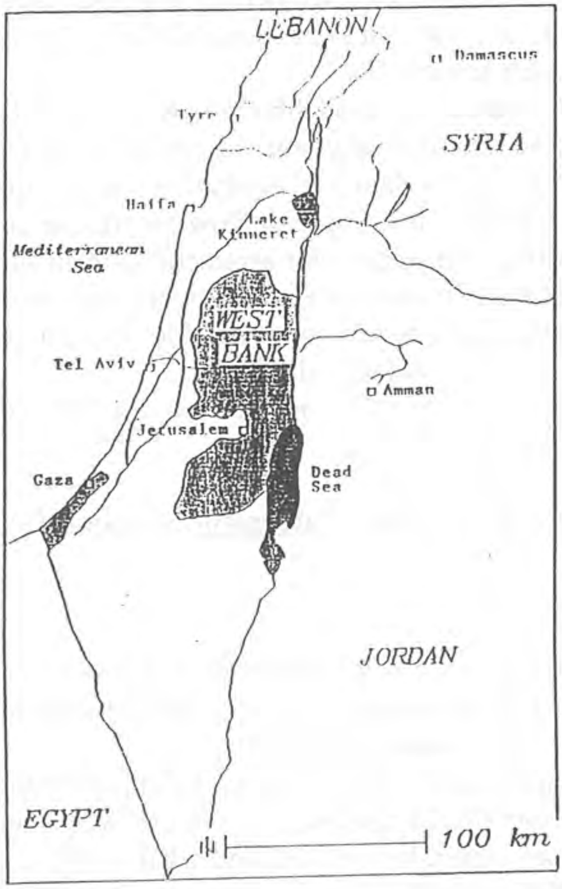

Figure 1: West Bank Map

2. Establishing Criteria for Catchment Selection in the OPT

The general term 'catchment' means a drainage basin, separated from adjacent catchments by a topographic divide (watershed). In establishing criteria for catchment selection in the OPT, it is important to take into consideration the following:

- the catchment is required for monitoring and training purposes;
- the catchment will be used for the purpose of understanding a water resources system and evaluating the management alternatives;
- the particular constraints in the OPT such as lack of access to some areas (either because these are military areas or because they are Israeli settlements) and limited availability of historic record water data, due to the strict control exerted over water and related data in the OPT by Israeli authorities.

The criteria adopted for catchment selection are as follows:

1) There should be a good stream flow gauging site at the catchment outlet.
2) There should be good access to the catchment area, and particularly to locations on the surface drainage system in wet weather for gauging floods.
3) As little as possible of the catchment should have ''restricted'' areas, such as military areas or Israeli settlements.
4) Sites and observers should be available for the hydrometric and meteorological equipment - especially ''secure'' sites for the climate stations.
5) Groundwater ''access'' points should be available - ideally through existing or disused wells. Access to springs to measure their flow before use is also important.
6) Information on existing (and past) water abstraction (from surface water/springs and groundwater) should be available, or else relatively easy to estimate.
7) Ideally, some historic data should be available for groundwater levels, spring discharges, surface runoff, rainfall and climate.
8) The size of the catchment should be neither too big nor too small, perhaps in the range of 50 to 200 km^2.

3. UNUT/PHG Approach to Satisfy Catchment Selection Criteria

3.1 General Procedure

When the ODA/EEC funded projects were approved in the summer of 1993, the teams of PHG and UNUT decided to follow separate approaches whilst maintaining a reasonable level of cooperation according to the following procedure:

1. An early decision was made, on the basis of data availability that the catchment should be selected from the 27 catchments of North West Bank (NWB). (Figure 2).
2. A search for data was undertaken in OPT and UK by PHG and UNUT respectively.
3. Data collected by PHG and UNUT were manipulated and analyzed by the two teams using different methodologies.
4. Each team came out with its own short list of catchments which have advantages over the rest of the 27 catchments of NWB.
5. The two teams met in the field and made their final selection from the two short-lists.

3.2 UNUT Approach

UNUT approach was based on the use of GRASS and ARC-INFO GIS packages to manipulate and analyse data from a report by Rofe and Raffety (1965) as follows:

1) use of ARC-INFO to digitise the data for a series of maps of the NWB. These maps show the following features: catchments boundaries, streams, roads, Palestinian

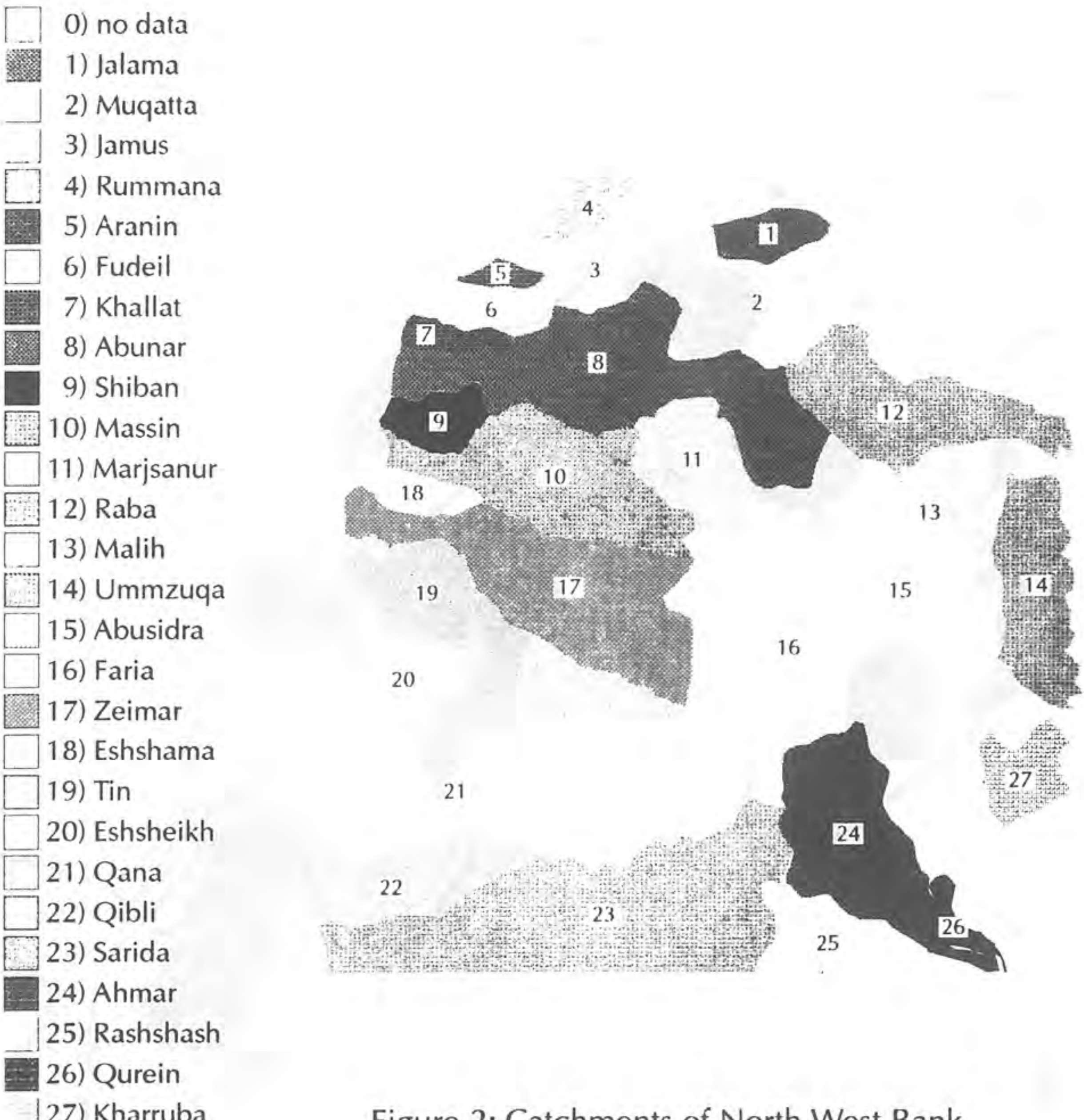

Figure 2: Catchments of North West Bank

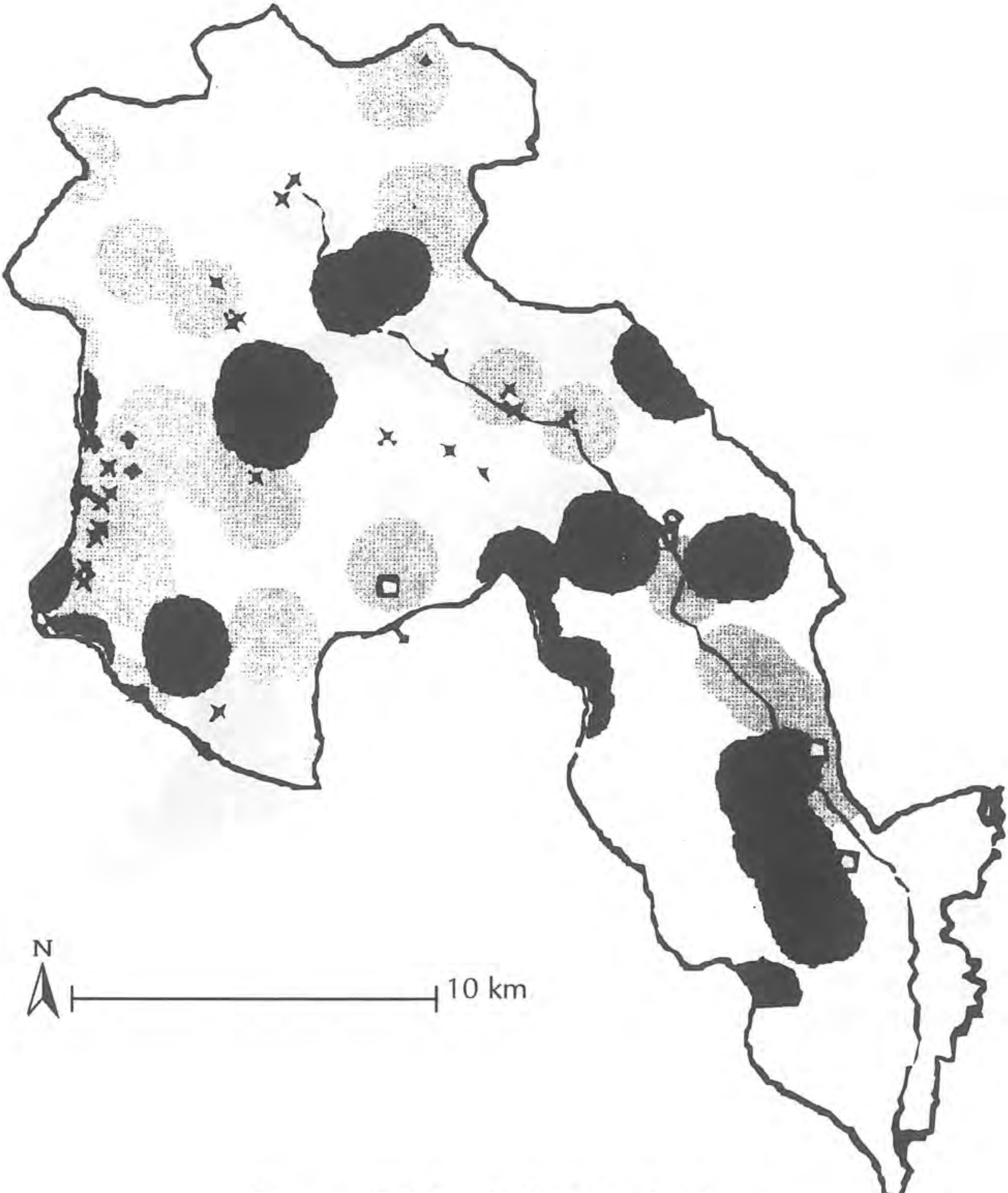

Figure 3: GIS Results for Faria Catchment

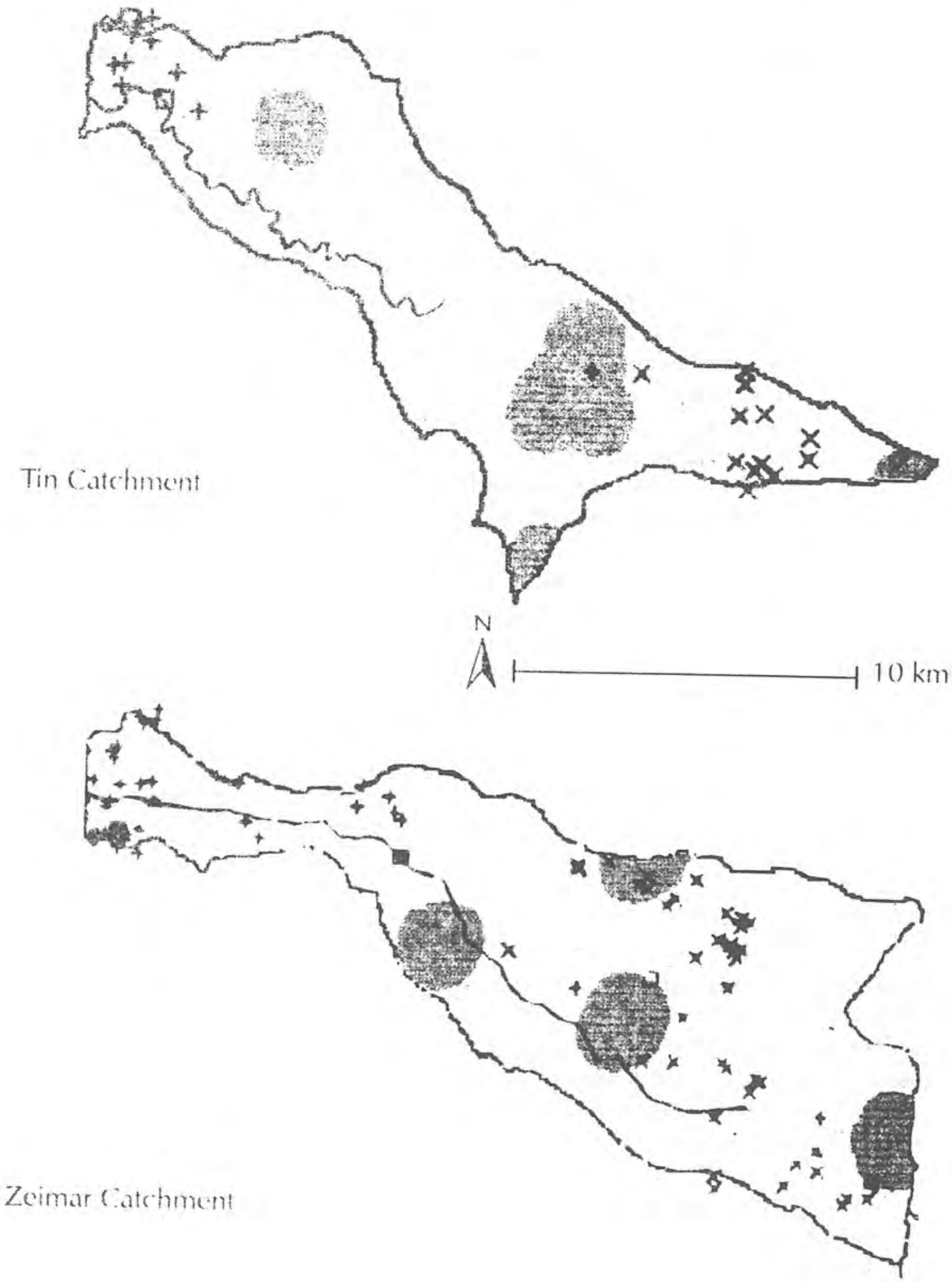

Figure 4: GIS Results for Tin and Zeimar Catchments

communities, Israeli settlements, rainfall, groundwater movement, hydrometric and hydrologic station locations (in the 1960's), soil moisture retention zones, spring locations, well locations and actual recharge;

2) with GRASS, the catchment selection criteria (section 2) were evaluated as follows:

- the surface areas of 27 catchments were calculated;
- the locations of hydrometric and hydrologic stations in 1960's were used as a guide to the availability of potential monitoring sites;
- maps of roads and river networks were plotted together in order to assess ease of access;
- the percentage of a catchment covered by Israeli settlements and or ''restricted'' areas was calculated. This area includes a buffer zone of 1 to 5 km around each of these areas;
- the term ''secure'' in the catchment selection criteria is taken to mean at least 1 km from an Israeli settlement and within or bordering a Palestinian community. These criteria are chosen in order to maximise the possibility that: sympathetic observers should be available, the equipment will be safe from theft or vandalism and the observers will not be in unnecessary danger by approaching restricted areas. The locations of hydrometric and hydrologic sites were plotted in relation to the distribution of secure areas; this aided the identification of ''suitable'' and ''unsuitable'' sites;

3) Some results of the analysis described in (2) are presented in Figures 3 and 4 (where red zones: Israeli Settlements with 1 km buffer zone, blue zones: Palestinian communities with buffer zones, yellow zones: roads with buffer zones, black line within the catchment: rivers, plus sign: wells, cross sign: springs, boxes: discharge gauges if on the river and meteorological sites elsewhere). For more details the reader is advised to consult Aliewi et al (1993).
4) Having collated all of the relevant data in maps, tables and graphs it was a relatively simple exercise to compile the following short-list: Zeimar-17, Tin- 19 and Faria-16

3.3 PHG Approach

Data suitable for evaluating the catchment selection criteria were compiled from the PHG database which were all collected in the field. A review of the sources and nature of these data is given in Rabi et al (1994). Basic information about NWB catchments is presented in Table 1. Tables 2, 3, 4 and 5 present specific details on the springs, wells, rainfall stations, and hydrological and meteorological stations respectively. The data in the five tables were evaluated as follows:

1. All catchments were classified into 3 classes: Class 1 (C1): catchments unlikely to be selected, Class 2 (C2): catchments unlikely to be rejected, Class 3 (C3): catchments likely to be selected.
2. The decision of which class each catchment should belong to was made on the evaluation to the following parameters: Area of the catchment, A (Km^2), Main Stream Length, MSL (Km), Ratio of no. of Israeli settlements to no. of Palestinian communities, ISPAL (%), no. of springs , NSP, no. of wells, NWE, Availability of Historic Records, AHR, Existence of Gauging Stations, EGS, and Accessibility, AC.

Table 1: Basic Information on NWB Catchments

Catchment ID	Catchment Area (Km^2)	Main Stream Length (Km)	No. of Palestinian Communities	No. of Israeli Settle-ments	No. of Springs	No. of Wells	No. of Met. Stations
Jalama-1	23.1	9.2	3	0	0	9	0
Muqatta'-2	112.7	17.0	7	3	5	32	1
Jamus-3	53.1	10.5	8	2	4	0	0
Rummana-4	19.4	7.0	2	0	4	2	0
Fudeil-6	27.7	5.5	11	1	2	1	0
Abu Nar-8	176.9	28.0	23	2	2	52	0
Sheiban-9	22.8	5.2	4	0	2	9	0
Massin-10	120.7	22.0	12	0	7	10	1
Marjsanour-11	58.5		7	0	0	4	0
Raba-12	96.0	16.0	10	0	22	8	0
Malih-13	119.5	14.0	5	1	6	3	0
Ummzuqa-14	75.2	5.0	3	1	1	7	0
Abusidra-15	69.0	15.0	4	1	0	1	0
Faria-16	375.1	36.0	21	5	24	69	2
Zeimar-17	156.6	24.5	30	3	34	21	1
Eshshama-18	16.3	8.0	1	0	1	4	0
Tin-19	122.3	18.0	15	3	8	21	1
Eshsheikh-20	77.9	9.0	11	3	0	12	0
Azzun		7.0	3	3	0	60	0
Kharruba-27	28.2	4.0	2	1	0	15	0

Table 2: Specific Details on NWB Springs

Catchment ID	No. of Non-Monitored Springs	Monitored Springs		
		No. of Monitored Springs	% of Monitoring Time for the Period, 1961-1989	Previous Gauging Stations
Jalama-1	0	0	–	0
Muqatta'-2	3	2	60	0
Jamus-3	4	0	0	0
Rummana-4	4	0	0	0
Fudeil-6	1	1	64	1
Abu Nar-8	2	0	0	1
Sheiban-9	2	0	0	0
Massin-10	2	5	7	0
Marjsanur-11	0	0	–	0
Raba-12	7	15	53	2
Malih-13	0	6	2 springs= 0	1
			3 springs= 7	
			1 spring= 53	
Ummzuqa-14	1	0	0	0
Abusidra-15	0	0	–	0
Faria-16	9	15	2 springs= 7	3
			13 springs= 82	
Zeimar-17	14	20	9 springs= 7	0
			7 springs= 82	
			1 spring= 57	
			2 springs= 5	
			1 spring= 1	
Eshshama-18	1	0	0	0
Tin-19	7	1	3	0
Eshsheikh-20	0	0	–	0
Azzun	0	0	–	0
Kharruba-27	0	0	–	0

Table 3 : Specific Details on NWB Wells

Catchment ID	No. of Wells		No. of Wells	
	Monitored[1]	Non Monitored	Used	Disused
Jalama-1	0	9	8	1
Muqatta'-2	20	12	21	11
Jamus-3	0	0	0	0
Rummana-4	0	2	2	0
Fudeil-6	0	1	1	0
Abu Nar-8	27	25	40	12
Sheiban-9	8	1	NA[2]	NA
Massin-10	6	4	NA	NA
Marjsanur-11	2	2	4	0.
Raba-12	7	1	4	4
Malih-13	2	1	NA	NA
Ummzuqa-14	1	6	NA	NA
Abusidra-15	0	1	1	0
Faria-16	49	20	65	4
Zeimar-17	13	8	16	5
Eshshama-18	1	1	2	2
Tin-19	15	6	NA	NA
Eshsheikh-20	11	1	NA	NA
Azzun	54	2	NA	NA
Kharruba-27	9	6	NA	NA

Table 4: Specific Details on NWB Rainfall Stations:

Catchment ID	No. of Rainfall Stations		Historical Records		
	Previous	Existing	Hourly	Daily	Monthly
Jalama-1	0	0	No	No	No
Muqatta'-2	3	3	No	Yes	Yes
Jamus-3	0	0	No	No	No
Rummana-4	1	1	No	Yes	Yes
Fudeil-6	2	2	No	Yes	Yes
Abu Nar-8	2	2	No	Yes	Yes
Sheiban-9	1	1	No	Yes	Yes
Massin-10	2	3	No	Yes	Yes
Marjsanur-11	1	1	No	Yes	Yes
Raba-12	1	1	No	Yes	Yes
Malih-13	2	2	No	Yes	Yes
Ummzuqa-14	0	0	No	No	No
Abusidra-15	0	0	No	No	No
Faria-16	6	3	No	Yes	Yes
Zeimar-17	3	3	Yes	Yes	Yes
Eshshama-18	0	1	No	Yes	Yes
Tin-19	2	2	Yes	Yes	Yes
Eshsheikh-20	1	2	No	Yes	Yes
Azzun	2	2	No	Yes	Yes
Kharruba-27	0	0	No	Yes	Yes

1 All monitored wells have records for monthly abstraction rates for the two hydrological years 1976/77 and 1977/78.

2 NA= Not Available

Table 3 : Specific Details on NWB Wells

Catchment ID	No. of Wells		No. of Wells	
	Monitored[1]	Non Monitored	Used	Disused
Jalama-1	0	9	8	1
Muqatta'-2	20	12	21	11
Jamus-3	0	0	0	0
Rummana-4	0	2	2	0
Fudeil-6	0	1	1	0
Abu Nar-8	27	25	40	12
Sheiban-9	8	1	NA[2]	NA
Massin-10	6	4	NA	NA
Marjsanur-11	2	2	4	0
Raba-12	7	1	4	4
Malih-13	2	1	NA	NA
Ummzuqa-14	1	6	NA	NA
Abusidra-15	0	1	1	0
Faria-16	49	20	65	4
Zeimar-17	13	8	16	5
Eshshama-18	1	1	2	2
Tin-19	15	6	NA	NA
Eshsheikh-20	11	1	NA	NA
Azzun	54	2	NA	NA
Kharruba-27	9	6	NA	NA

Table 4: Specific Details on NWB Rainfall Stations:

Catchment ID	No. of Rainfall Stations		Historical Records		
	Previous	Existing	Hourly	Daily	Monthly
Jalama-1	0	0	No	No	No
Muqatta'-2	3	3	No	Yes	Yes
Jamus-3	0	0	No	No	No
Rummana-4	1	1	No	Yes	Yes
Fudeil-6	2	2	No	Yes	Yes
Abu Nar-8	2	2	No	Yes	Yes
Sheiban-9	1	1	No	Yes	Yes
Massin-10	2	3	No	Yes	Yes
Marjsanur-11	1	1	No	Yes	Yes
Raba-12	1	1	No	Yes	Yes
Malih-13	2	2	No	Yes	Yes
Ummzuqa-14	0	0	No	No	No
Abusidra-15	0	0	No	No	No
Faria-16	6	3	No	Yes	Yes
Zeimar-17	3	3	Yes	Yes	Yes
Eshshama-18	0	1	No	Yes	Yes
Tin-19	2	2	Yes	Yes	Yes
Eshsheikh-20	1	2	No	Yes	Yes
Azzun	2	2	No	Yes	Yes
Kharruba-27	0	0	No	Yes	Yes

[1] All monitored wells have records for monthly abstraction rates for the two hydrological years 1976/77 and 1977/78.

[2] NA= Not Available

Table 5: Specific Details on NWB Hydrological and Meteorological Stations

Catchment ID	Hydrological Gauging Stations		Met. Stations	
	No. of Previous Sites	Existance of Sites Outside the Green Line	No. of Previous Stations	No. of Existing Stations
Jalama-1	0	No	0	0
Muqatta'-2	1	Yes	1	0
Jamus-3	1	Yes	0	0
Rummana-4	0	No	0	0
Fudeil-6	0	Yes	0	0
Abu Nar-8	1	Yes	0	0
Sheiban-9	0	Yes	0	0
Massin-10	1	Yes	0	0
Marjsanour-11	0	—	1	1
Raba-12	0	—	0	0
Malih-13	1	—	0	0
Ummzuqa-14	0	—	0	0
Abusidra-15	1	—	0	0
Faria-16	1	Yes	0	0
Zeimar-17	1	Yes	1	1
Eshshama-18	0	Yes	0	1
Tin-19	1	Yes	1	1
Eshsheikh-20	0	Yes	0	0
Azzun	0	Yes	0	0
Kharruba-27	0	—	0	0

Figure 5: Catchment ID Versus Catchment Frequency In Class 3

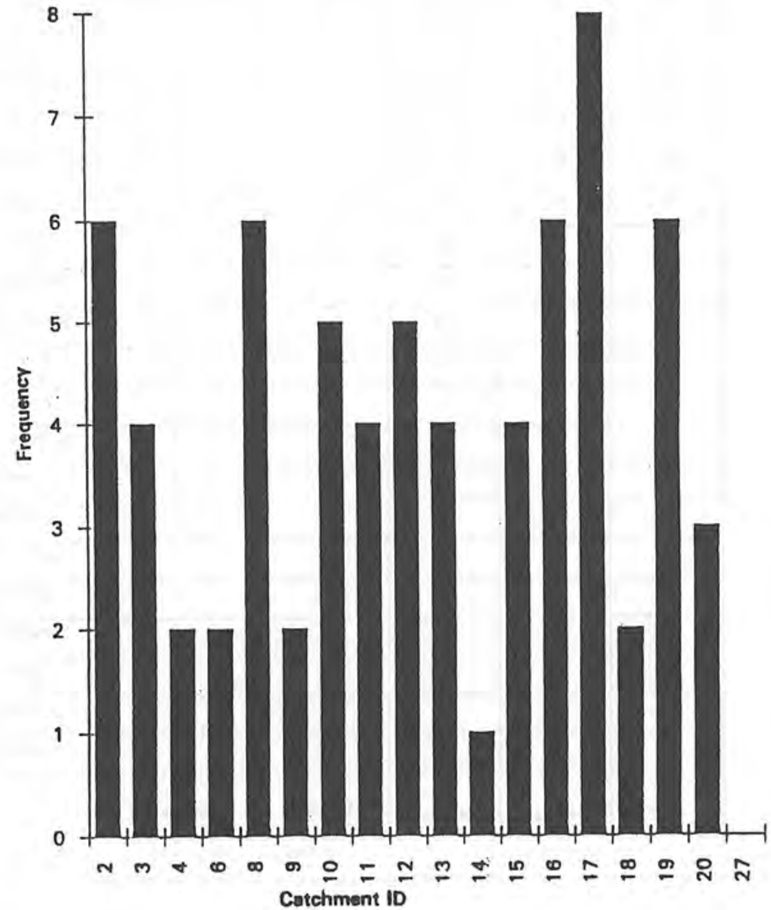

Table 6: Criteria for Catchment Classification

	C1	C2	C3
A	A<50, A>500	200<A<500	50<A<200
MSL	MSL<10		MSL>10
ISPAL	ISPAL>25		ISPAL<25
NSP	NSP<10	10<NSP<20	NSP>20
NWE	NWE<10	10<NWE<20	NWE>20
AHR	NO		YES
EGS	NO		YES
AC	NO		YES

3. The characteristics of each catchment of NWB were classified according to table 6. The frequency of each catchment in C3 was plotted in the histogram presented in Figure 5.
4. From figure 5, the following catchments can be shortlisted: Zeimar-17, Faria-16, Muqatta'-2, Tin-19 and Abu Nar-8

3.4 Finalisation of Catchment Selection in the Field

As seen above PHG and UNUT had arrived independently at Zeimar catchment as the most promising candidate for the study. The catchments in the short lists were then revisited by the PHG and UNUT teams. Full details of the visits and the suitability of Zeimar for the development of the hydrological monitoring system can be found in Younger and Strangeways (1993) . Also this catchment is suitable for the practice of water resources management. In particular there is scope for detailed analysis of the following problems: surface water pollution by sewage and quarry effluent; the provenance of waters in the catchment, either in terms of the two distinct groundwater flow systems in the area or in terms of the importation of water from other catchments to supply the City of Nablus (at the head of the catchment), questions of demand and supply and flood hydrology. Faria catchment is too large and deficient in the number of safe and accessible areas for installation of monitoring equipments. Abu Nar catchment has few springs (only 2) which made it unsuitable for the study. Muqatta' catchment has a large value of ISPAL (43%) which makes it unsafe for observers. Tin catchment is a suitable choice for the study but Zeimar has more detailed hydrological data as presented in Tables 1-5.

4. Conclusions

For the purpose of the EU funded project (Hydrological Monitoring Equipment), a study catchment had to be selected to serve the needs of a hydrological monitoring programme geared to the understanding of a water resources system and the evaluation of management alternatives all within a complete training programme for the Palestinian technical community. To aid the process of catchment selection, both UNUT and PHG completed the following: a list of criteria for catchment selection was prepared; a survey of water resources management related data was undertaken in the UK and OPT; the GIS and simple graphical and tabulated manipulation and analysis of the data were carried out in order to apply the criteria for catchment selection; a short-list of potential candidates for a catchment study was prepared from the results of the previous step; field verification of the final catchment selection was undertaken jointly by UNUT and PHG; and finally the Zeimar catchment was selected.

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Session - 6
Water Distillation

Nocturnal Condensation Cooling A New Desalination Techniques in Remote Areas

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Nocturnal Condensation Cooling
A new Desalination Technique in Remote Areas

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Abstract

This paper represents a proposed method **nocturnal condensation cooling** for desalination in remote and coastal areas such as villages, fishing villages, resorts, and similar areas where small populations can make a living but where water is scarce. An experimental study was carried out and computer simulation programs were developed to calculate water collection under ideal conditions.

Both experimental and theoretical studies indicated that this process is useful for small scale water production in the absence of energy sources. It can also be used in conjunction with solar desalination process. The design of one unit (100 m²) shows an annual production of 7610 gallon with production cost of less than 6 cent a gallon.

Nomenclature

a'	3.2437814
b'	$5.86826*10^{-3}$
c'	$1.1702379*10^{-8}$
C_L	The cloud factor
C_{PF}	Specific heat of film, J/gm. K
C_{pw}	Specific heat of sea water, J/gm. K
c_s	Humid heat in J/gm. K
d'	$2.1878462*10^{-3}$
h_c	The heat transfer coefficient in J/sec. m² K, from sea water to air and from air to polymer film.
h_{cA}	The heat transfer coefficient from film to ambient air, J/sec m² K
P_c	218.167 atm

P_c	218.167 atm
P_s	Water vapor pressure over salt water in atm.
P°_{Wsw}	Saturated vapor pressure of pure water at sea water temperature in atm.
P°_{WT}	Saturated vapor pressure at T in atm.
P°_{WTF}	Saturated vapor pressure of water at the film temperature in atm.
T_{Air}	Air temperature, K
T_c	647.27 K
T_F	Film temperature of the plastic cover in K
T_{sky}	Effective sky temperature in K (assumed 0 K, Lazzarin, 1979)
T_{sw}	Sea water temperature in K
X	$T_c - T$
λ	Latent heat of vaporization of water in J/g
ε	Emissivity of the film
σ	Stefen-Boltzman Constant = $5.67*10^{-8}$ J/sec. $m^2 K^4$

Introduction

After air, fresh water is the most important material necessary for human life. However , there is a tendency in recent years for the world's population to work and live in areas where most other resources are available, but where local supplies of fresh water may be less than adequate. Many coastal areas such as saudi arabia have plenty of sea water but little fresh water. In these areas, desalination is one of the most important factors in economic development. Although a number of desalination methods are possible, the cost of producing the water is critical to a successful method of desalination. This is because the various methods must compete with each other and also against the cost of transporting water from the nearest source of fresh water. However, there are some locations where fresh water availability is important, even if the cost is high. This is usually in remote areas where modern transportation and utilities are not available.

Numerous processes have been proposed for the desalination of water, with some attaining commercial status. Multistage flash desalination, electrodialysis, and reverse osmosis are already applied in a variety of practical installations. Solvent extraction, ion exchange, freeze separation, solar desalination and hydrate separation have been less developed, and are limited to special cases. These processes are considered to be the best methods for large scale systems where electrical power and fuel systems are available.

In the absence or shortage of energy sources, people may still have a cerious promblem in obtaining fresh water. A solution for these remote water problems requires a suitable, economical and simple process of desalination. Such a process could make remote areas more suitable for small groups of people involved in fishing, mining, oil production, tourist resorts and dry land agriculture. While solar desalination is suitable in these situations, and has been studied and reported on in many journals, it has been applied primeraly on an experimental basis.

This research was carried out to investigate a method of desalination which has the possibility of supplying the needs of small groups with a minimum use of power, fuel, or transportation. It involves using noctural radiation cooling to condense water from humid air in coastal areas. Very little appears to have been done on such applications of noctural radiation cooling to desalination (Al-Amri, 1986 and Lazzarin, 1979).

Both experimental and theoretical studies were carried out on the practicality of using this method in desalination.

Proposed Nocturnal Condensation Cooling Process

Howe (1974) classified the desalination processes according to the phenomena involved.

(I) Those involving phase changes in water, as in distillation, freeze separation and hydrate separation.

(II) Those using surface properties of memberanes, as in electrodialysis and reverse osmosis.

(III) Those using ion-selective properties of solids and liquids as in ion exchange and solvent extraction.

Included in the phase change process are methods of solar desalination, and the method proposed here.

In order to avoid the use of conventional energy source for producing fresh water by desalination, it has been proposed that other source of energy such as solar energy be used. Many investigators have studied the direct application of solar energy to the production of fresh water (Anon, 1970, Brice, 1963 and McCarthy, 1979) , with most of the research and development directed towards the basin-type of solar distillation.

These units consist of a basin containing saline water at a shallow depth, a sloping transparent cover and a collection gutter along the bottom edge of the sloping cover. In this device solar radiation passes through the transparent cover and is adsorbed in the salt water basin, converted to heat and transferred to the water. The warmed water produces water vapor, and the humid air rises by free convection. It then comes into contact with the inner surface of the cover which is cooled by the surrounding air, and part of the vapor condenses. The formed condensate flows down the sloping surface of the cover and is collected in the gutters at the base of the slope, and is removed from the still. Large solar stills of this type are operating in Australia and in Spain, the largest capacity being about 7000 gallons per day (McCarthy, 1979). Solar evaporation occurs only during daylight hours, and a method of using these systems at night would increase their output and reduce costs. Solar evaporation has good possibilities for use in remote areas since the equipment is simple and the small energy requirments are only for pumping, and this is also required for most fresh water sources.

Reverse osmosis methods have been developed to supply water for small scale use, but such systems require a source of power and a technical personnel and supplies (Marten, 1977, and Friedlander, 1966).

Sources of energy for processes as well as engines result from heat sources at higher temperatures and heat removal at lower temperatures. Thus a heat supply at ambient temperatures with removal of heat at lower than ambient temperatures can also be used as a source of desalination energy. The lower temperatures needed can be produced by noctural radiation into clear skies.

This process is represented schematically in Fig. (1). It will be described as it might be used in conjunction with solar evaporation or distillation.

During the night a film as shown in Fig. (1) will radiate heat to the sky, particularly on clear nights which are common in desert areas. It will drop in temperature until the heat radiated is balanced by heat transfer from the ambient air, from the air under the covr, and the heat given off by condensation of moisture from below. The cover film should drop enough in temperature to condense moisture vapor produced by the warm sea water.

As water is evaporated and heat is transferred, the salt water temperature will drop, and less water will be evaporated compared to heat transfer, due to the vapor pressure-temperature relationship. The deeper the sea water basin, the longer the sea water will remain warm enough to provide evaporation.

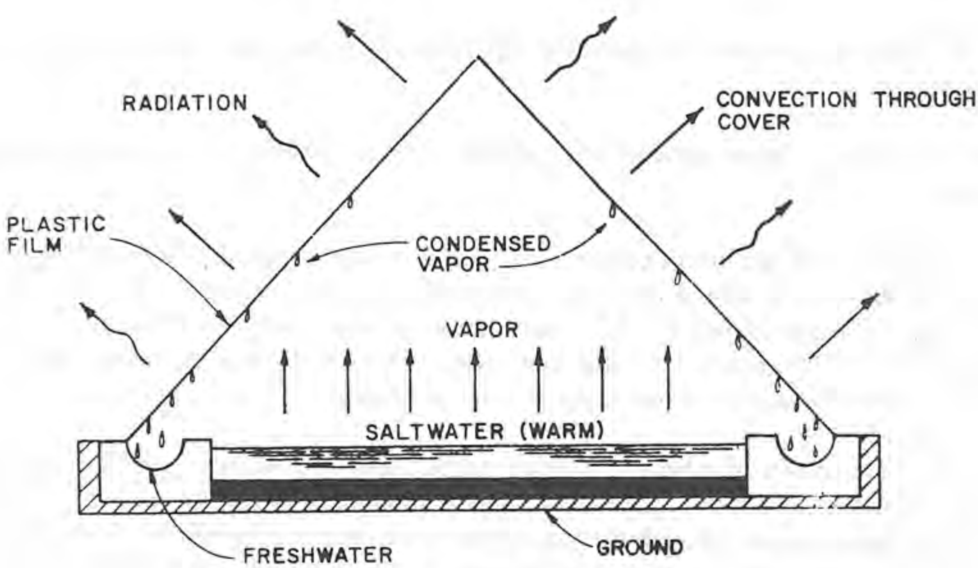

Fig. (1) Proposed system for desalination by radiation condensation cooling.

However, a source of warm sea water, such as from a shallow bay or a solar heated salt pond, or in some cases simply surface sea water, could be used at night to circulate through the salt water reservoir. Such sea water sources change in temperature very little due to their large volume and heat capacity. This would keep the salt water in the system at near the temperature of the warm sea water source, while the radiating film should be appreciably lower. This should maintain the rate of evaporation and condensation at a reasonable level during the night.

The film cover would be supported and sloped similar to a solar distillation system. The film cover should be opaque to infra-red for radiation at night, but transparent to visible light if the system is to be used both day and night. Also the bottom of the pool should have a black surface to adsorb heat during the day time for solar desalination. A drain is provided so that about have the feed brine can be drawn off to prevent high concentrations.

During the day time, heat from solar radiation can produce desalination as previously described.

It is seen from this description that the apparatus and process are extremely simple and inexpensive. There is no moving equipment other than water, pumps, there are no high or low pressures in the system and the system is substantially self-operating.

The disadvantages of this design are the relatively low production rates, and the compratively short life of plastic materials with the need for frequent replacement. The cover film in particular is subject to decomposition by ultra-violet radiation during the day time. If the system were operated during both day and night, a film transparent to ultra-violet and visible light and opaque to infra-red light should be used to transmit light energy in the day time from the sun, and to radiate infra-red heat at night from the film, and not allow directed radiation from the sea water to the sky.

If operated only at night, a film containing black pigment could be used, since no transmission of light is desired, and such films are much more resistant to the effects of ultra-violet light, and last much longer in the case of low cost plastic film such as polyethylene.

Other possibilities include the use of a limited amount of sea water during the night to give a desired temperature drop. The resulting low temperature salt water could then be used for air conditioning or other cooling purposes. Alternatively, waste heat from air conditioners, engines, and other source could be used to heat salt water for either daytime solar operation or nocturnal radiation operations.

There are number of possible combinations which could use nocturnal radiation for desalination, but this study concentrates only on investigating simple procedures to produce condensate by nocturnal radiation cooling. In addition it was desired to find the maximum flow of water that could be produced theoretically for comparison with the amount obtained in experiments.

Method and materials

A condenser was prepared using styrofoam ice chest, as shown in Fig. (2), by fixing a trough of aluminium (8.5 cm x 28 cm) under the low points of a sloped polyethylene film. One end was higher than the other to collect the condensed water from the polymer film and allow it to flow to a graduated cylinder. The drain water to the cylinder was connected to the low point of the trough. Five or six liters of water with 3.5% salt (by weight) were placed in the condenser at the desired temperatures. The desired covering for the styrofoam box, black polyethylene, was folded over the box and secured.

During clear nights temperatures of the salt water , and the surrounding air were recorded and the condensed water was measured.

Calculations

The computer calculations considered the principal energy and mass flows, and the energy and material balances around the system. The program was designed to find the following at any given set of conditions:

(a) The saturated vapor pressure of water at the sea water surface and at the film in atmospheres.

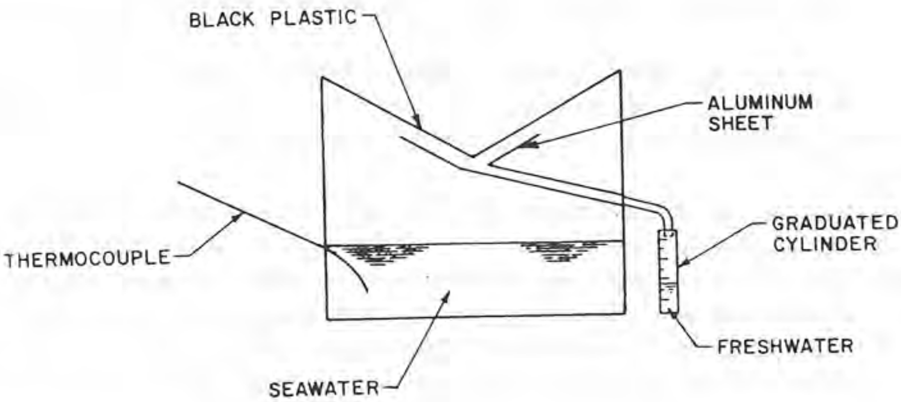

Fig. (2) Experimental condensation apparatus.

(b) The temperature of the cover film in Kelvin.
(c) The rate of fresh water collection per square meter.

At any time, on the basis of one square meter of horisontal area,

$$Q_{acc} = Q_{Csw} + Q_{mt} - Q_{rad} - Q_{cair} \tag{1}$$

Q_{acc} is the rate of accumulation of heat content of the film in Joules/sec.m^2.

Q_{acc} is zero for steady state collections and equal to total film heat capacity times dT_F/dt for dynamic calculations.

Q_{Csw} is the heat of convection from sea water through air to the film in Joules/sec.m^2

$$Q_{Csw} = h_c(T_{sw} - T_F)G \tag{2}$$
$$= U\,(T_{sw} - T_F) \tag{3}$$

where,

$$h_c = 1.24(\Delta T)^{0.33} \text{ J/sec.m}^2\text{ K} \tag{4}$$

In Equation (4), ΔT is the temperature difference from salt water to the film divided by two, or the ΔT from salt water to air or from air to film. This simplified equation is given by Hsieh (1986) for heat transfer from bulk air to horizontal cylinders. The heat transfer with moisture vapor is included in Q_{mt}, and effects of the low humidities on purely thermal heat transfer should be negligible. While a single modified heat transfer coefficient could be used between parallel surfaces, there are undoubtedly two heat transfer boundary layers involved, and non-parallel surfaces with different areas. Thus two coefficients, one at the polymer film and one at the water surface, were used. These were combined into an overall thermal heat transfer coefficient. U is the overall heat-transfer coefficient in J/sec.m^2.K, from sea water to the film.

$$U = h_c G \tag{5}$$

The geometric factor (G) was used to modify U based on the angle of the film and area available for convection heat transfer compared to horizontal area, and to allow for the two heat transfer films between the sea water and the polymer.

For example, the overall convection heat transfer coefficient times area from water to film for the 45° angle film system in Fig. (2) is found from,

$$\frac{1}{AU}=\frac{1}{Ah_c}+\frac{1}{1.414Ah_c} \tag{6}$$

where A is the horizontal area of heat transfer, 1 m², and the area of the film = $(2)^{1/2}$; $A = 1.414$. Thus,

$$UA=\frac{h_c}{1.707}=h_cG=U \text{ or } G=\frac{1}{1.707} \tag{7}$$

Values of G were 1/2 for parallel horizontal surfaces due to the two heat transfer films between the salt water and polymer. Thus, the use of G allows calculations to be based on horizontal areas rather than actual polymer film area.

Q_{mt} is the heat transferred to the film by mass transfer and condensation.

$$Q_{\mathrm{mt}}=\lambda K'\left[P_s-P^{\circ}_{\mathrm{WTF}}\right]\frac{18}{29}G \tag{8}$$

where,
$(18/29)K'$ = mass-transfer coefficient in g/sec.m², mol.fract., from sea water to air and from air to the film.

The geometric factor was also used to modify Q_{mt} for condensation on angled surfaces for reasons similar to those for convection heat transfer. Experimentally, it has been shown by Perry and Chilton (1973) that for air-water systems the value of $h_c/K'c_s = 1$.

Therefore,

$$K'=\frac{h_c}{c_c} \tag{9}$$

$$P_s = 0.978(P^{\circ}\mathrm{wsw}) \tag{10}$$

Note: At 1 atm. total pressure, P_s and P°_{WTF} = mol. fractions of H_2O in air at the salt water surface and the polymer film surface.

Q_{rad} is the heat of radiation to the atmosphere in Joules/sec.m². This 's based on the horizontal areas exposed to the sky and is assumed to be distributed over the total corresponding polymer film area.

$$Q_{\mathrm{rad}} = \sigma(T^4_{\mathrm{F}} - T^4_{\mathrm{sky}})(1 - C_L)\varepsilon \tag{11}$$

$$Q_{\mathrm{CAir}} = h_{\mathrm{cA}}(T_{\mathrm{F}} - T_{\mathrm{Air}})G \tag{12}$$

Q_{CAir} = the heat convection to the atmosphere in J/sec. m².

To calculate $P°_{WTF}$, and $P°_{Wsw}$, saturated vapor pressures of water at the film and salt water temperatures, the following equation by Keenan and Keyes (1936) was used.

$$\mathrm{Log}_{10}(P_c / P°_{WT}) = \frac{X}{T}\left[\frac{a' + b'x + c'x^3}{1 + d'x}\right] \tag{13}$$

The computer program for steady-state calculations is a trial and error program to find a film temperature at which radiation is balanced by convection heat transfer from both sides of the film and the heat of condensation from mass transfer. At the steady-state temperature, the rate of water collection W_{mt}, was obtained from Q_{mt}/λ. Steady-state conditions would exist if salt water temperature were maintained constant by pumping or by a large source of sea water. Such calculations could also approximate rates at average conditions.

A modification of this program calculated rates of condensation from ambient humid air only to a film radiating heat from its surface. Similar trial and error methods to determine film temperature were used.

For a dynamic simulation, a program was written to calculate condensation rates and rates of temperature change at any given conditions for the system shown in Fig. (2). The dynamic simulation describes the condensation system with time. The condensate accumulated, salt water temperature, and film temperature are calculated over small increments of time starting from the initial conditions. These calculations determined the change in temperature of the salt water and film and the amount of condensate collected with time for up to 12 hours. Input data were initial conditions, low air temperature, time of low air temperature, and final time and air temperature. Other inputs were:

D_{Time} = increment of time in seconds (usually 30).
W_{Film} = weight of the cover film in g/m² of the film.
F_{coll} = water that remains on the bottom side of the film after draining a surplus in g/m² of the film (a constant).
S_w = the amount of sea water used in the experiment in g/m² of horizontal area.

A basis of 1m² horizontal area was used in all calculations . A "Do Loop" was written to take conditions at any given time and to find the required data for the following increment of time. The entire calculations of the "Do Loop" were made according to the transfer equations given in equations (1) to (13).

The water accumulated on the film before draining increases by ΔF_w, with

$\Delta F_w = W_{mt} = Q_{mt} D_{Time}/\lambda$ from equation (8). (14)

If F_w is greater than the maximum after draining, (F_{coll}) then water collected, (ΔW) is $F_w - F_{coll}$, and F_w is set back to F_{coll}.

The sea water temperature will be decreased because of heat transferred to the film by convection, mass transfer and any heat losses, and the new sea water temperature can be determined by the following equation:

$$\Delta T_{\text{sw}} = -(Q_{\text{csw}} + Q_{\text{mt}} + Q_{\text{s}}) D_{\text{Time}} / (S_{\text{w}} . C_{pw}) \tag{15}$$

The heat lost through the sides and bottom (Q_s) could be determined as outlined by Cooper (1969) in modelling solar evaporation. However, in these experiments the salt water was contained in two foamed polystyrene boxes, and in addition the temperatures of the salt water were near ambient (as they would be in practice). For these reasons, Q_s was assumed to be 0. Estimates based on the thermal conductivity of the boxes indicated that at the highest temperature used the error might be as high as 100g of water collected per m² in 8 hours. Sources of heated sea water were considered by Malik and Tran (1973) in a similar simulation. With such sources (above 49°C), losses should be more significant.

The temperature of the film will change due to mass and heat transfer,

$$\Delta T_{\text{F}} = +\frac{(Q_{\text{csw}} + Q_{\text{cAir}} + Q_{\text{mt}} - Q_{\text{rad}})}{(W_{\text{Film}} . Cp_{\text{F}} + F_{\text{w}} . Cp_{\text{w}})} D_{\text{Time}} \tag{16}$$

The change in the amount of salt water at the end of the time increment is found by

$$\Delta S_{\text{w}} = -W_{\text{mt}}, \tag{17}$$

because the sea water decreases by the amount of water vaporized to the film. It was assumed that the amount of salt water was sufficient to make concentration changes negligible.

The temperature of the air falls from the starting temperature (T_{AirST}) until the lowest air temperature ($T_{\text{Air}L}$) is reached at time L. It then goes slowly higher to the final temperature T_{AirE} at time E. Assuming air temperatures are linear with time, the following two equations provide the new air temperature (T_{Air}).

If the time is less than the lowest temperature time then,

$$T_{\text{Air}} = T_{\text{AirST}} - (T_{\text{AirST}} - T_{\text{Air}L}).\ \text{Time}/\text{Time}_L \tag{18}$$

If the time is more than the lowest temperature time then,

$$T_{\text{Air}} = T_{\text{AirL}} + \frac{(T_{\text{Air}E} - T_{\text{Air}L}).(\text{Time} - \text{Time}_L)}{(\text{Time}_E - \text{Time}_L)} \tag{19}$$

With new values for Time, T_{F}, T_{sw}, F_{w}, W, S_{w} and T_{Air}, the program will calculate the next increment and continue to the final time desired. The final time should be no longer than the time of darkness in one night.

Results and Discussion

Some tests were made to determine the heat transfer by radiation from styrofoam trays containing 500 ml of water and covered by polymer films as well as a combination polymer and aluminium foil film. These tests were intended to determine the difference between convection heat transfer with aluminium foil and convection and radiation from the polymer film systems. The results were inconclusive due to the low rates of heat transfer being difficult to measure accurately, and also the inability to distinguish between radiation from the water through the polymer films which would not cause condensation and radiation from the polymer film which should lead to condensation. In general, the lowest heat loss was from the aluminium foil, the next lowest from black polyethylene, and higher heat loss from clear polyethylene and a Tedler (polyvinyl fluoride) film. This seams to indicate that clear polyethylene and a Tedler are partially transparent to infra-red and allow some radiation from the water through the film. Since it is necessary for radiation to occur from the film to produce condensation, black polyethylene was used in experiments on condensation. However, further study should be made on other polymers that would be clear to visible light, but opaque and good emitters for infra-red heat radiation. Some preliminary results indicated that heat losses from the styrofoam containers was approximately 1/4 to 1/3 of the heat radiated from the polymer covers, but these results were inconsistent.

Table (I). Comparison of experimental and calculated results of condensation over salt water(3.5%)

Exp. No.	Tsw_{ST} (C°)	Tsw_E (C°)	Mean TSW (C°)	Water collected experim. (g/m²)	Calculated** st. state water collected 8 Hr (g/m²)	Dynamic calculation water collected 8 Hr (g/m²)	Dynamic calculation Tsw_F (C°)
1	29	24.0	26.5	428.10	829.77	549.50	20.00
1*	29	24.0	26.5	428.10	819.87	547.94	20.10*
2	30	25.2	27.6	436.76	885.23	594.46	20.50
3	31	25.6	28.3	421.62	922.10	641.82	20.70
4	32	26.5	29.3	454.10	980.10	638.68	21.65
5	33	27.0	30.0	432.43	1023.17	702.90	22.40
6	34	27.5	30.8	497.30	1067.85	682.10	24.00
7	35	27.0	31.0	627.03	1076.66	724.10	24.62
8	36	27.5	31.8	614.05	1123.21	734.20	25.50
9	37	25.0	31.0	540.54	1076.66	904.43	24.62
10	38	24.0	31.0	596.76	1076.66	845.97	26.44
11	39	25.0	32.0	607.73	1145.97	804.69	28.20
12	40	28.0	34.0	609.73	1273.10	1027.42	27.10
13	41	26.0	33.5	670.27	1245.90	1063.10	27.22
14	42	25.0	33.5	745.95	1245.90	1205.29	27.66
15	43	26.0	34.5	791.35	1316.44	1267.28	27.74
15*	43	26.0	34.5	791.35	1286.10*	1265.60*	28.10*

*Calculated with emissivity = 0.95 for black polyethylene.
**Based on mean experimental temperature.

The results on evaporation and condensation are summerized in Tables (I,II,and III) which show that this process can produce water on a small scale. The theoretical calculations, based on ideal conditions of emissivity, heat and mass transfer, and atmospheric conditions, indicate more production than the experimental work. This should be expected due to one or more of the following factors:

(1) The nights during the period of experimental work were not fully clear, and even when clear, the sky often had a slight haze. This affected the heat radiated to the sky and thus the surface temperatures and condensation.

(2) Wind will increase the heat transfer coefficient on the top of the condensation surface (see Fig. 2) without increasing the heat and water vapor transfer within the condenser to the condensing surface.

(3) The production rates were probably reduced by small heat losses, that could not be adequately measured, through the container from the salt water. Since relatively small quantities of heat were involved in the experiments, small losses could be significant.

Table (II). Rates of condensation from air at a given partial pressure of water vapor

Exp. No.	Temp air (K)	Partial Pressure H_2O (atm.)	Geometry factor G	Experiment collection (ml/m²)	Water Calculated 8 Hr (ml/m²)
1	296.0	0.0254	1.0	68.0	1278.6
2	294.3	0.0200	1.0	37.0	1011.6
3	290.9	0.0169	1.0	150.0	861.2
4	291.5	0.0162	1.0	135.0	824.1
5	294.3	0.0168	1.0	70.0	848.5
6	294.8	0.0187	1.0	118.0	944.0
7	295.4	0.0206	1.0	76.0	1038.9
8	298.2	0.0234	1.0	59.0	1171.1
9	297.0	0.0205	1.0	35.0	1029.1
10	296.0	0.0219	1.0	54.0	1102.8
		(All of the above collected on metal car top)			
11	284.3	0.0130	2.0	126.0	820.2
		(60° Angle, Black Polyethylene)			
12	284.3	0.0130	1.0	110.0	670.7
		(flat, Black PE)			
13	284.3	0.0130	1.0	103.0	670.7
		(metal, car)			
14	289.0	0.0174	2.0	76.3	1079.1
		(60° Angle, Black PE)			
15	289.0	0.0174	1.0	83.9	891.0
		(flat, Black PE)			
16	289.0	0.0174	1.0	83.3	891.0
		(metal, car)			

Table(III). Monthly means for salt water and air temperatures provided from a desalination plant at Yanbu, Saudi-Arabia (Shucker,1993) and dynamic calculation of water collection over salt water

Month	Mean sea water temp (°C)	Mean air temp start (°C)	Mean time at lowest air temp (hr)	Lowest air temp (°C)	End air temp (°C)	Calculated water collected (ml/m²)
June, 1992	29.4	28.00	9.98	23.86	25.66	674.51
July, 1992	30.4	30.75	10.63	24.80	26.45	718.61
Dec.,1992	27.8	25.45	10.44	19.89	20.74	611.79
Jan., 1993	26.6	23.70	10.71	17.40	18.00	563.23

All calculations based on 12 hr. and 33 cm depth of sea water.

When experimental results are plotted against theoretical calculations in Fig. (3 and 4), they indicate that experimental results are approximately linear with theoretical at steady-state, and under dynamic conditions. Fig. (5) shows maximum production as a function of temperature. This indicates that higher temperature salt water gives much better results. This corresponds to the results of Malik and Tran (1973) who studied heated sources of sea water (above 49°C) to supply energy for nocturnal desalination using solar evaporators. Their study involved larger driving forces, heat transfer rates, and mass transfer rates, and for this reason, more reliable measurements and better correlation of results.

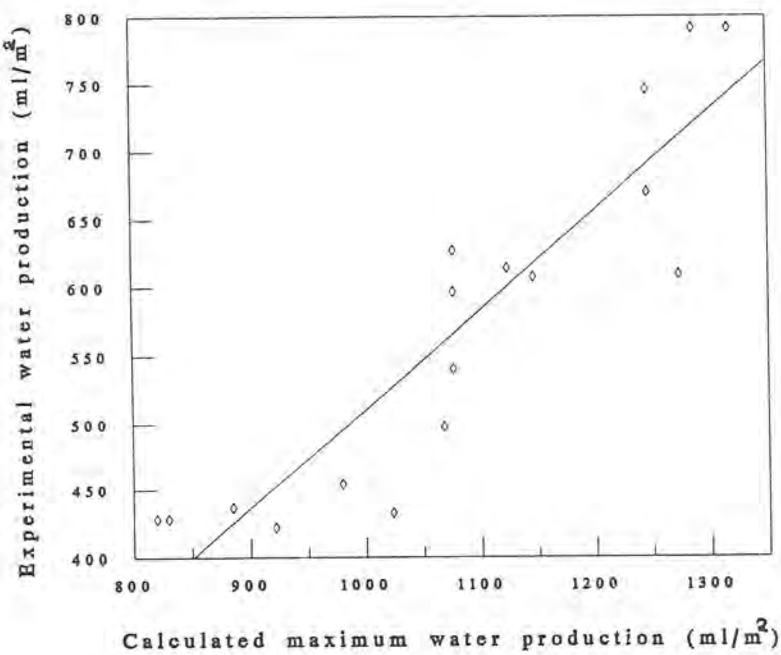

Fig. (3) Comparison of experimental and calculated results at steady-state for water condensation on a film supported over sea water (8 hr)

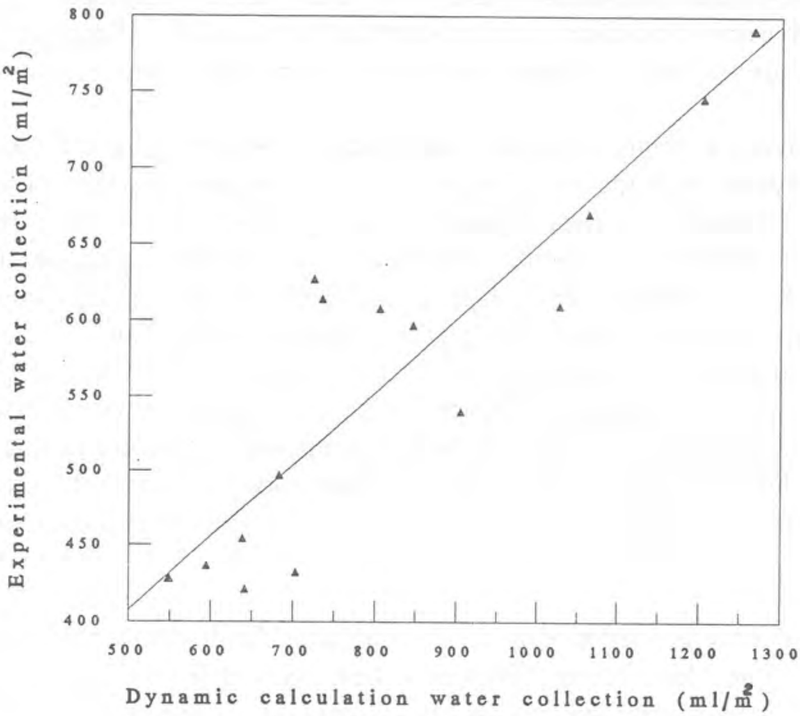

Fig. (4) Experimental vs. calculated water collection using dynamic simulation for water condensation on a film supported over sea water (8 hr).

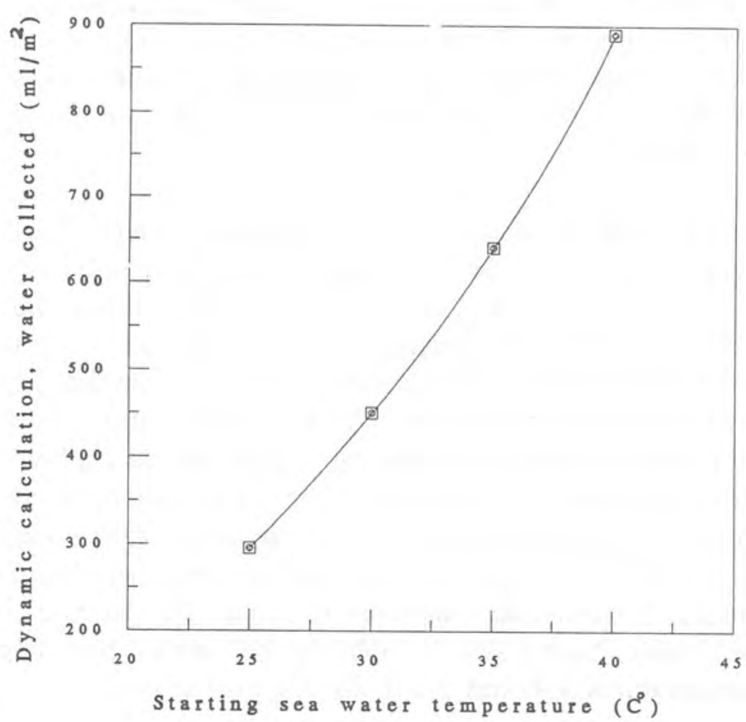

Fig. (5) Calculated water collection vs. starting sea water temperature for water condensation on a film supported over 33 cm of sea water (8 hr).

Table (I) shows that the results of calculation under steady-state conditions are higher than those from a dynamic simulation at the same mean temperature. The dynamic results are probably lower because of very nonlinear rates as a function of salt water temperatures.

Table (II) shows a simple example of the effects of the atmospheric infra-red radiation where dew was collected from the top of a car parked in the open on clear nights. As a rule, after some hours of darkness on clear nights, the car top falls below the dew point, and the moisture in the air begins to condense. Table (II) also includes data obtained as dew collection on black polyethylene film, which is equivalent to the car top collection on the same night. This type of water collection is possible whenever humid air and clear night time skies exist simultaneously. The calculation for this situation based on steady-state and constant humidity, shows a higher difference between computer results and experimental results. The differences between theoretical and experimental are probably due to the same reasons as in condensation over salt water, and even more because of variations in the humidity during the night and the lack of source of moisture other than the air to maintain air humidity.

This kind of dew formation can occur over large land areas in a humid but clear environment as in some coastal areas. However, heat transfer from the ground, which will have been heated by the sun during the day, could completely prevent or reduce such dew formation. Surfaces insulated from the ground such as shrubs or grass near the sea could collect appreciably more dew than sand or bare ground. In two tests, dew collection was measured on both sand and sand covered with grass. The results in one case were 50 ml/m^2 on sand and 121 ml/m^2 on grass and sand. Temperatures at the sand surface were about 22°C and in the grass were about 21°C. In another case 43 ml/m^2 were collected on sand, 123 ml/m^2 were collected on a car top, and 173 ml/m^2 were collected on sand covered with grass during the same time period. This indicates that some vegetation might survive on dew near desert coast lines, provided that it gets a start and begins to collect dew, and that it is not completely consumed by animals.

Table (III) gives the result of calculations by the dynamic simulation of condensation over salt water using weather and sea water data taken from an existing desalination plant at Yanbu-Saudi Arabia (Shucker, 1993). This shows water can be produced using the proposed process, with more production during the summer months when salt water temperature is higher than in winter. The results show a considerable increase in condensation with higher temperature salt water which is also shown in Fig. (5). In any application of this method, the source of sea water should have as high a temperature as possible. In shallow bays, and salt water ponds temperatures are genarally appreciably higher than in the open sea. Therefore such warmer waters should be used in this method where possible. One possible method of using nocturnal radiation would be to construct a condensing polymer film directly over a shallow bay or inlet, and pipe the collected condensate to shore. This method could eliminate the need for pumping sea water. Such a system could also be used for solar desalination, but would produce lower temperatures and rates due to dispersion of solar energy.

Costs

The costs were calculated for the desalination system shown in Fig. (6). Assume the area for each unit is 100 m^2, and the average production per night = 800 ml/m^2 due to almost constant sea water temperature.

Therefore the production = 7610 gallon/year, and the total operating costs were found to be \$450.00 per year.
Thus unit production cost = \$450.00/7610 = \$0.059/gallon

This cost is too high for use of this method in cities or where power is available for reverse osmosis. However, this is not too much to pay for drinking water in isolated areas or other areas with limited supplies, power, and facilities. In addition, a proper design could add approximately 1 l/day per square meter or about 25% (Sayigh,1977) to solar distillation at almost no extra cost. The economics of such a system would depend primarily on the solar economics.

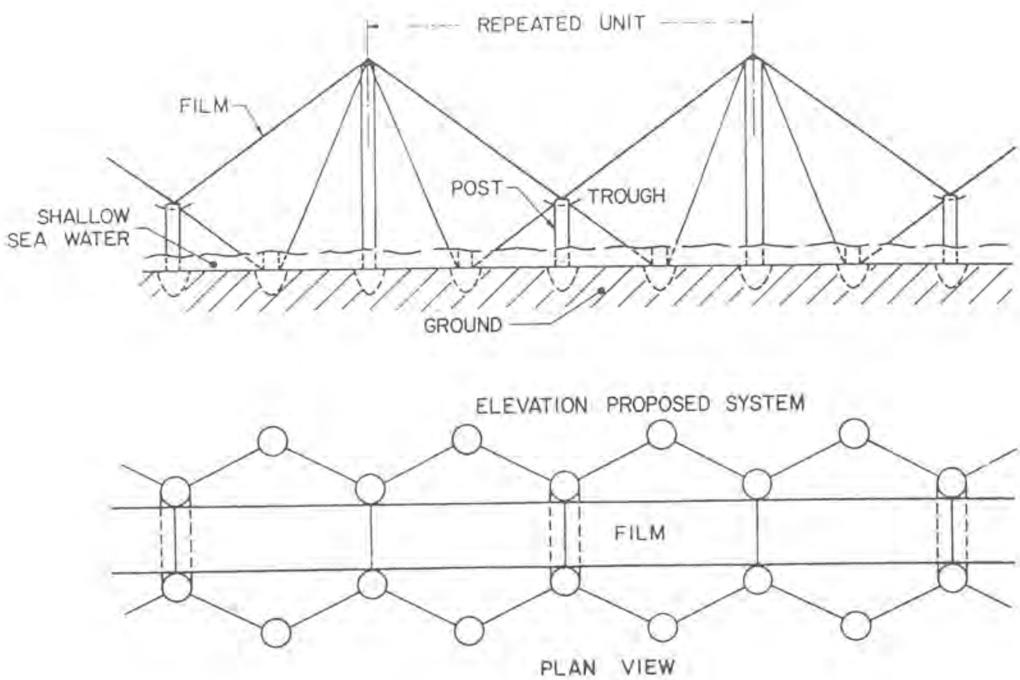

Fig. (6) Design of the proposed method

Conclusions

The study of desalination using nocturnal condensation cooling has led to the following conclusions:

(1) This process is useful only for production of fresh water where relatively high costs can be justified.

(2) This process is based on a simple design, and therefore it requires a low level of support, utilities and transportation.

(3) It is recommended that future study on this method consider the effect of emissivity and surface characteristics of various polymer films. Because the combination of daytime solar distillation with nocturnal radiation desalination appears to be possible with same apparatus, this combination should be investigated.

(4) Higher temperature sea water appears to be the most desirable, and simple methods of obtaining a warmer sea water supply should be given further consideration.

(5) Further investigation should be made on the possible growth of plants near the ocean where natural dew collection on vegetation is greater than that on sand or bare ground.

(6) Although the cost per 1000 gallons is higher than other methods of desalination, the simplicity of nocturnal condensation desalination or dew collection could make it a practical and reasonabl method for remote locations along coastal areas with clear skies.

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DESIGN FEATURES OF SHOAIBA PHASE-II
500 MW-100 MIGD POWER AND DESALINATION PLANT

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ABSTRACT

Shoaiba phase two is an extension to Shoaiba phase one (Makkah-Taif) [1] and presently under construction , which is located 75 Km on the coastal line south of Jeddah. It is expected that the plant shall be commissioned mid 1997. The contract was awarded to a constrium from Bechtel and Hanjung under the consultancy of Ewbank Preece Power & Water Limited. It is expected to be the largest project of it's kind in this decade on the West Coast of Saudi Arabia.

The plant is designed to produce 500 MW power and 100 MIGD potable water. The plant shall consist of five dual purpose units, each unit comprises of a boiler, back pressure turbine and two super large multistage flash evaporator (MSF) units. The capacity of each MSF unit would be 45,500 m3/d i.e. 10 MIGD, which considered second largest MSF unit ever built [4]. The generator connected to each turbine shall be able to generate 100 MW. The power generated from Shoaiba phase two would be connected to the Western Region grid. The potable water produced from the plant shall be distributed to the three major big cities of the western region of Saudi Arabia, namely Jeddah (60 MIGD), Makkah and Taif (40 MIGD). The water for Makkah and Taif shall be pumped through already existing pipeline, while the water produced for Jeddah shall be pumped through a new pipeline. The distillate produced from MSF units is destituted from alkaline minerals, hence it is very corrosive. A complete post treatment unit which is better design from phase one is included in the plant layout for treating the distillate to render the final product water passivated i.e. having a tendency to deposit a thin protective film of calcium carbonate, to protect the pipeline and improve water platability.

GENERAL DESCRIPTION

1. POWER PLANT

Each boiler is designed to be able to produce enough steam to generate 100 MW power and the exhausting steam, from back pressure turbine would be enough to supply two brine heaters by the required heat for the production of 20 MIGD. An excess amount of steam would be available in each boiler to supply steam through a reducing station to 2.5 bar low pressure header to a third brine heater when one boiler is out for maintenance.

The plant is designed to produce designated water capacity (100 MIGD) with only four boilers in service.

For maximum operational flexibility the five power units are connected from high pressure steam side,as well as from low pressure steam side.

From Shoaiba phase one operating experience, it was felt that dump condenser is not needed for phase two.

2. EVAPORATOR DESIGN

Ten evaporators multistage flash type are cross tube configuration, brine recirculating type and single deck. The design capacity of each unit is 45,500 m3/day i.e. 1896 m3/hr based on sea water summer temperature 33 C. It was concluded from Shoaiba phase one operating experience with high temperature additive, that the optimum top brine temperature is 110 C single mode operation. The evaporator design is still flexible enough to operate the evaporator at reduced output whenever it is required by plant operation without the need for evaporator shut down for flalshing brine gates adjustment. Evaporator performance ratio 9 kg/2326 kj was choosen for economical reason. The evaporator total number of stages is 21 (18 stages heat recovery + 3 stages heat rejection).

One of the Shoaiba phase two evaporator design feature is that the brine recycle flow is maintained constant under any operating condition. Maintaining high brine recycle velocity at 2 m/s in the tube help to great extent, internal surface of the tubes in clean condition, production control could be done by reducing top brine temperature. Summary of the evaporator basic design parameters is shown in table 1.

TABLE (1)

Daily distillate production	45,500 m3/D
Performance ratio	9 kg/2326 kj
Total number of stages	21
Heat recovery stages	18
Heat reject stages	3
Sea water temperature	33 C
Top brine temperature	110 C
Blow down temperature	40 C
Sea water salinity	40.5 g/kg
Brine recycle salinity	63 g/kg
Blow down salinity	70 g/kg
Make up flow	4425 T/hr
Blow down	2529 T/hr
Steam flow	221.6 T/hr

3. EVAPORATOR MATERIAL OF CONSTRUCTION

a. The evaporator is of rectangular shape, material of construction is carbon steel lined with stainless steel (evaporator floor lined with 317 L, side walls with 316L and top with 304 L). All evaporator internals e.g. flashing brine gates, distillate trough, division walls, demisters and demisters support are made of stainless steel 316L. The entire stainless steel surface exposed to flashing brine must be passivated to improve pitting corrosion.

b. Tubes for brine heater are made of modified 70/30 Copper-Nickel (with 2% Fe & 2% Mn). SWCC operating experience consider this choice is the best to avoid brine heater condensate dumping due to high Copper content.

c. Brine heater tube plates and waterboxes are carbon steel lined with 90/10 Copper Nickel.

d. Tube material for the first four stages of heat recovery are made of 66/30/2/2 copper nickel similar to brine heater. The reason for this selection is based on superior corrosion resistance of this material to carbon dioxide attack in the high temperature stages. Tube material for the rest of heat recovery stages is 90/10 copper nickel. Tube plates and water boxes in all heat recovery stages are made of carbon steel lined with 90/10 copper nickel. As a part of integrated additive treatment system on line mechanical cleaning system is incorporated for brine heater or both brine heater and heat recovery.

e. Operating experience with 60/30/2/2 copper nickel tubes for heat rejection in Shoaiba phase one proved to be an excellent choice. Therefore same selection of material was choosen for Shoaiba phase two. Tube plates and water boxes for heat rejection section are made of carbon steel lined with 90/10 copper nickel.

4. Corrosion control in MSF Evaporator.

SWCC has adopted the following measures to control corrosion in the MSF evaporators

a. Install sacrificial anodes in the water boxes in heat rejection, heat recovery and brine heaters for controlling tube inlet, exit and tube plates corrosion.

b. The evaporator is equipped with an external deaerator, rectangular shape. Dissolved oxygen in the deaerator effluent is 20 ppb. To achieve this figure deaerator liquid loading shall not exceed 20 kg/sm2. Make up spray type distribution system is recommended. The deaerator sump volume should be sized for one minute operation of make up flow.

c. Sodium sulphite injection system to brine recycle suction line is incorporated to guarantee low dissolved oxygen in brine recycle. Recommended dosing rate of sodium sulphite is four times the concentration of the dissolved oxygen in the deaerator effluent.

5. DESIGN PARAMETERS FOR SIZING THE EVAPORATOR

Parameters dipicted in table 2 are the basic for sizing the heat transfer area for brine heater, heat recovery and heat rejection as well as the evaporator dimensions.

SWCC operating experience with large MSF evaporator operated with high temperature additive concluded that the demister height shall not be less than 2.5 m. Distillate trough height from stage floor must be not less than 1.5 m. Therefore for Shoaiba 2 stage height is 4.2 m. Stage length tends to vary from one stage to the next taking in consideration the area required for demisters in each stage. It can be descerned from table 2 that flashing brine loading per unit stage width is 957 T/hr-m. The normal flashing brine loading per stage in other SWCC large size, recirculating brine MSF evaporator is in the range of 700 to 900 T/hr-m.

6. CHEMICAL PREPARATION AND DOSING SYSTEM

Operating experience gained from Shoaiba 1, concluded that for large number of evapaorators it is recommended based on economical reasons and greater operational flexibility to have central preparation and dosing system for each of the following chemicals:

TABLE 2

DESIGN PARAMETERS FOR BRINE HEATER, HEAT RECOVERY & HEAT REJECTION

ITEM	BRINE HEATER	HEAT RECOVERY	HEAT REJECTION
Brine (CW) velocity m/s	2	2	2
Tube ID mm	28	28	34
Tube OD mm	30	30	36
Thermal Cond. w/m2k	25	25-48	25
Fouling factor m2k/w	$30x10^{-5}$	$20x10^{-5}$	$20x10^{-5}$
Overall heat transfer coef w/m2k	2024	2528	2197
Tube length m	18.1	17.89	17.65
No. of Tubes	3860	3775	2722
BR Flow (CW) T/hr	17,120	17,120	18,200
Tube bundle Dia m	2.8-3.4	2.8-3.4	2.8-3.4

a. Antifoam
b. Anti scalant
c. Sodium sulphite

The evaporator is designed to be operated with single mode. Therefore closed type single header dosing system was chosen. This tends to ensure uniform chemical concentration and dosing. For economical reasons and achieving maximum operating flexibility the evaporator is designed with single blow down pump. To obtain highest availability from the evaporator an emergency blow down valve is included in the design, obviating total unit shut down in case any blown down pump is taken out of service for any reason. Therefore an additional injection points for antifoam, anti scalant and sodium sulphite after emergency blow down valve is implemented.

7. VACUUM SYSTEM

Vacuum system consists of single hogging ejector for start up capable of evacuating the evaporator in two hours and for normal service three stage twin ejectors and four barometric condensers. Material of construction for barometric condensers and drain legs is GRP. Vacuum system design is based on 150% of the calculated quantity of air inleakage and carbon dioxide gas liberated in flashing stages of the evaporator.

MSF POST TREATMENT

The distillate produced from a MSF plant is destitued from alkaline elements, hence it is very corrosive to the piping and not platable. Normally, the pH of the distillate from the additive treated plant is 6.5 to 6.0 due to presence of carbon dioxide. Most of the distillate produced in SWCC plants has to be pumped through long distance pipelines. These pipelines are steel with cement lining. Pumping distillate without passivation through these pipes would cause serious damage to the cement lining and hence to the steel which eventually would fail. The cost of the pipeline is several order of magnitude the cost of the entire dual purpose plant. In order to obviate pipeline corrosion and make distillate drinkable, passivation of the distillate is a must. Distillate passivation requires that the following conditions must be fulfilled:

i. The passivated distillate should be oversaturated with respect to $CaCO_3$. The oversaturation should be in the range of 4-10 mg $CaCO_3$/l. That is, the theoretical $CaCO_3$ precipitation potential should be 4-10 mg $CaCO_3$/l corresponding to a Langlier's saturation index up to about + 0.3.

ii. Calcium and alkalinity values should be at least 50 mg/l as $CaCO_3$ in order to prevent leaching from the cement lining. These values should be present in approximately stoichiometric concentrations.

iii. The ratio of alkalinity to the sum of chloride and sulphate should be at least 5:1 in which all concentrations are expressed as mg/l $CaCO_3$.

iv. pH should be in the range of 8.3 to 8.5.

In addition proper flow rate design is important to prevent corrosion damage because of mechanical stress and erosion effects. Overdosage of chlorine for disinfection should be avoided as high chlorine concentration will enhance metallic corrosion of e.g. mild steel and copper.

In order to satisfy the above mentioned requirements to passivate the distillate from a MSF plant, the following steps are performed:

a. Aeration
b. Carbonation
c. Disinfection

Distillate from the MSF plant is destituted from oxygen since the distillation plant is working under vacuum. Addition of compressed air to the distillate tends to saturate the distillate with oxygen to improve its taste. The maximum total dissolved solids content does not exceed 25 mg/l in distillate from MSF plants.

To satisfy the second requirement to render distillate not corrosive, calcium shall be added to the distillate. Injecting carbon dioxide to distillate will cause a drop in the pH value. The distillate is then passing through a limestone filter, which tends to dissolve calcium in the form of calcium bicarbonate as follows:-

$$CaCO3 + CO2 + H2O \longrightarrow Ca(HCO3)2$$

The post treatment is completed with addition of chlorine or calcium hypo chlorite as safeguard against any possible bacteriological contamination. It is obvious that the post treatment for distillate from a MSF plant is primarily a matter of a compromise between corrosion control in the pipeline and the cost the post treatment.

SWCC specification for distillate passivation calls for the following:

TDS	-	$<$ 110 mg/l
Ca as CaCO3	-	50 mg/l
Na + K	-	12 mg/l as Na
SO4	-	2 mg/l
Cl	-	$<$ 18 mg/l
HCO3 as CaCO3	-	50 mg/l
Total Alkalinity	-	1 mequ./l
Total Hardness	-	1 mequ. /l
pH	-	8.3-8.5
O2	-	5 mg/l
Available Chlorine	-	0.5 mg/l

CONCLUSION:

SWCC operating experience for more than 25 years is implemented in Shoaiba phase 2 specifications attempting to achieve lowest attainable water cost and highest availability for 20 years expected life time from the evaporators.

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A Permanent System for The Protection Of Desalination Plant Seawater Intakes from Oil Pollution

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A PERMANENT SYSTEM FOR THE PROTECTION OF DESALINATION PLANT SEAWATER INTAKES FROM OIL POLLUTION

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DUBAI ALUMINIUM COMPANY LIMITED.

ABSTRACT :

Dubai is the second largest city in the United Arab Emirates with a population of 500,000 and relies almost totally on Multistage Flash Distillation (MSF) plants for its supply of potable water. The city has a daily consumption of 80 million gallons, and this demand is growing steadily.

Dubai is situated on the southern coast of the Arabian Gulf and is a major port for one of the world's busiest shipping ways. Its desalination plants are located along a single 10 Kilometre stretch of coast line, and as a result of extensive offshore oil production and continuous traffic of oil tankers, the area is exposed to a constant threat from oil pollution caused by collision or grounding of oil tankers and oil spillage.

In response to the massive pollution which occurred during the Gulf conflict and the reasons described above the Dubai Aluminium Company Limited (DUBAL) which used to supply 40 % of Dubai's potable water, decided early in 1991 to install a permanent and effective barrier to protect its seawater intakes from the threat of oil pollution.

The objective of this paper is:

a) To present in detail the numerous factors which led DUBAL to opt for the construction of a permanent breakwater and boom system as the most effective method of protecting its seawater intakes.

b) To examine various types of constructions and profile of breakwaters proposed.

c) To examine the effects construction of the breakwater had on the operation of DUBAL's desalination plant (i.e. chemical consumption, turbidity, etc.).

INTRODUCTION :

Dubai Aluminium company limited (DUBAL) plant lies approximately five Kilometres north of the port of Jebel Ali in Dubai. DUBAL's main production is aluminium metal. As aluminium smelter processing consumes high energy, an adjacent power generation plant was built to supply constant electrical power to the smelter. DUBAL owns several gas turbines and the waste heat from the gas turbines is utilized in the waste heat boilers for the production of dry saturated steam. This steam is used in the back pressure steam turbines to generate electricity (72 Mega-watt). The exhaust steam is then used in the evaporators brine heater (Heat Input Section) to produce fresh water to Dubai. The desalination plant of DUBAL consists of two back pressure steam turbines and six Multi-Stage Flash (MSF) evaporators. Therefore, DUBAL has a dual role as an aluminium smelter and desalination plant.

The plant supplies water to the city of Dubai in conjunction with the other desalination plant in the Emirate (Dubai Electricity and Water Authority - DEWA).

The intakes are located approximately 405 metre into the sea of about 5.5 metre of water below datum. The level of the intake is approximately 1.0 metre above the sea bed. The total capacity through the intakes (all three lines) varies from 40,000 tonnes per hour (normal) to a peak capacity of 57,000 tonnes per hour. DUBAL's six evaporators are capable of producing 30 million gallon of drinking water per day. In 1991 this represented about 40 % of Dubai total drinking water consumption.

BACKGROUND :

On the 25th of August 1987 a Panamanian coastal tanker (AKARI) carrying heavy fuel oil lost electrical power supply and the use of main engines. The vessel was listing badly, and was towed to the east of the port of Jebel Ali with tug assistance. Around 1000 tonnes of heavy fuel oil escaped. It was estimated that 25-30 kilometres of the coast were polluted. The desalination plant of DUBAL was contaminated due to tar balls penetrating into the intakes, and on load cleaning was carried out by the use of Taprogge balls operation. This incident was relatively minor in comparison with the potential amount of oil which could be spilt into the sea by a bulk carrier carrying up to 250,000 tonnes of oil.

This incident along with the massive pollution which has occurred during the Gulf conflict serve to illustrate the seriousness of the problem. A major oil spill could result in a forced shutdown of the desalination plant for several weeks which if happened would jeopardise the state's drinking water supply.

Taking into consideration the above described incident and the fact that this area is situated in one of the world's busiest oil tanker shipping ways, a decision was made to build a permanent and effective barrier to protect DUBAL's seawater intakes from the threat of oil slick.

PHYSICAL PROPERTIES OF OIL :

In order to have a clear picture of the nature of the oil slick, it is important to know the physical properties of oil. This is important in the process of weathering and oil slick movement. Oil is usually classified into four categories according to the International Tanker Owners Pollution Federation Limited (ITOPF) groupings :

Group I	Group II	Group III	Group IV
S.G. < 0.8 (°API > 45) e.g. Gasoline Naphtha kerosene	S.G. 0.8 - 0.85 (°API 35 - 45) e.g. Abu Dhabi Qatar Marine Gas Oil	S.G. 0.85 - 0.95 (°API 17.5 - 35) e.g. Arabian Light Arabian Medium Arabian Heavy Iranian Heavy Kuwait	S.G. > 0.95 (°API < 17.5) e.g. Heavy Fuel Oil

(API gravity - American Petroluem Institute. Most crude oils and petroluem fractions has values of API gravity in the range of 10 to 80. Light hydrocarbons have values of API gravity ranging upward from 92.8).

API gravity = 141.5 / (S.G. (60/60 °F)) - 131.5

Oils within Group I will evaporate within 24 hours however, for those in Group IV only 10 % of the oil will have evaporated in the same time period.

WEATHERING PROCESSES :

Spreading :

Spreading is the most significant process during the early stages of a spill. The spreading is predominated by gravitational forces and as a result a high viscosity oil will spread slower than low viscosity oil. The spreading is then replaced by surface tension. After several hours of an oil spill a narrow band of oil will form "windrows".

Dispersion :

The speed of dispersion is governed by the viscosity of the oil. Droplets of oil are formed by the act of waves and turbulence on the surface of the sea. These can remain in suspension or return to the surface. Dispersion increases the total surface of the slick thus allowing the process of weathering to take place (e.g. sedimentation and biodegradation).

Evaporation :

There is a large range of evaporation rates for different types of oil. Less volatile oils may persist in the environment for several weeks, whereas kerosene and gasoline can evaporate within 24 hours.

Emulsification :

Under some circumstances a water in oil emulsion may form. Small droplets of water are absorbed into the oil increasing the volume of oil by a factor of three to four times (and its density). The emulsions which are formed tend to become extremely viscous and, as a result, it becomes difficult to disperse. In moderate to rough seas most oils rapidly form emulsions. The rate of emulsification is a function of the viscosity of the oil and sea state. High viscous oils will form emulsions more slowly than oils with low viscosity.

Oxidation and Dissolution :

Both of these processes play a minor role in the weathering of oil. Neither change the physical properties affecting spreading to any significant degree.

Sedimentation :

Sedimentation is a long term process which affects the weathering of oil. Sinking of oil usually brought about by adhesion of particles of sediment or organic matter to the oil. Most heavy fuel oils and water in oil emulsion have a specific gravity near unity and therefore require a very little entrained matter to produce sinking.

Shallow water conditions as in DUBAL case are often laden with suspended solids providing favourable conditions for sedimentation. Heavy contamination may lead to accumulation of large amounts of sediment in the oil forming dense tar balls. This can create a number of specific problems as tar balls tend to sink and can be found at any level in the water column. Systems such as booms are of limited use and air bubble curtains can easily be breached by heavy balls.

Biodegradation :

Marine bacteria, moulds and yeast which can utilise oil as a source of carbon and energy is known as a biodegradation process, and it only takes place at an oil/water interface. This is a long term process and the rate of biodegradation is subject to temperature and the availability of oxygen and nutritions such as compounds of nitrogen and phosphorous.

The movement of the slick is dependent on the current direction and partly depends upon wind speed and direction. Shear forces generated by the wind move the slick in the direction of the wind. The dominant direction of the wind average throughout the year is from north west to west direction and thus it is an onshore wind.

In general the movement of the oil slick will depend upon the time of the day. At night the slick will be pushed offshore but during the day the direction of transport will be onshore (temperature in version).

To summerise the effect of the environmental conditions on slick it is clear that the movement of the slick is heavily dependent upon the prevailing environmental conditions and the effects of current, wind and waves will be superimposed on each other. Tidal currents will move the slick in a direction parallel to the coast whilst waves will drive the slick inshore, while during the daytime the wind will drive it onshore and at night offshore. In general the movement of the slick will be onshore.

METHOD CONSIDERED FOR INTAKES PROTECTION :

Several methods have been considered to ensure a continuous supply of seawater from the intakes. The problem was approached from two directions; either the oil is prevented from entering the intake or the water is treated to remove oil onshore. The methods that have been considered includes:

1 - The construction of a rubble mound breakwater structure to protect the intake.
2 - The use of floating booms and pneumatic barriers.
3 - Extending the existing intake into deeper water.
4 - The construction of a reinforced concrete 'house' to protect the intake.
5 - The use of various onshore treatment techniques.

DESIGN ALTERNATIVES :

Five alternative schemes have been investigated, each of the designs offers different levels of protection for the intake structure and desalination plant. For example, the breakwater scheme will filter water and block the movement of tar balls towards the intake. The floating boom and bubble barrier scheme, however, will not prevent the movement of large tar balls and its general effectiveness will be much reduced in heavy sea conditions. The latter scheme is cheaper than the breakwater.

1 - Breakwater Scheme :

Three alternative breakwater schemes were considered. Option A is a groyne structure extending from the shoreline to the intake. It has an arm extending from the shoreline to the intake which can be placed north or south of intakes (Figure 1). The structure encloses the inlet on three sides leaving an oil slick boom with a deep net to protect the side closest to the shore.

Option B and C are modifications to this design (Figure 2 & 3). The groyne extending from the beach to the inlet has been removed for option B. This increases the wave activity directly adjacent to the site and therefore provides less shelter for the intake. The wave climate in this case (option B and C) will be more severe than in option A reducing the level of protection. Option C consists of several similar offshore breakwaters designed to reduce erosion rates at the beach by dissipating wave energy offshore.

The breakwater provides the highest level of security against oil slick pollution. The structure acts in three ways to prevent pollution reaching the intake. Firstly the structure will trap oil at any level in water column. Thus, tar balls, which would evade booms and pneumatic barriers, will be trapped at a low level and the slick will "run aground" at higher level. It is possible that some oil would penetrate the breakwater and make its way into the intake basin but during times of severe pollution, an oil mop boom and sheets could be placed inside the basin to catch any oil percolating through the breakwater.

There will also be much less chance of the slick being drawn into the intake because of the relatively calm conditions within the basin. Thus, the level of dissolution and formation of oil droplets by turbulence will also be low.

There are however several disadvantages. An oil spill may clog the pores of the breakwater. If the structure cannot be effectively cleaned then the pattern of flow through the rock will be changed. In very severe circumstances it has been known for water to be drawn through the subbed below the breakwater.

2 - Floating Boom :

During the AKARI oil pollution incident, a curtain type of boom with a net was deployed at DUBAL to protect the intakes. Oil booms have three basic components, namely, in a floatation cell supporting a freeboard above the water level and a skirt below preventing oil being swept underneath. There are large variety of booms available for work in open sea conditions or in rivers.

It is widely acknowledged that booms used in open sea conditions are of limited effect for the following three main reasons :

1 - Flow down the face of the boom.
2 - Splash over due to wave action.
3 - Droplet shedding.

The booms cannot contain oil against water velocity above 1 knot (0.5 metre per second) acting at right angle. The high viscosity oil tends to accumulate at the boom face and to flow vertically dawn and under the skirt whereas low viscosity oils are carried under the boom as droplets.

And in conclusion a floating boom solution provides effective protection in low currents and calm sea conditions. Unfortunately DUBAL is an exposed location and the degree of effectiveness of such a system is much reduced.

3 - Pneumatic Barrier :

They are commonly known as bubble barriers which have been used in a number of maritime circumstances. Air forced through a perforated pipe rises in bubbles forming a curtain and creating an upward current. This upward current causes a vertical circulation system to be set up thus redirecting sediments or an oil slick to the surface and then away. The air bubbles create a counter current on the water surface that held the oil against a water flow of up to 0.7 knots.

The limitations of bubble barriers appear to be about 0.5 meter per second and wave motions in excess of 1 meter per second render them ineffective. Therefore, this kind of protection is of no practical use and cannot be considered as a feasible alternative. Also, they would be difficult to maintain.

4 - Extension of Intake into Deeper Water :

There is a possible alternative which is to extend the existing two meter diameter intake pipe into deeper water thus reducing intake velocities. A further reduction in velocity can be achieved by careful redesign of the intake head. The overall effect of this solution would be to reduced the likelihood of oil droplets being drawn into the system.

Taking into consideration a successful design which has been in service for several years. It is generally recommended that this design of intake is placed in at least 9.0 meter water below mean sea level. The admiralty chart shows that the sea bed shoals very slowly and thus the intake would have to be located approximately six Kilometre offshore.

Extending the intake into deeper water does not provide the level of security against oil pollution compared to the breakwater scheme.

5 - Concrete Protection Structure :

As an alternative to protecting the intake with a rubble mound breakwater structure, a reinforced concrete "house" can be constructed to enclose the intake. The structure would consist of three walls with the forth side protected by a vertical moving sluice gate and a net to catch tar balls. The sluice gate could then be lowered or raised to prevent oil being drawn into the plant. The gate will be automatically operated by a control system which receives warnings from oil slick detectors located outside the structure.

The system suffers from a number of disadvantages. It would be subjected to high level of maintenance and inspection to insure that the control systems were functioning and the mechanical system associated with this gate operated. This highlights the virtue of the breakwater scheme which is essentially passive. Concrete and steel would also deteriorate in the aggressive marine conditions and in time this could lead to an expensive repair and replacement costs.

The level of protection affordable to the desalination plant by the structure is likely only to be in the medium to high range. This is due to the fact that no energy dissipation of wave occur. Indeed waves being reflected off the vertical walls both outside and inside the structure will increase the level of turbulence close to the intake. This will result in a greater quantity of oil droplets being forced into suspension and being drawn under the gate. Therefore it is highly likely that the current situation will be made worse rather than better.

After studying and evaluating the above mentioned alternatives, DUBAL decided to opt for the construction of breakwater option A and a boom system as the most effective method of protecting the seawater intakes due to the following reasons :

1 - High protection against oil slick.
2 - The wave activity within the water basin is small.
3 - An oil mop boom system can easily be deployed in the breakwater basin in severe water condition.
4- The use of dispertion near desalination intakes cannot be considered as the oil will be taken into suspension and will eventually enter the desalination plant.

BREAKWATER EFFECTS ON DESALINATION OPERATIONS :

Pre-construction :

As mentioned earlier DUBAL water intake was in shallow water with no effective protection against any oil slick. As a result of the intakes being in a shallow water (400 meter off shore), the desalination plant has experienced high turbidity seawater specially during winter season were any small storm will bring about a tremendous amount of silt to the plant (turbidity readings varied from 5 - 300 NTU). Laboratory tests have shown that suspended solids caused by seawater turbidity has the affect of absorbing antiscalent chemicals. As a result it was necessary to increase the antiscalent dose rate to tackle the excessive suspended solids entering the plant i.e. to ensure free antiscalent chemical is present.

During Construction :

Due to the construction work of the breakwater (1991), a massive amount of silt (suspended solids) was observed, caused by the disturbance to the sea bed. This has necessitated an increase in the antiscalent and the antifoam chemical consumption. As a result, several heat input section units (brine heater) were acid washed.

Post-construction :

The breakwater construction ended in late 1991. This has resulted in a tremendous decrease in the seawater intakes turbidity (maximum reported 50 NTU) which resulted in a decrease in the antiscalent and the antifoaming chemical consumption. This has also resulted in a substantial reduction in the frequency of the evaporators acid cleaning.

However, it was noticed that the rejected blowdown to the culvert was being recycled through the intakes. This recycling process had caused an increase in seawater temperature through the intakes by five degree celcuis. As a result, the evaporators Bottom End Temperature has increased causing a reduction in the flash range. Therefore, causing a noticeable reduction in the Mega-watt generations from the steam turbines.

In addition, the occurrence of the silt which was built up after the construction of the breakwater up to date has caused a problem, which is now under investigation. Therefore, DUBAL has recommended to dredge the accumulated silt of three to four years and the work will soon begin.

SEAWATER INTAKE TURBIDITY READING (1989-1993)

YEAR	TURBIDITY READING - NTU (HOURS)						
	05 - 10	11 - 20	21 - 30	31 - 50	51 - 100	101 - 150	>150
1989	N/A	758	313	255	121	9	22
1990	12	714	304	248	130	45	NIL
1991	921	562	214	173	174	77	7
1992	756	235	105	26	2	NIL	NIL
1993	591	187	67	51	3	NIL	NIL

CONCLUSION :

Several oil tanker accidents along with the massive disturbance during the Gulf conflict had necessitated DUBAL to look for a permanent solution against oil slick threat to ensure a persistent drinking water supply to the city of Dubai. Several methods of protection against oil slick threat have been evaluated. The evaluation has led to the construction of an extended breakwater (option A) with an oil boom system as a permanent and effective way of protecting the seawater intakes against any oil threat.

The chemical consumption had increased during the construction of the breakwater due to the disturbance of the sea bed caused by heavy rock dumping. At end of the construction the breakwater has benefited DUBAL in two ways, as a protection against oil slick threat and also it has worked as a wave stabiliser, thus reducing the seawater turbidity. However, unfortunately the design as such has led to clear evidence of rejected brine recirculation from the culvert to the seawater feed causing higher evaporator's Bottom End Temperature (BET). In addition, seasonal change in the current direction has resulted in the accumulation of silt in the breakwater basin followed by a significant recycle.

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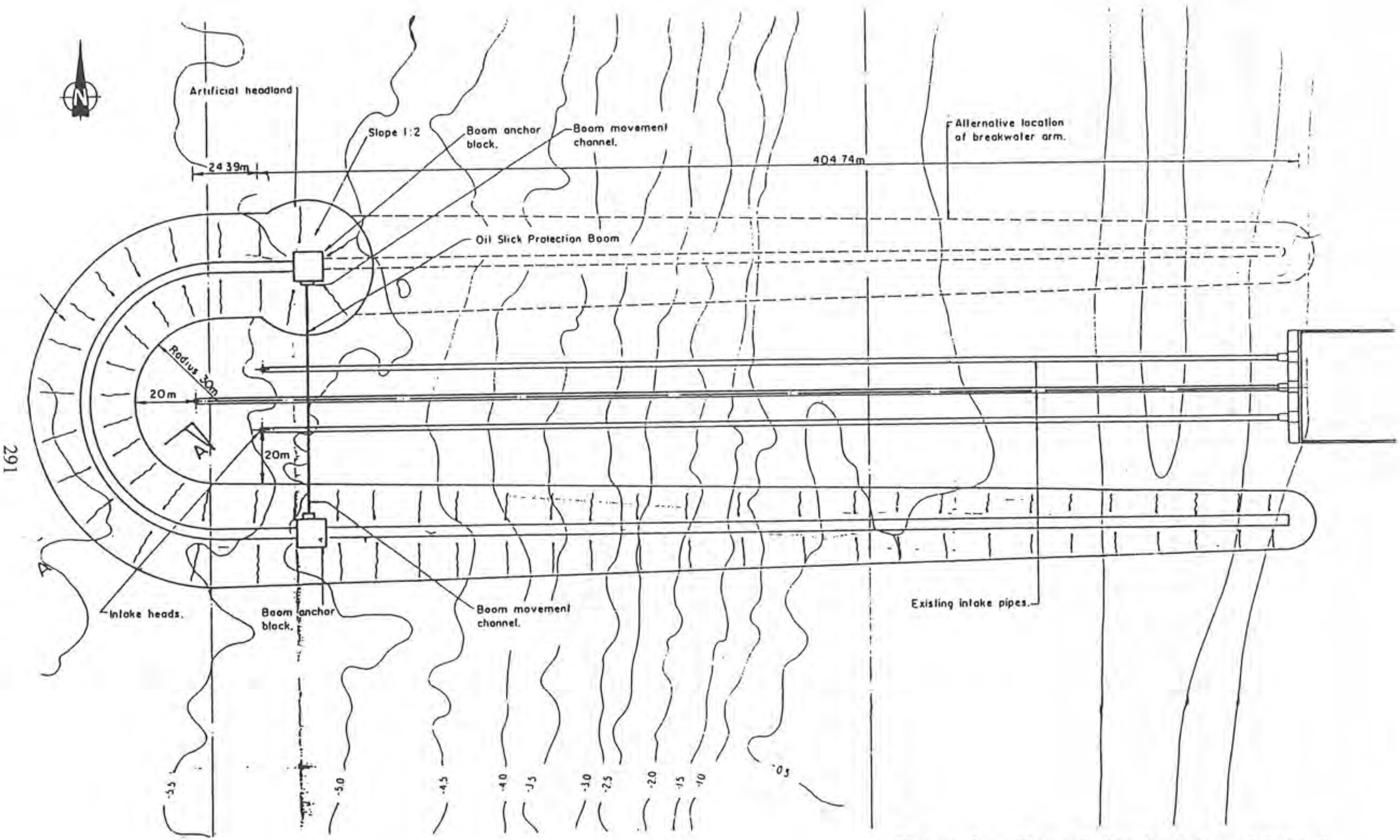

Figure 1 - Plan of Breakwater - Option A

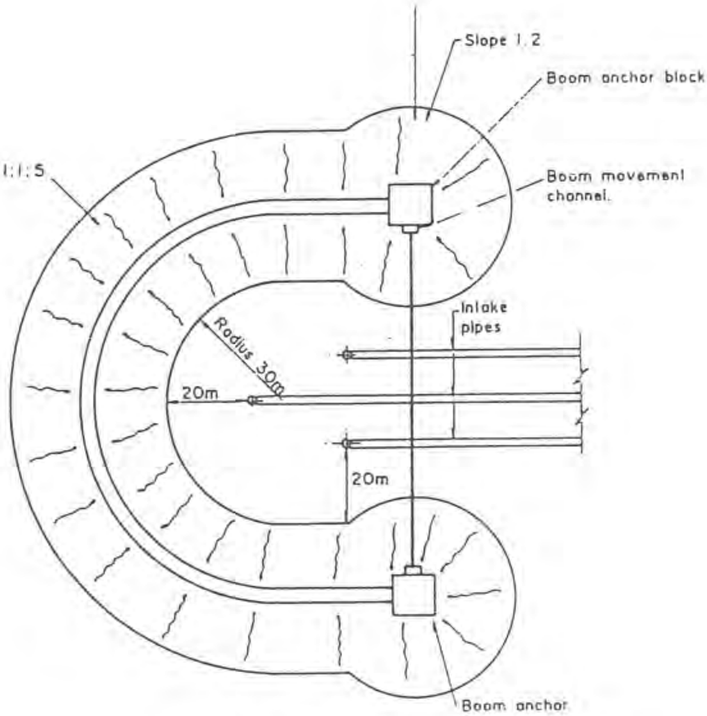

Figure 2 - PLAN OF BREAKWATER SCHEME B
Scale 1: 1250

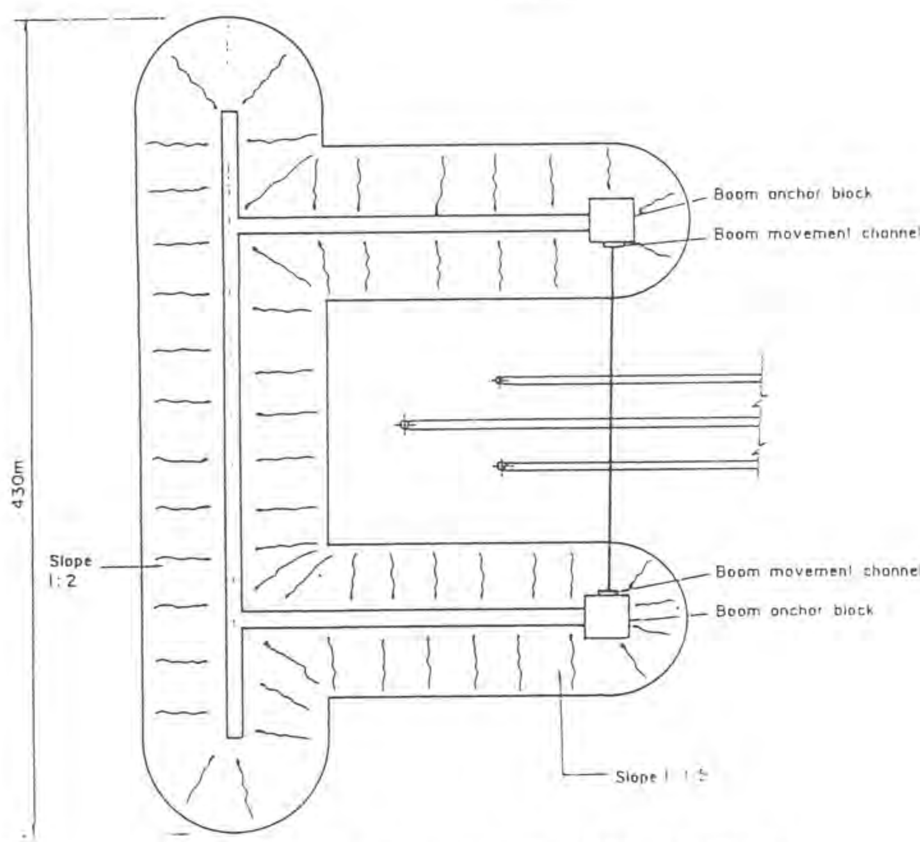

Figure - 3 - PLAN OF BREAKWATER SCHEME C

Session - 7
Reverse Osmosis Filteration

Foulants Investigation And Identification In an RO Membrane

Ahmed Hashim Ahmed

FOULANTS
INVESTIGATION AND IDENTIFICATION IN AN RO MEMBRANE

BY

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ABSTRACT

A comprehensive University research investigation was performed on a Du Pont B-10 PERMASEP® Permeator to identify the foulants at the Ras Abu Jar-Jur RO desalination plant. The objective of the research was to identify the types of foulants encountered by Du Pont's B-10 permeators during the course of their operating life at Ras Abu Jar-Jur in the desalination of high salinity brackish water from Aquifer C. To accomplish this vital task, one B-10 permeator (SN 8308923, Model 6840), which was in operation at Ras Abu Jar-Jur for about six years (since October 1984), was granted, by courtesy of the Ministry of Works, Power and Water, Water Production Directorate, to perform the necessary studies and investigations. At the University of Newcastle Upon Tyne, where I performed my Master research work (1991/92), extensive experimental investigations were conducted to identify the foulants endured by this permeator; the work mainly involved membrane chemical cleaning and fibre bundle autopsy; calcium and sulphate scaling and iron deposits were the potential foulants investigated. It was evident that biological fouling was predominant; the AAS analyses instituted the existence of a significant amount of calcium and a relatively lower amounts of sulphate and iron; the ED X-ray analyses performed on a fibre specimen confirmed the existence both of silica and sulphur scaling, whose presence was originally considered controversial, and revealed new types of aluminium and titanium scaling. This paper will include an assessment of all observations and results, a background to the nature of conditions to which the above permeator was subjected to, and the history of membrane fouling and adopted cleaning remedies at Ras Abu Jar-Jur, till the end of 1990.

KEYWORDS

B-10 permeator chemical cleaning and fibre bundle autopsy; AAS, SEM and ED X-ray analytical evaluation; recognized and unrecognized foulants.

AN INTRODUCTION

To bring to light the potential foulants expected to have been endured by the permeator, it would be convenient, first of all, to study the background of the nature of the brackish feed water at Ras Abu Jar-Jur, the prevailing plant conditions, the pretreatment design and RO system; (Figure 1).

Designed to produce 46,000 m^3/day of drinking water, the Ras Abu Jar-Jur brackish water RO desalination plant was commissioned in October 1984. The plant is fed from Aquifer C (a high salinity brackish water underground reservoir ($\approx$ 12,200 mg/l TDS, 1988) and rising), through 15 deep water wells. The well water contains dissolved H_2S gas, dissolved hydrocarbons, anaerobic bacteria (SRB), silica and inorganic suspended solids; it, thus, requires extensive pretreatment before reaching the RO system [1]. At the plant, the feed water goes through three sets of filtration systems : (1) dual-media filters (DMF); (2) activated carbon filters (ACF); and (3) micron guard filters (MGF), respectively. The DMF, comprising 10 filters, is designed for the removal of suspended matter and debris picked-up from the raw water wells and inner walls of the underground GRP pipeline, (the DMF has long been by-passed, in May 1990). The ACF, comprising 18 filters, is designed for the adsorption of dissolved H_2S and hydrocarbons. Exiting the ACF, the feed water is injected first with Na-HMP for sulphate ($CaSO_4$) scaling prevention, (Na-HMP was replaced by FLOCON®-100 and Belgard® BRO in March 1991), and with H_2SO_4 for alkaline ($CaCO_3$) scaling prevention, just before the MGF. The MGF, comprising 8 filters, is designed to guard against fine (microscopic) suspended matter, micro-organisms and other foulants. The feed water exiting this last filtration defense is passed to 7 high pressure pumps (HPP)/RO Blocks, (one additional HPP/RO Block unit was incorporated into the RO system, commissioned in January 1994). Comprising 301 permeators and operating at 65% conversion, each RO Block is arranged in 3 brine stages and the permeators are arranged in a 5:3:2 ratio. The RO Blocks combined product is post-treated before it is used for drinking.

During the initial plant operation, performance was very satisfactory; however, membrane fouling problems, especially biological growth, began soon after, within the first year of operation [1]. One of the earliest fouling problems detected was the excessive biological activity in the DMF and ACF vessels; later, it was deduced that the over-ground GRP lines at the pretreatment were translucent enough to allow sufficient sunlight through for photosynthesis to occur, resulting in a massive algal formation and bacterial growth, thus, causing an increase in DO in the feed water. The bacterial population eventually managed to advance to the MGF and then to the RO system, resulting in the biological fouling of permeators and, consequently, a decline in plant performance. To remedy this problem, the lines were painted to inhibit biological growth, the DMF were periodically sterilized with $Ca(OCl)_2$ and the ACF were constantly back-washed; in addition, the activated carbon itself was regenerated about 3 times a year and the replacement frequency of cartridge filter elements was increased. The RO Blocks were regularly scheduled for SBS or HCHO sterilization and chemical cleaning; however, biological fouling of the permeators was not fully eliminated, making it the major fouling problem at Ras Abu Jar-Jur. Further investigations revealed that biological fouling was also attributed to aerobic bacteria growing in the Na-HMP tanks; Na-HMP is a polyphosphate whose reversion into orthophosphate (which is a nutrient for micro-organisms) is influenced by concentration, temperature, pH and time [2]; this problem was dealt with by the addition of small quantities of SBS in the Na-HMP tanks to keep the injection system continuously sterilized. Yet, biological fouling of the permeators persisted.

The second possible type of permeator fouling at Ras Abu Jar-Jur is that of colloidal sulphur formation. It is relevant to mention that the pretreatment at Ras Abu Jar-Jur is a pressurized system; this feature is crucial to avoid the oxidation of the dissolved H_2S by preventing air from entering into the system, which could consequently cause the deposition of colloidal sulphur on the surface of the fibre membranes. Further, during the first year of operation, the activated carbon became saturated which led to an anxiety from the possible formation of colloidal sulphur fouling [2]; in addition [1], when algal growth, at the pretreatment, was eliminated, anaerobic environment was sustained where SRB activity flourished, utilizing sulphate ions in the feed water for their energy and generating H_2S as a by-product, thereby raising the dissolved H_2S level which added to the possible colloidal sulphur fouling formation.

The third type of possible fouling at Ras Abu Jar-Jur is the formation of $CaCO_3$ and/or $CaSO_4$ scale on the fibre membranes; even though the feed water is pretreated using H_2SO_4 and Na-HMP (the latter was substituted in March 1991), there is always the possibility of these salts precipitating, especially during the failure of these systems. Theoretical calculations by Sasakura Engineering Co. Ltd. [3] verified that at feed water TDS of 12,200 mg/l and RO conversion of 65%, only calcium carbonate has a definite scale forming ability whilst the calcium sulphate formation is only marginal; moreover, the brackish water at Ras Abu Jar-Jur is under-saturated with respect to $CaSO_4$ [4].

The possibility for dissolved hydrocarbons to cause membrane fouling at Ras Abu Jar-Jur was trivial, since its presence is within tolerable limits, < 0.05 mg/l [5], and all traces are effectively adsorbed by the activated carbon. The well water at Ras Abu Jar-Jur also contains silica, but its precipitation could only be a threat at RO conversions higher than design [4]; however, tests performed through Du Pont [6] on fibre and Reemay specimen obtained from a B-10 permeator, autopsied at Ras Abu Jar-Jur, revealed the presence of, what was called, sporadic colloidal silica fouling; it was argued that the DMF is the major suspect for this fouling and suggested it to be by-passed. During 1988/89, inorganic ions were detected in both citric acid and Na_4-EDTA spent cleaning solutions; additional chemical cleaning was performed where several kilograms of Ca ions were removed [3], which indicated a definite inorganic fouling. In the years that followed, a rapid decrease in productivity was experienced, suggesting membrane fouling, which led to more frequent cleaning, and also to the development of new membrane cleaning techniques at Ras Abu Jar-Jur [1].

In conclusion, it is apparent that the membrane fouling experienced by the B-10 permeators at Ras Abu Jar-Jur is caused by several foulants in different proportions; there is never any consistency in the results of all the calculations, autopsies and analytical techniques employed to specify the exact nature of fouling. Consequently, the results obtained through this research will play an important role in the verification of the existing foulants. (Table I shows the history of this B-10 permeator).

1.0 CHEMICAL STERILIZATION AND CLEANING

The objective of the chemical cleaning is to pursue two major tasks : (1) remove the suspected foulants from the B-10 permeator *in situ* using Du Pont approved chemical cleaning agents; and (2) identify the types, and approximate weights, of the foulants present.

The unit constructed to perform the permeator sterilization and chemical cleaning is shown in Figure 2 [7,8]. During permeator cleaning, several measures were considered; of importance : (1) a good quality, < 500 mg/l TDS, chlorine-free water was used for all applications; (2) to ensure cleaning efficacy, crystalline and powdery chemical solutions were first prepared in a glass beaker before transferring to the cleaning tank; and (3) before and after each cleaning, the permeator was thoroughly flushed to prevent harmful interactions between the different chemicals used. The following, in chronological order, are the chemical sterilization and cleaning methods performed on the permeator, and the expected foulants to be removed [7,8,9] :-

1. Formaldehyde "HCHO" sterilization (0.5 and 2.0 wt.%); followed by
2. Detergent cleaning (0.5 wt.%, Decon 90) + NaOH at pH 11.0, for the removal of biological (organic) matter.
3. Citric acid (2.0 wt.%) + NH_4OH at pH 4.0, for the removal of $CaCO_3$ and metal oxides.
4. Citric acid (2.0 wt.%) + NH_4OH at pH 8.0, for the removal of $CaSO_4$ and $SrSO_4$.
5. Ethylenediaminetetraacetic acid-tetra sodium salt "Na_4-EDTA" (1.5 wt.%) + HCl at pH 7.0 ~ 8.0, for the removal of $CaSO_4$ and $SrSO_4$.
6. Sodium hexametaphosphate "Na-HMP" (1.0 wt.%), for the removal of metal oxides.

1.1 HCHO Sterilization

When the permeator was initially inspected, soon after arrival from Bahrain, a very strong and distinct H_2S smell was evident, which could have been caused only by the SRB growing within the permeator with the water being stagnant for a long time; (a liquid sample was extracted for bacterial tests). It was, therefore, necessary to sterilize the permeator to eliminate biological fouling prior to cleaning. The liquid HCHO used is manufactured by the Aldrich Chemical Company Ltd. as 35 wt.%. The sterilization procedure employed, with respect to flow and pressure guidelines, was strictly accordant to that specified in Du Pont's Permasep Engineering Manual (PEM) [8]. The initial permeator flushing was the longest, where huge quantities of water were used to expel all traces of H_2S from the permeator. Owing to the severity of biological fouling of the permeator, HCHO sterilization had to be performed twice; the first was carried-out using 0.5 wt.% for a period of 96 hours, and since biological fouling smell was still evident in the effluent, it necessitated a second sterilization at a higher concentration, 2.0 wt.%, for 120 hours.

1.2 Detergent Cleaning

Even though detergent cleaning is recommended by Du Pont for the effective removal of colloidal fouling from B-10 permeators [7], it is also effective in removing non-viable biological matter following permeator sterilization. The detergent used, Decon 90 liquid cleaner, is an approved Du Pont cleaning agent; it is manufactured by the Decon Laboratories Ltd. as 100 wt.%. The detergent cleaning procedure employed, with respect to flow, pressure and pH guidelines, was strictly accordant to that specified in the PEM [7]. 0.5 wt.% detergent solution was used for cleaning the permeator, which was circulated through for two hours to ensure cleaning effectiveness.

1.3 Citric Acid Cleaning at pH 4.0

Citric acid cleaning at pH 4.0 was performed to remove the suspected $CaCO_3$ scale and metal oxides deposits from the permeator; in the case of this particular permeator (i.e. from Ras Abu Jar-Jur), iron is the most likely metal to cause metal oxide fouling. The citric acid crystals used are manufactured by Merk Ltd. (BDH). The citric acid cleaning procedure employed, with respect to flow, pressure and pH guidelines, was strictly accordant to that specified in the PEM [7]. 2.0 wt.% citric acid solution was used for permeator cleaning, which was circulated through for two hours; at the end of circulation, a sample of spent solution was retained for Ca, CO_3 and Fe ions analysis.

1.4 Citric Acid Cleaning at pH 8.0

Citric acid cleaning at pH 8.0 was applied to remove the suspected $CaSO_4$ and $SrSO_4$ scaling from the permeator. The citric acid cleaning procedure employed was exactly the same as the one at pH 4.0, in all details, [7]. A sample of spent solution was retained for Ca and SO_4 ions analysis.

1.5 Na_4-EDTA Cleaning

Na_4-EDTA is a chelating agent used to chelate, or sequester, divalent alkaline earth cations like Ca, Ba and Sr, which are the principal scale forming ions [10]. This type of permeator cleaning is applied in a weak alkaline environment; Na_4-EDTA is one of few chemicals that are effective in removing the most tenacious $CaSO_4$ scale. The Na_4-EDTA cleaning was used to remove the suspected $CaSO_4$ and $SrSO_4$ scaling. The Na_4-EDTA powder used is manufactured by the Aldrich Chemical Company Ltd., and the cleaning procedure employed, with respect to flow, pressure and pH guidelines, was strictly accordant to that specified in the PEM [7]. 1.5 wt.% Na_4-EDTA solution was used for permeator cleaning, which was circulated through for two hours; a sample of spent solution was retained for Ca and SO_4 ions analysis.

1.6 Na-HMP Cleaning

Na-HMP cleaning was applied for removing deposits of metal oxides from the permeator, (as stated earlier, iron is most likely to cause metal oxide fouling). The Na-HMP powder used is manufactured by Merk Ltd. (BDH). The Na-HMP cleaning procedure employed, with respect to flow and pressure guidelines, was strictly accordant to that specified in the PEM [7]. 1.0 wt.% Na-HMP solution was used for permeator cleaning, which was circulated through for two hours; a sample of spent solution was retained for Fe ion analysis.

2.0 PERMEATOR DISASSEMBLY AND FIBRE BUNDLE AUTOPSY

This part of the experimental work involves : (1) B-10 permeator disassembly; and (2) fibre bundle autopsy. The objective of the permeator disassembly is to remove the fibre bundle safely out of its shell, (disassembly procedure was adapted from the PEM [11]); the tools and equipment used were conforming to those specified in the PEM [11], (some of the equipment used, like the end plate puller and the manual hydraulic pressure unit, were fabricated at the University's workshop). The objective of the fibre bundle autopsy is to cut-open the bundle, using a long sharp-blade, to examine and assess the condition of the fibre membranes and Reemay, starting from the outer perimeter right through to the centre of the bundle, and to retrieve samples of scaled fibres, (the autopsy procedure was adapted from practical experience at Ras Abu Jar-Jur). In no time, the cut-bundle windings were rolled and thoroughly inspected from beginning to end; samples of fibres were retrieved from the outer perimeter, middle and centre of the bundle, in addition to two relatively large particles found trapped within the inner Reemay in the direction of the incoming feed, close to the distributor tube, with one particle adhering strongly to the Reemay and had to be scissored-out. Additional fibres were taken at random along the rolled bundle for fibre elongation tests.

3.0 ELECTRON-MICROSCOPY OF THE HFF MEMBRANES

This part of the experimental work involves the electron-microscopy of the fibre membranes, and particles, retrieved during autopsy. The objective is to explore the surface morphology of the fibres and examine the two particles under the SEM, to assess the fouling condition, and to identify the elemental constituent of the scale present (left behind following the chemical cleaning). In addition, fibres from the bundle were cross-referenced against a specimen of new fibres.

The procedure used to prepare the samples of fibres and particles for Electron-Microscope scanning required great precision. A tiny portion from each sample was carefully selected and cemented on to aluminium stubs, using a quick drying carbon conductive cement; six samples were prepared in this manner. The samples were dried and sterilized, in a sputter-gold coater prior to scanning to prevent contamination of the SEM, and placed inside the Electron-Microscope and scanned from different angles. With the use of the *LINK QX-200* Electron-Microscope, the Energy Dispersive X-ray Analysis technique was employed to identify the elemental constituents of the scale present on the fibres from the centre of the bundle.

4.0 OBSERVATIONS AND RESULTS

4.1 Bacterial Test Results

The liquid sample extracted from the permeator was used to prepare two bacterial cultures, using marine-agar, where one was incubated at 30°C (303 K) and the other at 25°C (298 K), for seven days; at the end of the incubation period, it was observed that bacterial growth was three times lower in the second culture.

4.2 Chemical Cleaning Observations

1. At the end of the first and second HCHO sterilization, the circulated cleaning solution colour changed from colourless to dark, accompanied by a very strong foul smell, suggesting the

partial removal of biological mass from the permeator. Half way through the detergent cleaning, the solution became dark, and changed completely black towards the end of circulation; the effluent was accompanied by a distinctive foul smell, suggesting a massive bacteria and slime removal from the permeator.

2. At the end of the citric acid cleaning at pH 4.0, the colour of the solution changed to light-yellow, indicating the removal of iron deposits from the permeator.

3. Throughout the citric acid (pH 8.0), Na_4-EDTA and Na-HMP cleaning, the colour of the solution remained unchanged till the end of circulation.

4. With regard to cleaning solutions preparation involving crystalline and powdery chemicals, it was observed that citric acid crystals were the easiest to dissolve and Na_4-EDTA powder was slightly hard to dissolve, while Na-HMP powder was extremely hard to dissolve. During the Na-HMP solution preparation, as soon as the powder was placed in water, it began changing into a waxy substance which slowly hardened and adhered to the bottom of the glass beaker and became extremely difficult to dissolve; in the end, stirring had to be done in a warm water bath.

5. When chemical cleaning was completed, the 10 μm cartridge filter was inspected. First observation was the distinctive bacterial fouling smell caused by biological growth within the cartridge filter windings. Second observation was peculiar in the sense that the bacterial population was concentrated at the lower end of the filter and less towards the top.

4.3 Spent Cleaning Solutions Analyses Results

The Ca and Fe ions analysis was performed, using the Atomic Absorption Spectrophotometer, at the Department of Chemistry, University of Newcastle Upon Tyne. The SO_4 ion analysis was conducted by J & H S Pattinson (Public Analysts, 10 Dean Street, Newcastle Upon Tyne).

1- Citric acid (pH 4.0) spent solution analysis : Ca = 66 mg/l; and Fe = 1 mg/l;
2- Citric acid (pH 8.0) spent solution analysis : Ca = 15.6 mg/l; and Total SO_4 = < 2 mg/l;
3- Na_4-EDTA spent solution analysis : Ca = 4.4 mg/l; and Total SO_4 = < 2 mg/l; and
4- Na-HMP spent solution analysis : Fe = 0.75 mg/l.

4.4 Permeator Disassembly Observations

Even though most of the external parts of the permeator were corroded, disassembly of all parts was exceptionally effortless, and the fibre bundle was removed from the shell with great ease.

4.5 Fibre Bundle Autopsy Observations

1. The Reemay was observed to be stronger at the outer perimeter and the centre of the bundle, and was slightly weakening towards the middle radius of the bundle.

2. The plastic distributor tube was found completely broken-off at the feed end of the bundle.

3. In spite of all the chemical cleaning, the fibres were severely scaled at the centre of the bundle (i.e. in the direction of the incoming feed), and less severe towards the outer perimeter of the bundle.

4.6 SEM Results

The Electron-Microscope scanning investigation was performed at the Department of Mechanical, Materials and Manufacturing Engineering (Materials Division, Electron-Microscopy

Unit, University of Newcastle Upon Tyne). Below is a list of a few electron-micrographic results obtained using the SEM, and the X-ray graphical analyses determining the elemental constituents of the scale.

1- Photo A shows new HFF membranes, where smoothness and cleanliness are evident;
2- Photo B shows fibres from the outer perimeter of the bundle, with little scaling;
3- Photos C and D show fibres from the middle of the bundle, with a higher degree of scaling;
4- Photos E and F show fibres from the centre of the bundle, with the most severe scaling problem encountered. Figure 3 is an X-ray graph of this fibre sample, (the analysis shows the presence of Si, Ti, Al and S); and
5- Photo G shows the small scale particle and Reemay from the centre of the bundle. Figure 4 is an X-ray graph of this sample, (the analysis shows the presence of Si and Ti). The relatively larger particle was identified as carbon (activated carbon).

5.0 DISCUSSION

(1) The results of the bacterial test were not sufficient enough to draw any significant conclusions.

(2) The dark colour exhibited by the detergent cleaning solution may characterize the effectiveness of the sterilization/cleaning combination method employed for the removal of biological fouling.

(3) The use of both citric acid (at pH 8.0) and Na_4-EDTA cleaning for the removal of $CaSO_4$ and $SrSO_4$, was mainly to determine which chemical is most effective; although this could have been better accomplished if another permeator was available, however, the analyses of spent solutions confirmed the removal of insignificant amounts of SO_4 ions (< 2 mg/l, per cleaning), therefore, no SO_4 scaling.

(4) From spent solutions analyses, the total Ca removed from the permeator is ≈ 86 mg/l, which = 15.48 g; the Total SO_4 removed is ≈ < 4 mg/l, which = < 0.72 g; the total Fe removed is ≈ 1.75 mg/l, which = 0.315 g. Therefore, based on these results, it is evident that the most significant amount of scale present is that of calcium, and least significant are that of sulphate and iron.

(5) When the fibre bundle was being autopsied, there was no longer any evidence of biological fouling, which may be considered another evidence of the effectiveness of the cleaning methods employed.

(6) It is unusual to find the distributor tube completely broken-off, as it was; closer examination of the tube condition and the nature of the break strongly suggests that a very sudden high pressure must have had been exerted on the permeator at some stage during its operating life.

(7) The hollow fibres are normally very ductile; however, the elongation test performed on fibres from the bundle showed that they were unusually brittle and nearly lost all their ductility. This may indicate a degradation in the structure of the fibre polymer; such degradation could occur due to a short exposure to an oxidizing agent.

(8) With regard to the activated carbon particle found at the centre of the fibre bundle, the only source of this particle is the ACF; this could indicate a failure at some part in the MGF assembly.

(9) The electron-micrographs of the fibres show that the scaling severity is higher at the centre of the fibre bundle and decreases towards the outer radius. This is because the fibres at the centre are facing the incoming feed water, so they are bound to be most severely scaled; and as the

feed water traverses towards the outer radius of the bundle, the fibres and Reemay act as a filter, trapping tiny suspended matter within.

(10) The ED X-ray analysis performed on the samples in Photos E/F and G show the presence of a very high level of Si; this confirms the existence of silica fouling, which may be either SiO_2 scaling or colloidal silica fouling, or a combination of both.

(11) The X-ray analyses also indicates the presence of Ti in both samples. The feed water at Ras Abu Jar-Jur was never analysed for Ti, because it is one of the rare metals that are not at all anticipated to be present; in any case, its appearance in both samples cannot be logically explained, yet. The presence of Au is the result of the sputter-gold coating.

(12) The X-ray analysis performed on the sample in Photos E/F indicates the presence of Al. The Ras Abu Jar-Jur feed water analysis do not show Al as one of the constituents, which implies that it is either minutely present or has never been analysed for. There is also the possibility that the Al detected is that of the aluminium stubs.

(13) This X-ray analysis also shows the presence of S; again, it confirms the existence of sulphur fouling, which may be either SO_4 scaling or colloidal sulphur fouling, (since the spent solutions analyses showed negligible amounts of SO_4, the detected S would, most probably, represent colloidal sulphur).

(14) Neither of the two ED X-ray analyses showed any evidence of Sr deposits.

6.0 CONCLUSIONS

The following conclusions were determined based on the experimental research study of one single permeator from Ras Abu Jar-Jur, that may or may not be representative of all the other permeators which were in operation along side of this permeator.

(1). It may now be concluded that the most predominant permeator scaling problem at Ras Abu Jar-Jur is $CaCO_3$, which comes after biological fouling in terms of severity. A potentially damaging high pH circumstance will promote carbonate fouling of the RO permeators.

(2). The formation of silica scaling at Ras Abu Jar-Jur has always been a matter of controversy; now, however, there is no doubt that it is one confirmed scaling problem at Ras Abu Jar-Jur.

(3). The formation of colloidal sulphur fouling was also a subject for debate, because there was no substantive evidence to its existence. However, it is now apparent that colloidal sulphur fouling is present.

(4). Even though the ED X-ray analyses detected the presence of Ti and Al, however, their presence in the raw feed water needs to be analytically ascertained.

(5). Motivated by the above point, it would be wise to carry-out a regular, and complete, chemical and biological analyses of the raw water at Ras Abu Jar-Jur, to see whether or not the composition of constituents has changed from previous years, and if new constituents have intruded into the raw-water wells from the surrounding underground geology.

PRÉCIS

Inspired by the whole investigation of the membrane fouling episode *in situ*, there emerges one fundamental apprehension; the unequivocal necessity for pilot-testing facility at any RO desalination plant. In the present era of RO technology impetus and the universal state-of-the-art RO applications, especially in the field of water desalination, a pilot plant (miniature archetype of the RO plant itself) must be recognized as one indispensable part of any RO plant design. Pilot plants serve as the rudiment tool in the investigation and assessment of permeators performance, new chemicals compatibility and pretreatment optimization to overcome operating difficulties and envisage new techniques to combat membrane fouling contingencies.

RESEARCH IN THIS ENDEAVOUR SHOULD NEVER CEASE

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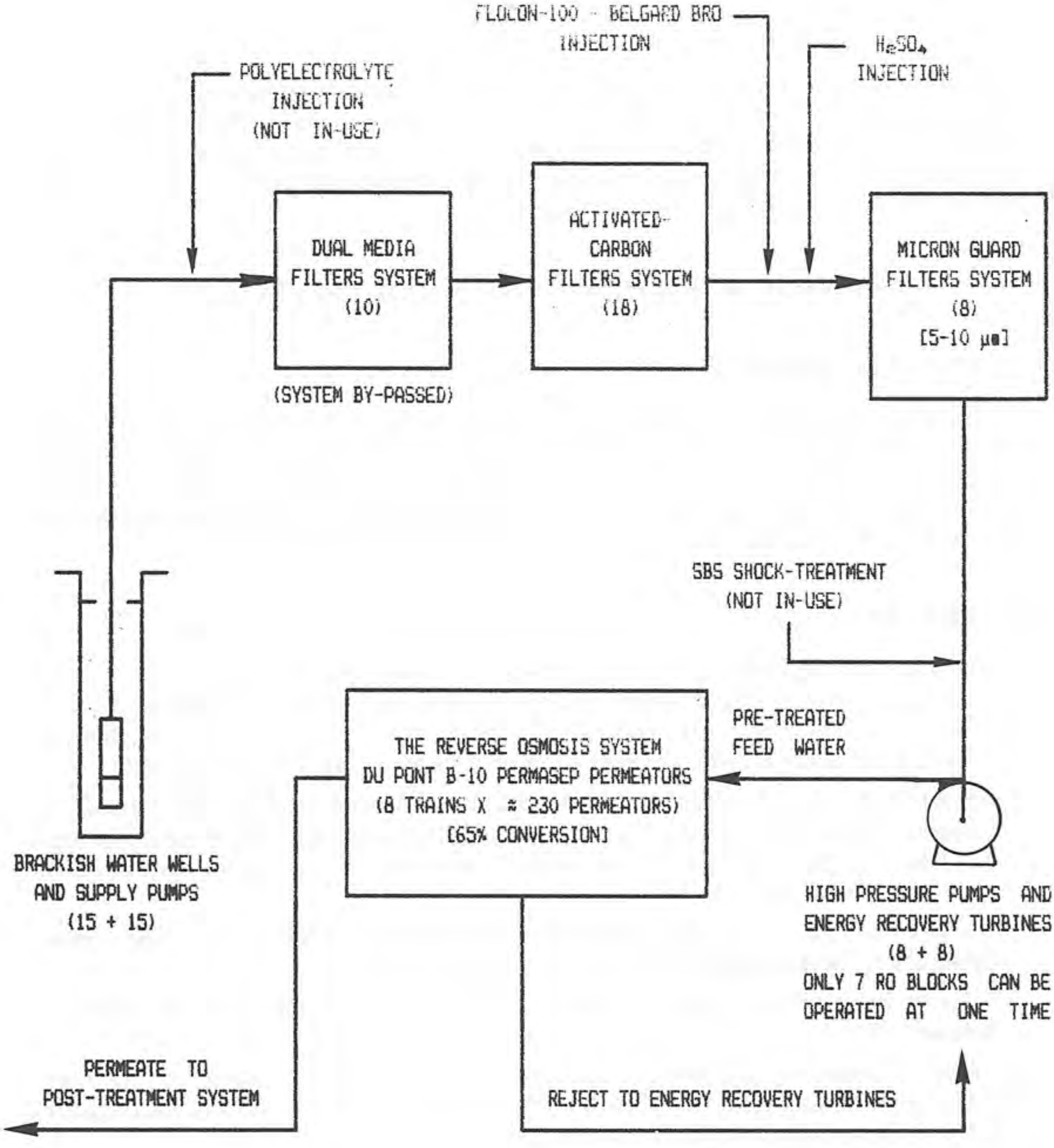

Figure 1

A BLOCK DIAGRAM
ILLUSTRATING THE PRE-TREATMENT SYSTEM
AT THE RAS ABU JAR-JUR BRACKISH WATER RO DESALINATION PLANT (BAHRAIN)

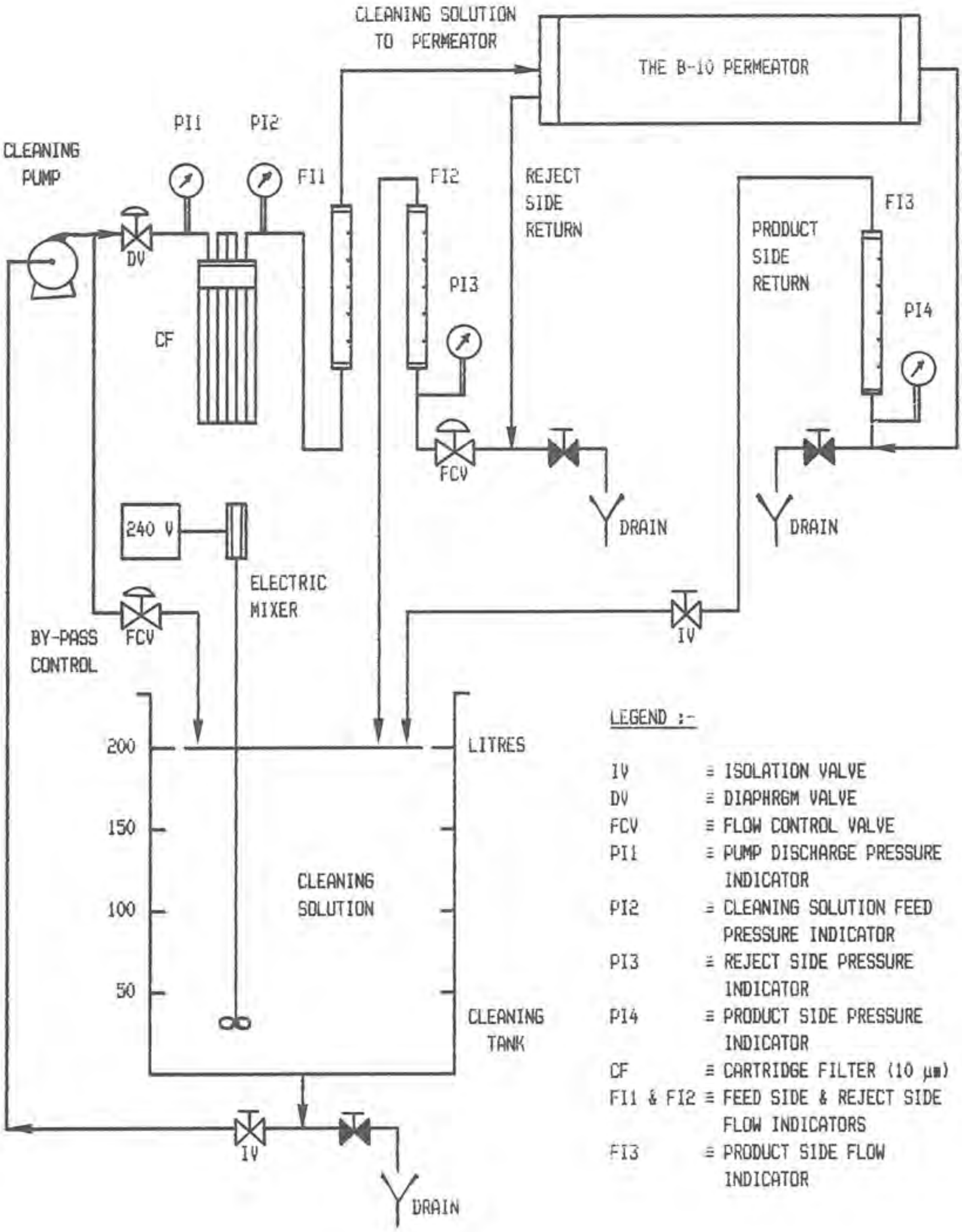

Figure 2

A SCHEMATIC ILLUSTRATION
OF THE CHEMICAL CLEANING UNIT

(Adapted from Du Pont [7,8])

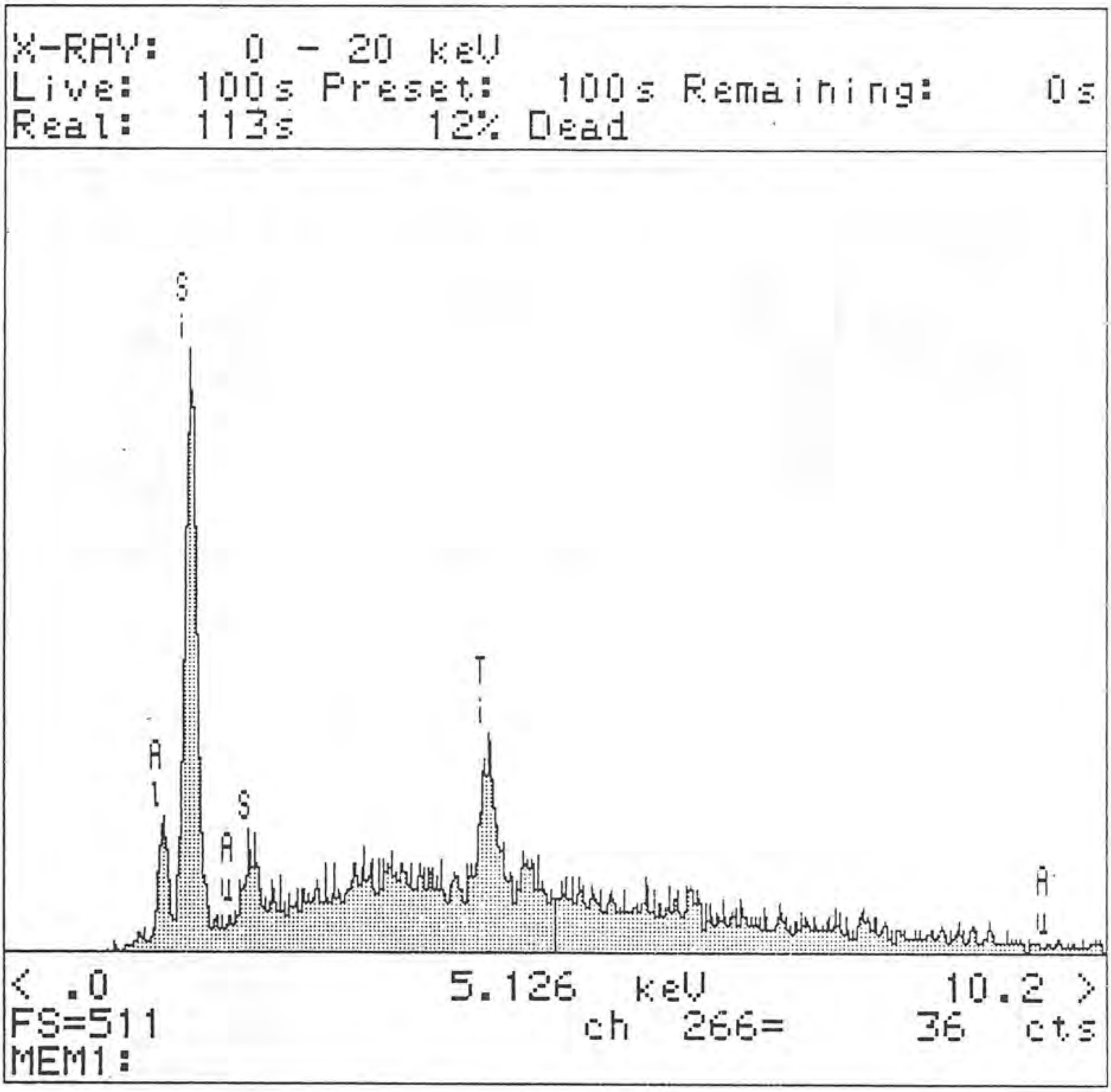

Figure 3

ENERGY DISPERSIVE X-RAY ANALYSIS
OF THE SCALED FIBRES FROM THE CENTRE OF THE BUNDLE
(ELEMENTAL CONSTITUENTS : SILICON, TITANIUM, ALUMINIUM, AND SULPHUR)

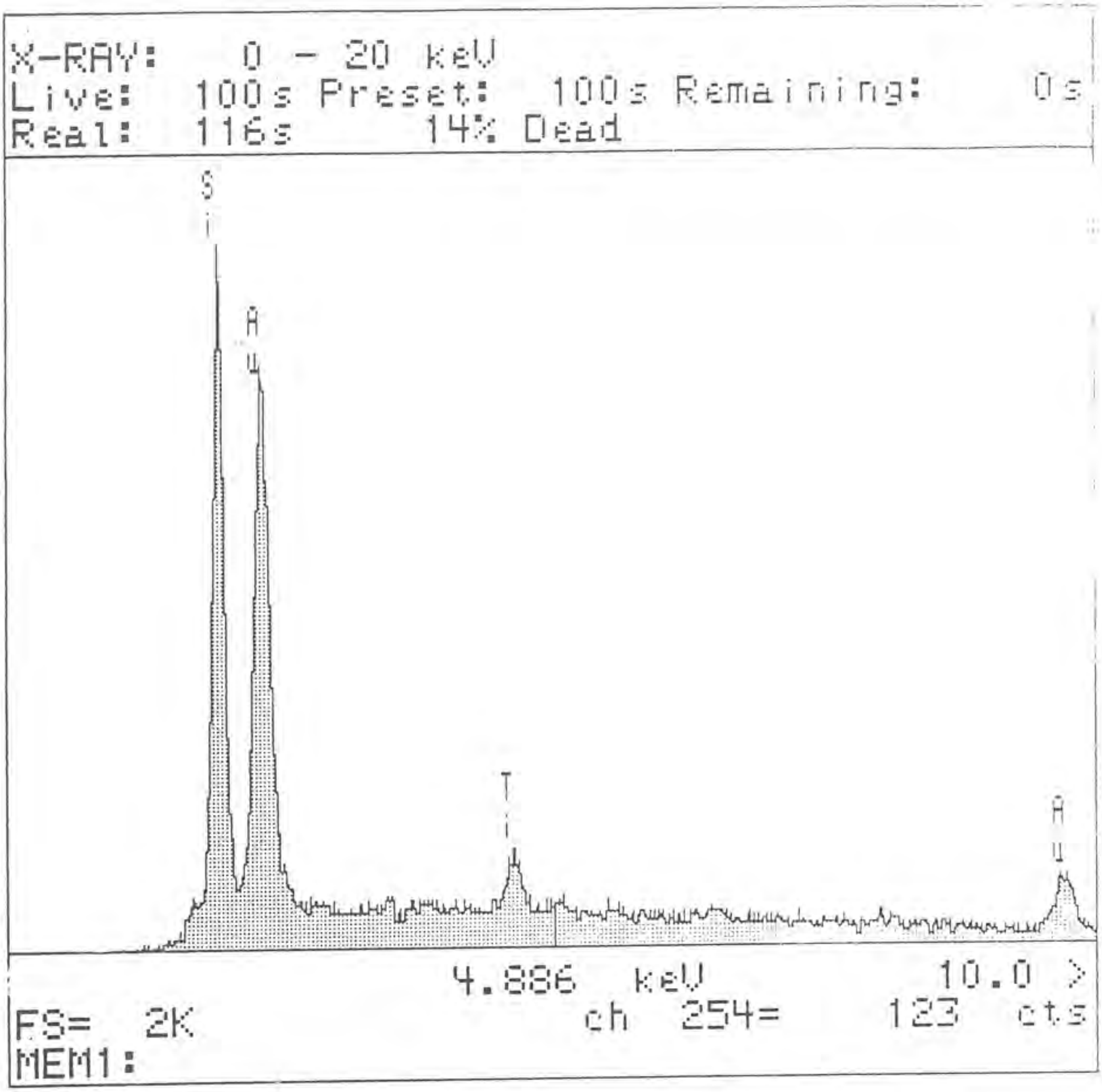

Figure 4

ENERGY DISPERSIVE X-RAY ANALYSIS
OF THE SCALE PARTICLE FROM THE CENTRE OF THE FIBRE BUNDLE
(ELEMENTAL CONSTITUENTS : SILICON AND TITANIUM)

[illegible]	[illegible]
[illegible] #	[illegible]
[illegible]	[illegible]-0658-07
[illegible] Manufacture	[illegible]
Date [illegible] Commissioning	[illegible] 1984
Date [illegible] Out of Serv.	March 19[illegible]
Position in the Plant	E 18 03 (RO Block E, First Stage, third in eighth row)
Cleaning History	1) No Cleaning During the first year of operation
	2) 1986 : 0.5 wt.% SBS Sterilization, (monthly)
	3) 198[illegible] : 0.5 wt.% SBS (or 2.0 wt.% HCHO Sterilization) + 0.5 wt.% Detergent (Ariel) Cleaning, (monthly)
	4) 1988 : 2.0 wt.% HCHO Sterilization + 0.5 wt.% Ariel Cleaning + 80 mg/l PT-B and 80 mg/l PT-A Treatments, (monthly)
	5) 1989 : 0.5 wt.% SBS (or 0.5 wt.% HCHO Sterilization) + 0.5 Ariel Cleaning + 2.0 wt.% Citric Acid Cleaning at pH 4.0 + 1.5 wt.% Na4-EDTA Cleaning + 80 mg/l PT-B and 80 mg/l PT-A Treatments, (monthly)

Table 1

HISTORY OF THE B-10 PERMEATOR *IN SITU*

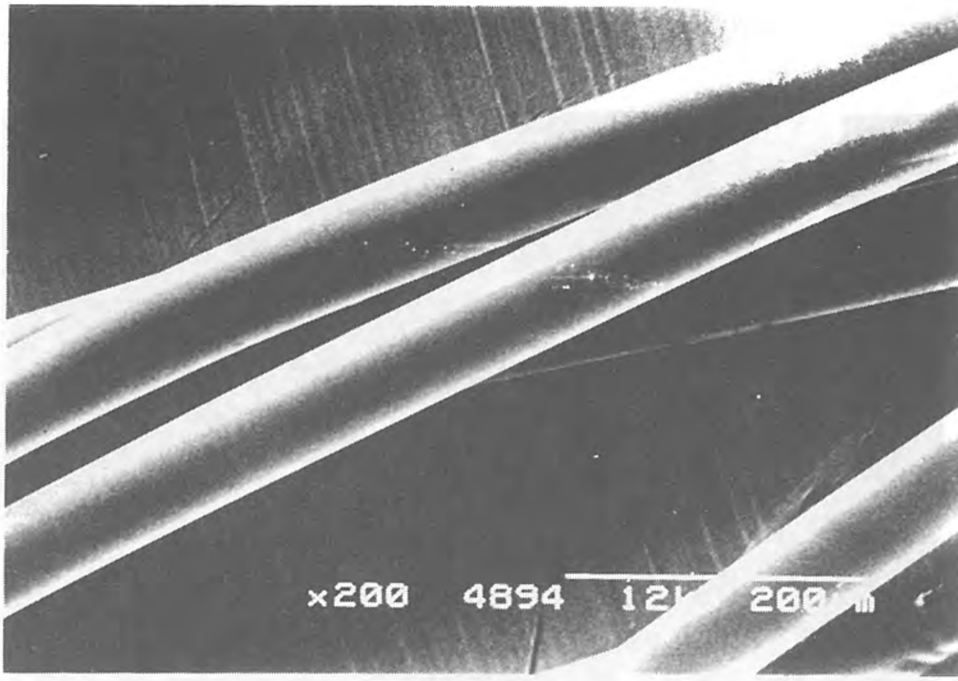

Photo A

NEW POLYAMIDE HOLLOW FINE FIBRES (x200)

AN ELECTRON-MICROGRAPH
SHOWING THE CLEANLINESS AND SMOOTHNESS OF NEW FIBRES

Photo B

FIBRES FROM THE OUTER PERIMETER OF THE BUNDLE (x100)

AN ELECTRON-MICROGRAPH
SHOWING A SLIGHTLY SCALED FIBRES

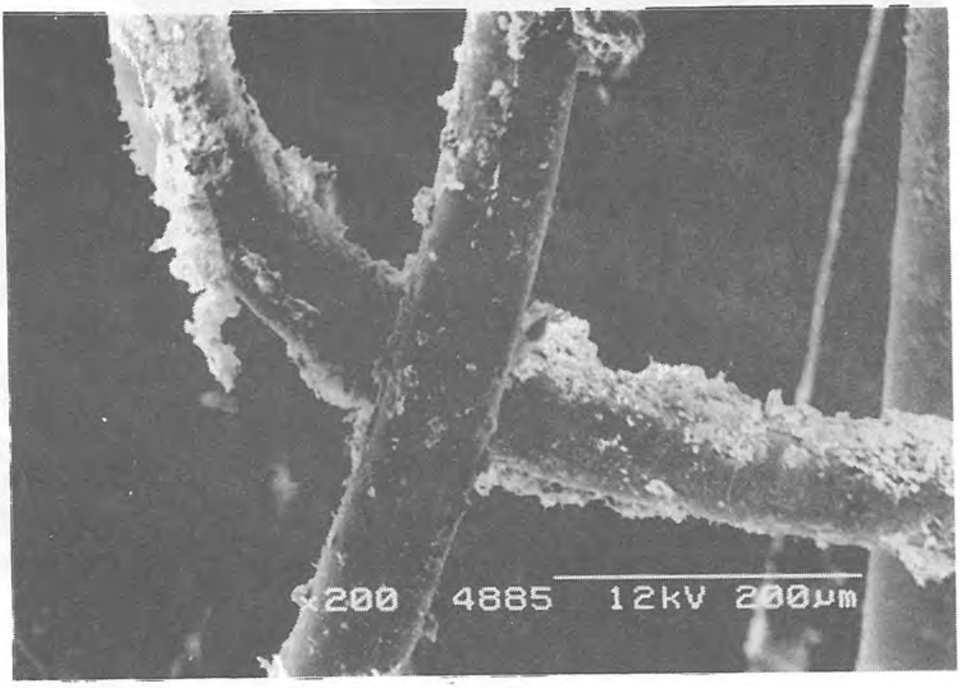

Photo C

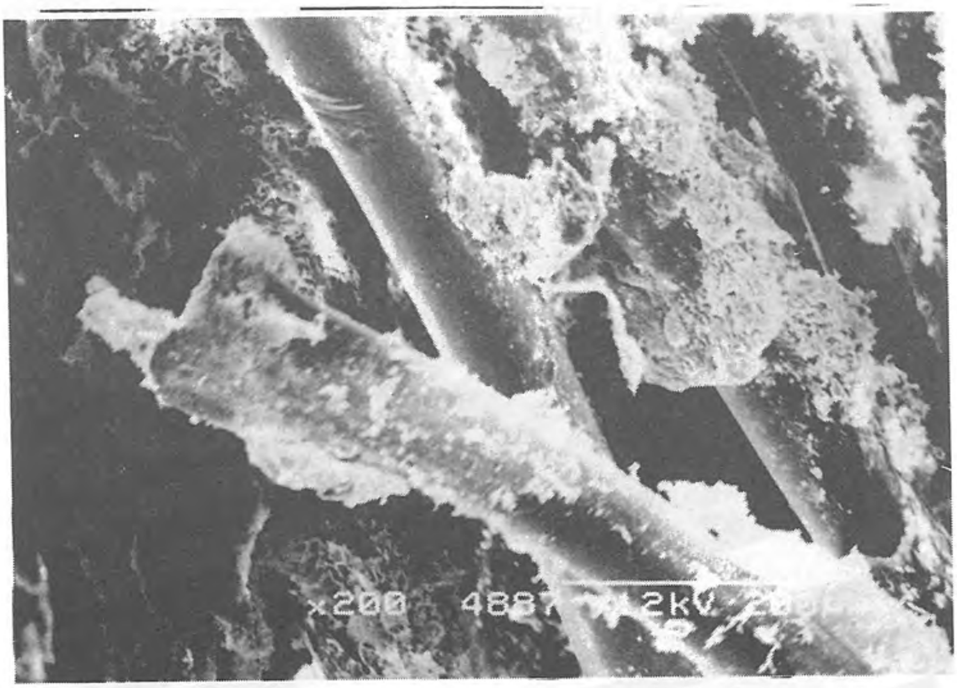

Photo D

FIBRES FROM THE MIDDLE OF THE ROLLED BUNDLE (x200)

TWO ELECTRON-MICROGRAPHS
SHOWING DENSELY SCALED FIBRES

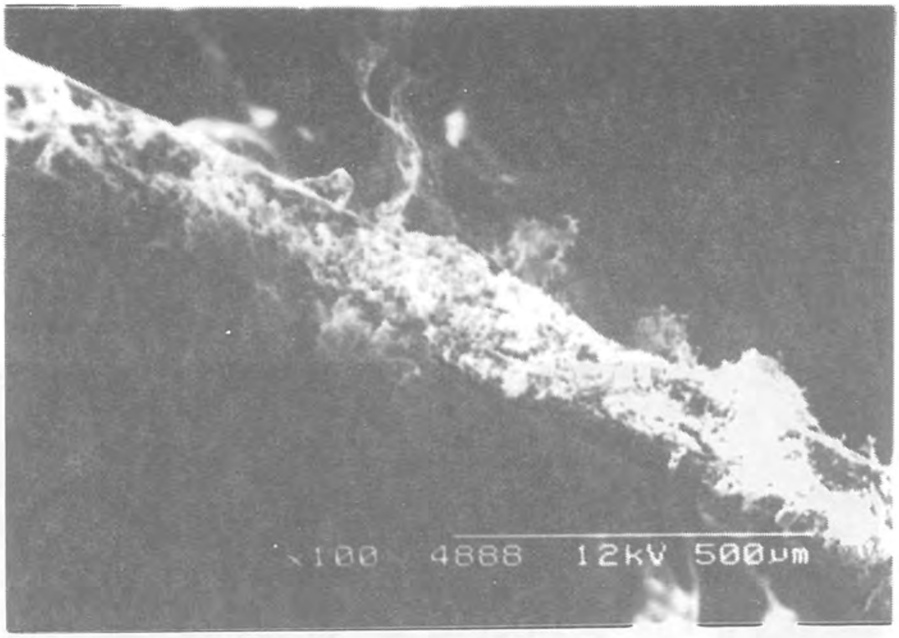

Photo E

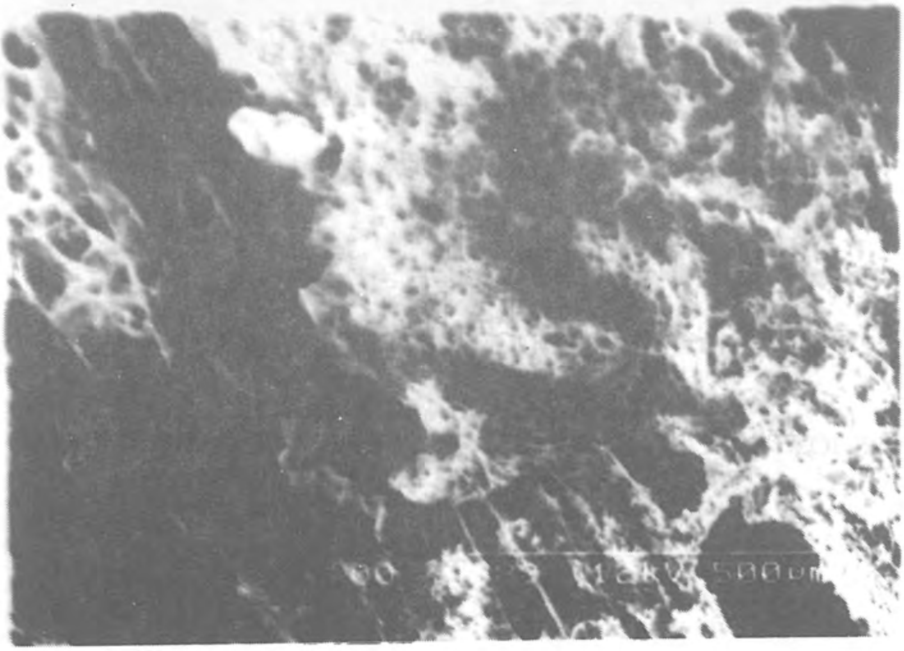

Photo F

FIBRES FROM THE CENTRE OF THE BUNDLE (x100)

TWO ELECTRON-MICROGRAPHS
SHOWING A VERY SEVERE FIBRE SCALING CONTINGENCY

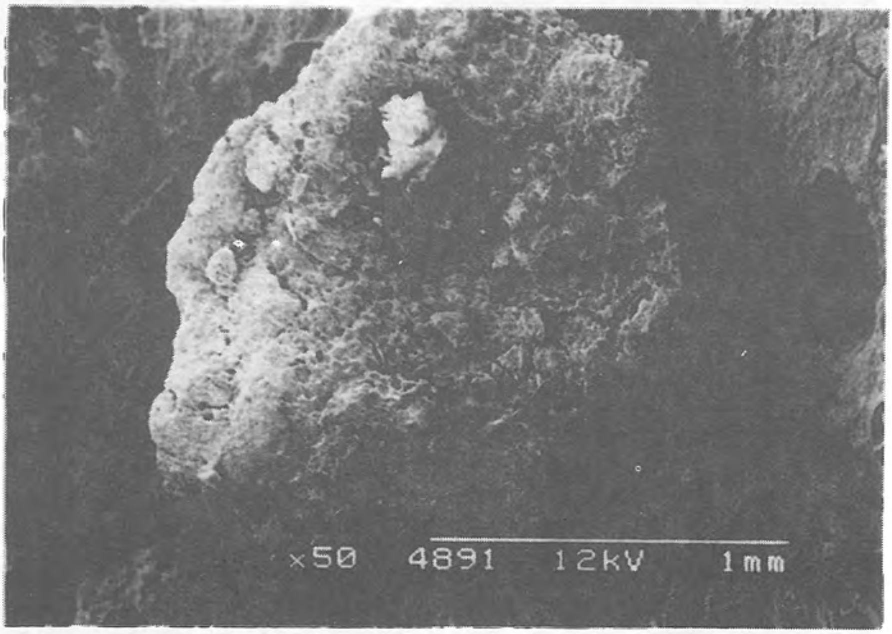

Photo G

SCALE PARTICLE ON REEMAY (x50)

AN ELECTRON-MICROGRAPH
SHOWING A GREY SCALE PARTICLE ON A PIECE OF REEMAY
FROM THE CENTRE OF THE FIBRE BUNDLE

A New Techniques In Cleaning Polyamide Reverse Osmosis Membranes

Irving Moch, J.R. And Ali B. Hamida

A NEW TECHNIQUE IN CLEANING POLYAMIDE REVERSE OSMOSIS MEMBRANES

by

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ABSTRACT

Biological fouling of reverse osmosis (RO) membranes is a major problem existing in open intake plants using brackish or sea feed waters. Much effort has been directed towards its control. No matter how effective these procedures are, water plants must, at some point, be shut down and cleaned to remove the biological debris which has accumulated in the RO devices. This cleaning of the RO membranes has to be done carefully so as not to irreversibly adversely affect the system's ability to produce specification quality and quantity water.

Permasep* Products, DuPont Company has recently developed a new cleaning technique which has proven to be highly successful in use on aramid hollow fiber membranes. The chemistry involves employing sodium hypochlorite ions in a manner such that this chemical's highly oxidative-chlorination powers are harnessed to remove safely biological products from the RO membrane surface. The excellent effectiveness of this procedure has permitted a significant extension of the time between water plant cleaning, thus resulting in a major increase in a plant's availability. This paper describes this cleaning technique in detail, and reviews some of the many Middle East plant sites where the process has been satisfactorily utilized.

Key Words - cleaning reverse osmosis plants, membrane cleaning, chlorine, high pH cleaning, reverse osmosis

INTRODUCTION

In the last several decades, reverse osmosis (RO) has grown from a developmental technology into a very successful commercial process. Many thousand of cubic meters per day of saline water are desalinated into a high quality product. Uses range from portable water supplies to commercial and industrial applications. RO plants are on-stream many hours during the day, achieving high utility rates. Still, essentially all plants must shut down and be periodically cleaned to remove foulants which have collected inside the RO elements. The length of continuous time that a plant operates can be a strong function of the effectiveness of this cleaning, in essence the efficiency of the chemicals employed in this task. Over time adequate compounds have been found to remove some of the many types of foulants which are adversely affecting the RO performance. Such foulants include organics, colloids, metal oxides and various calcium and sulfate precipitants. Removal of organics, however, particularly those resulting from biological activity, has proven to be difficult because of their high adherence or ability to coat and stick to RO membrane surfaces. This paper discusses a novel technique which has been developed to remove organics and biomasses derived from bacteria, algae, etc. from RO devices thus restoring plants to full performance levels.

Development Studies

In approaching the problem of how to remove/clean organics from RO devices certain fundamental issues needed to be addressed:

- The membranes to be cleaned were polyamides which meant that they had limited resistance to oxidative compounds and could be easily chlorinated. Such membrane chemical reactions at normal system temperatures could not be permitted as RO performance had to be improved by the cleanings, i.e. salt rejection and flow improved and element pressure drop decreased.

- The organics coating/sticking to the membranes could come from oily feed waters or be biomasses resulting from uncontrolled biological activity. The treatment has to thoroughly remove these compounds.

- The new process had to permit repetitive cleanings and be successful in improving RO performance over the full life of the membranes. All materials of construction in an RO module had to be unaffected by the new procedure. Thirty to fifty treatments had to be forecasted.

- The cleanings had to be easily done and be cost effective.

With these criteria before us, a number of developmental cleanings were evaluated. A literature search involving both the theoretical and applied articles was undertaken. As a result of these efforts an enhanced cleaning method was developed which involved the use of chlorine (250 mg/l) at high pH (11.8 to 12.0) for contact times of about 30 minutes. Table 1 shows the procedure currently in use in a number of DuPont Permasep* Products B-10 seawater desalination plants located in the Middle East using the Arabian Gulf as feedwater. Key items in this procedure are:

1) At high pH, chlorine will not chlorinate DuPont's aromatic polyamide membrane.

$$Cl_2 + H_2O \rightleftarrows HClO + HCl \qquad (1)$$

$$HClO + OH^- \rightleftarrows ClO^- + H_2O \qquad (2)$$

The dominant species present in high pH chlorinated water is the hypochlorite ion (ClO^-) (Equations 1 and 2). The higher the pH the more ClO^- is present. The upper limit of pH in any RO device is 11.0 for continuous exposure and 12.0 for limited times. The reason for this is that all RO devices contain somewhere in their construction a polyester material of construction. At a pH above 11.0 polyester hydrolysis rate starts to become significant. Tests showed that after almost 3 weeks of continuous exposure to pH 12.0 at 36°C, polyester maintained essentially all its properties. Thus to maintain module integrity the upper pH limit for this cleaning procedure is 12.0. No other material, shell or bundle, in the RO device is affected by this high pH.

At low pH's, for example below 7.5, the dominant species are shown in Equation 1. The hypochlorous acid (HClO) is a very potent disinfectant agent. It, however, also chlorinates the polyamide membrane, ultimately destroying its RO effectiveness.

In laboratory tests we have demonstrated that for long time periods, 12,000 mg/l hours at 35°C and 46,000 mg/l hours at 25°C, the DuPont membrane can withstand exposure to hypochlorite ions at pH 11.8 to 12.0 without showing significant RO membrane deterioration. Figures I and II show the effect of this enhanced cleaning on two of the most important fiber technology parameters - fiber stiffness and its breaking strength. As shown, hypochlorite ion concentration and temperature are key variables.

2) The hypochlorite ion (ClO^-) is an effective oxidizing agent of organics without adversely affecting the DuPont aromatic polyamide membrane. Laboratory tests on biologically fouled membranes using this procedure improved performance in a number of cases over 30%. In one case this procedure improved RO performance another 10+% after the permeator had been disinfected and cleaned using a more conventional cleaning procedure. In all instances permeator pressure drops were restored to as-new condition.

Photomicrographs of fibers taken from permeators cleaned by this procedure showed membrane surfaces free from all debris. The condition of these fibers was equivalent to virgin material. Further the permeator shell, feed tube, and feed and brine ports were free of any organic deposits. These conditions are generally not true following inspections of conventionally cleaned bio-fouled permeators.

3) Care must be exerted to be sure that in removing the high pH chlorinated water from the permeators, the pH of the solution never drops below a pH of about 11. This cautionary comment is required so as to be sure membrane chlorination does not occur. It is for this reason, that the first water flush following the cleaning solution contact is with a plentiful supply of pH 11.8 to 12.0 good quality water. After this purge of the cleaning solution, the system may be safely returned to normal plant pH's.

4) The high pH's seen in this procedure will strip the permeators of their PT-B (tannic acid) and a little of the PT-A (polyvinyl methyl ether) posttreatments. As a consequence, like in all other cleaning procedures, reapplication of these chemicals is required.

5) The short application times of the various chemicals used in this procedure indicates a plant turnaround within 8 hours. The chemicals are common and in low concentrations so cleaning costs are low.

Having satisfactorily demonstrated this new cleaning technique in the laboratory it was now ready to demonstrate the procedure's viability in field applications.

Field Trials

In late 1992 and continuing today, DuPont started implementing this new cleaning technique. The procedure employed is as given in Table 1. The results have been outstanding; RO product flow and quality have shown major improvements. Bundle pressure drops have all been significantly reduced.

The cases below are all sites in the Arabian Gulf. The plants at the time of the cleanings were generally fouled by excessive biological activity. The Gulf sites are shown in this paper because success in this high feedwater TDS-high temperature environment is the most meaningful demonstration of this new procedure than any other locality in the world. Note in Site No. 4, by customer preference, the plant was not posttreated with PT-A and PT-B after the cleaning.

Site No. 1	Bundle Pressure Drop (Bar)	Product Flow (cumh)	Product Quality μS/Cm)
Before Cleaning	0.62	65	1100
After Cleaning	0.31	70	900
Site No. 2			
Before Cleaning	0.94	66.8	3380
After Cleaning	0.38	76.8	1990*
*PT-B only			
Site No. 3			
Before Cleaning	1.66	35.8	2150
After Cleaning	0.59	44.0	2100
Site No. 4			
Before Cleaning	0.72	14.4	2060
After Cleaning	0.21	16.9	3250**
**Without PT-A/PT-B posttreatment			
Site No. 5			
Before Cleaning	0.24	16.9	3250
After Cleaning	0.20	19.4	700
Site No. 6			
Before Cleaning	0.89	64.6	995
After Cleaning	0.41	67.2	920
Site No. 7			
Before Cleaning	1.10	0.72	1059
After Cleaning	0.24	0.74	697

Figures III and IV show the typical rapid drop in bundle pressure drop during thefirst 15-20 minutes of the high pH-sodium hypochlorite solution circulation through the permeators.

Conclusions

A new membrane cleaning method was developed and successfully demonstrated by DuPont in many SWRO plants operating on the Arabian Gulf and in other locations.

Biofouled Permasep* hollow fine fiber permeators can be successfully cleaned by using the new technique. This procedure will result in reduced operating costs, less membrane cleaning and greater plant utility.

Extensive field demonstrations have proven that a 15-20 minute treatment of sodium hypochlorite solution at pH 11.8-12.0 is sufficient to lower significantly the bundle pressure drop, almost to the level of a new permeator.

The new developed short cleaning technique can be applied up to 30 times over the life of the membrane.

Acknowledgements

The authors would like to acknowledge with appreciation the input and assistance of Renee Brighthaupt and Donald Hildabrant in conducting all laboratory tests. We gratefully acknowledge the permission of the management of the Water Production Directorate in Bahrain, the Water Electricity Department in Abu Dhabi and the Ministry of Electricity & Water in Dubai to run the field trials.

TABLE I

SODIUM HYPOCHLORITE, HIGH pH ENHANCED CLEANING PROCEDURE

Hypochlorite, high pH cleaning is a critically important procedure which should be done with maximum care, since, if not performed properly, there can be a negative and irreversible impact on the permeators.

1. Measure the flow of permeators to be cleaned.

2. Flush with low TDS RO quality water.

3. Fill the Circulation Tank with RO product water (TDS less than 1000 mg/l). Adjust pH to 11.8 to 12.0 using NaOH. Circulate the tank to be sure the solution is thoroughly mixed.

4. Circulate the pH 11.8 to 12.0 solution through the RO bank to be cleaned for about 15 minutes to ensure permeators and piping are filled with low TDS water pH 11.8 to 12.0. Divert Circulation Tank contents to drain. It is essentially that the system to be cleaned at pH 11.8 to 12.0 before adding the hypochlorite solution. (May need to add more NaOH during circulation, to maintain pH 11.8 to 12.0 in the cleaning "system").

5. Refill Circulation Tank with RO low TDS water and adjust pH to 11.8 to 12.0 using NaOH. Add sufficient NaOCl to tank so that the concentration throughout the system to be cleaned equals 250 mg/l while maintaining the pH 11.8 to 12.0.

6. Circulate the 250 mg/l NaOCl solution at pH 11.8 to 12.0 through the permeators at 7.0 US gpm/permeator at 150 psig feed pressure for 15-20 minutes. Ensure that bundle delta P is always below 140 kPa (20 psi); measure the hypochlorite concentration and pH about every 5 minutes in the Circulation Tank. If the pH drops below 11.8, stop circulation and add sufficient NaOH to achieve a pH of 11.8 to 12.0, thus insuring that the desired pH range is always maintained. Save the Circulation Tank samples for possible future analytical evaluations.

7. At the end of 15-20 minutes, stop circulation; drain Circulation Tank only; prepare a solution of $NaHSO_3$ at pH 11.8 to 12.0, 2500 mg/l (approximate) in the Circulation Tank. Ensure thorough mixing of this solution in the tank before feeding forward to the permeators.

   It is essential that the solution circulated through the permeators must have a solution concentration of 1500 mg/l $NaHSO_3$ minimum.

8. Circulate solution of 1500 mg/l $NaHSO_3$ at pH 11.8 to 12.0 through permeators for 30 minutes. Check to be sure hypochlorite level is zero. (It is essential that the hypochlorite level be zero before proceeding further).

9. While circulating the $NaHSO_3$ high pH solution, add more $NaHSO_3$ to the Circulation Tank via the Mixing Tank until pH is 7.0.

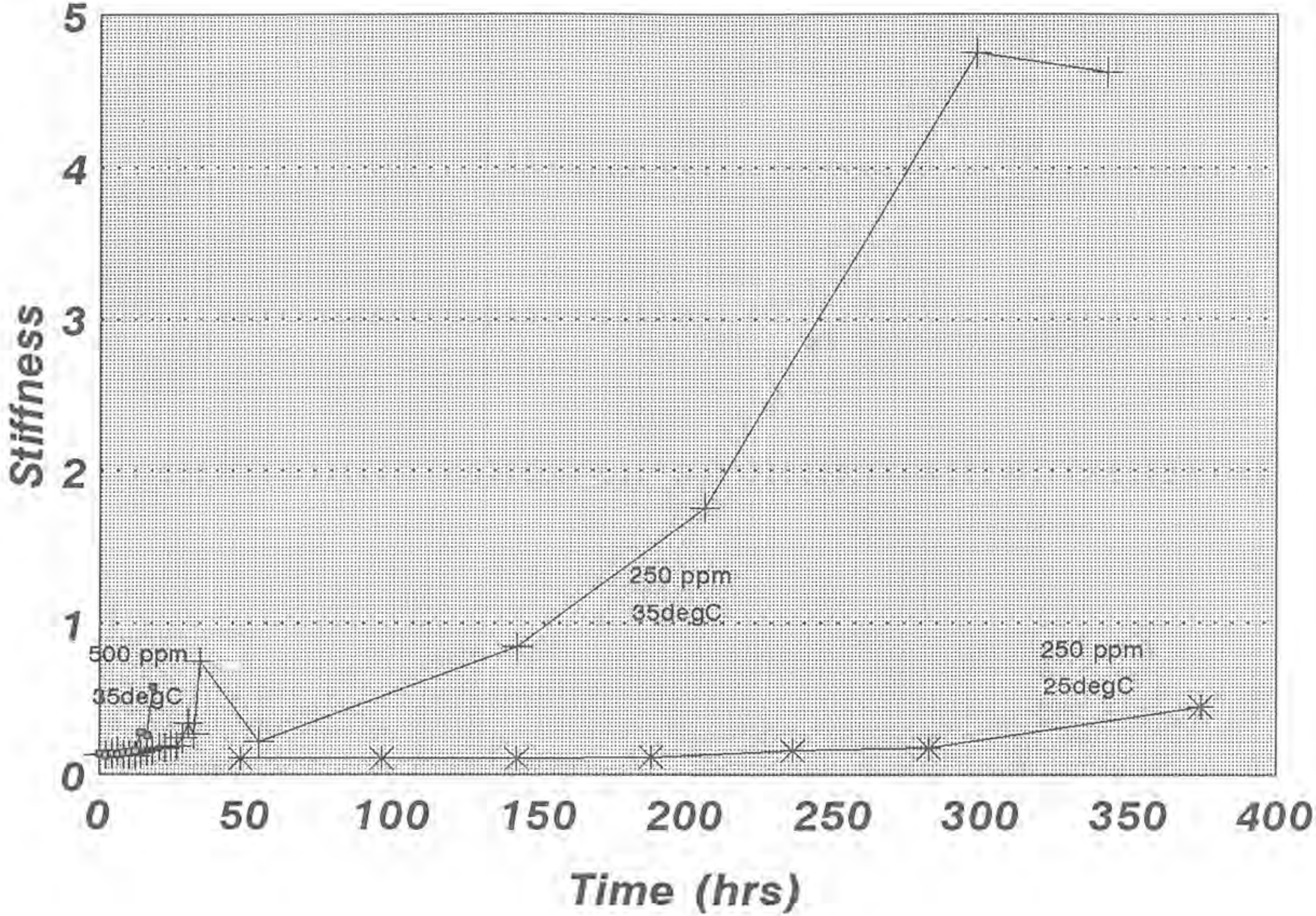
10. Stop circulation and dump the solution.

11. Flush permeators with RO product water (chlorine-free) for 15 minutes.

12. PT-B/PT-A posttreatment in standard manner and measure the flow of the permeators just cleaned.

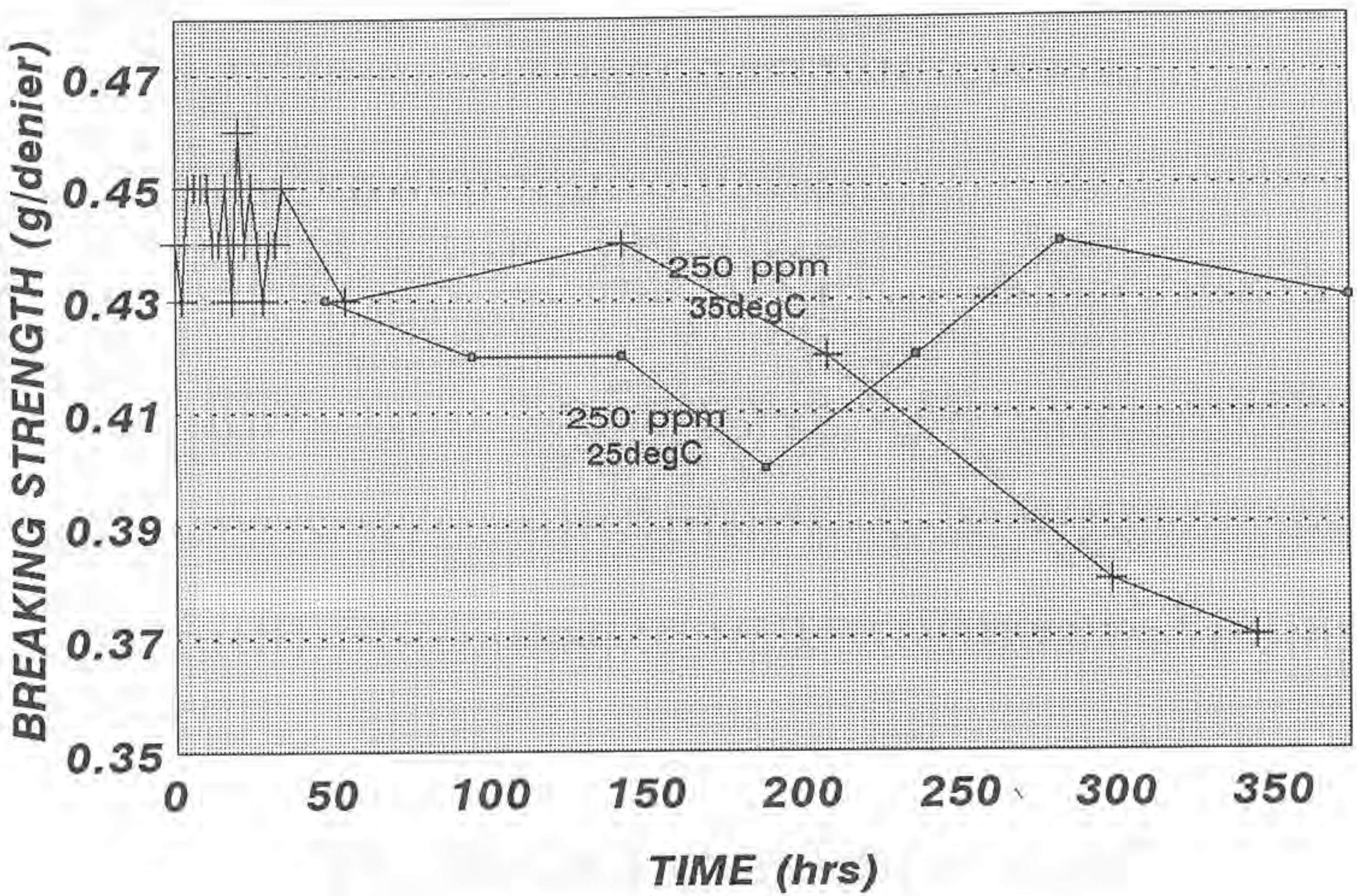
Enhanced Cleaning Tests

FIGURE I



Enhanced Cleaning Tests
Breaking Strength (grams/denier) vs Time (hrs)

FIGURE II



SODIUM HYPOCHLORITE, HIGH PH CLEANING

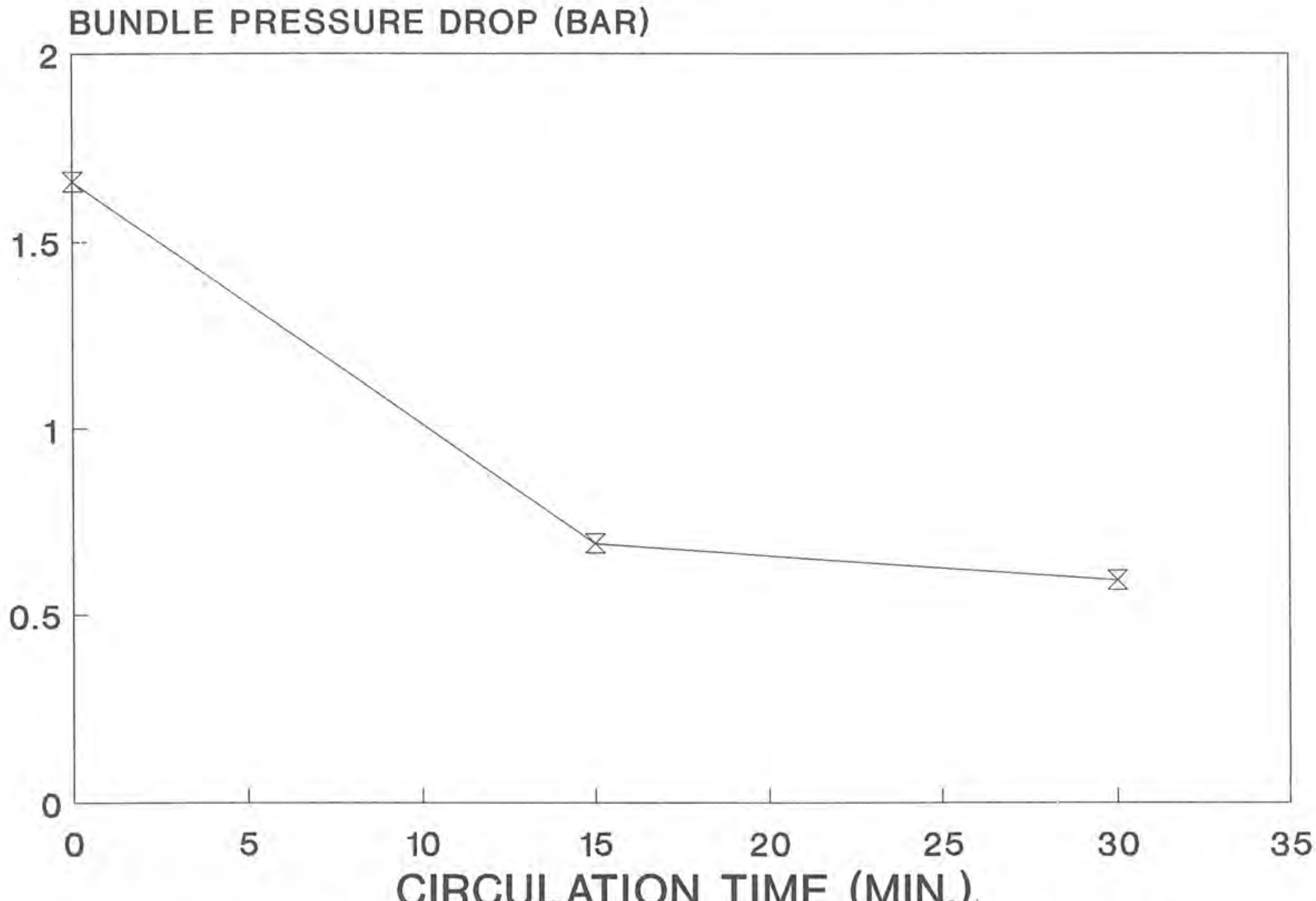


FIGURE III - PRESSURE DROP VS. CIRCULATION TIME

SODIUM HYPOCHLORITE, HIGH PH CLEANING

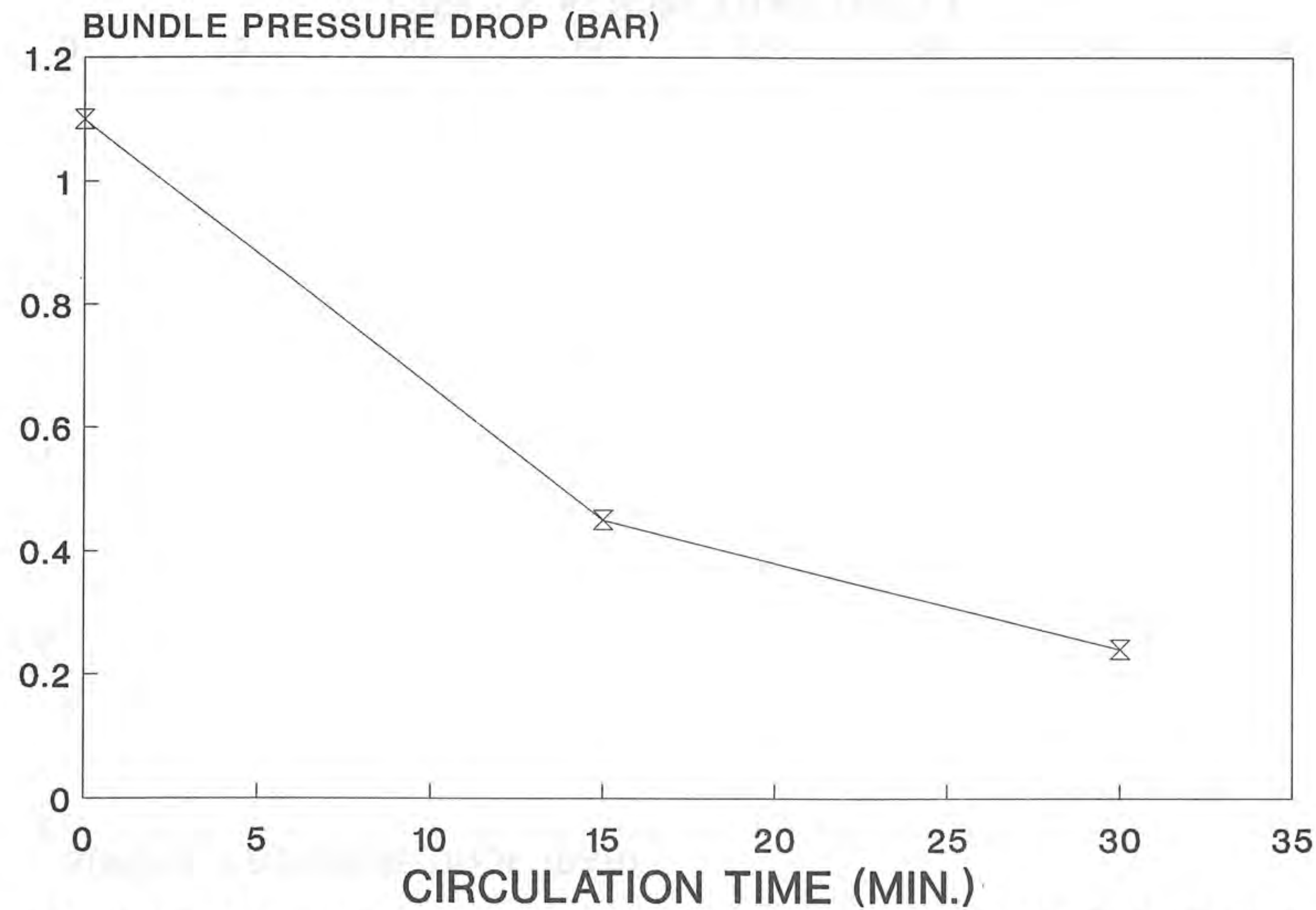


FIGURE IV - PRESSURE DROP VS. CIRCULATION TIME

Performance Evaluation of Six Years Operating Experience of Sea Water R.O. Desalination In Al-Hamara, U.A.E.

Ezzat Zaki Isnasious
Ali Saif Mouded and Ali Ben Hamida

PERFORMANCE EVALUATION OF SIX YEARS OPERATING EXPERIENCE OF SEA WATER R. O. DESALINATION IN AL HAMRA, U.A.E

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ABSTRACT

Significant membrane and system improvements have been made in seawater reverse osmosis (RO) technology during the last decade, which has led to a rapid increase in the use of this technology.

Based on extensive experience by the U.A.E. Armed Forces Engineering Corps in seawater desalination, reverse osmosis has proven to be an efficient and reliable process.

This paper covers the design parameters and the performance of the S.W.R.O System after six years of operation in which the plant has produced over 3.0 million M3 of excellent quality potable water with a very low membrane replacements/additional rate.

INTRODUCTION

In the summer of 1988, the **U.A.E. Armed forces** commissioned its first Sea Water Reverse Osmosis (SWRO) desalination plant, at ***Al Hamra*** area, located in the western region of Abu Dhabi.

The **U.A.E. Armed Forces** have been operating the SWRO system at Al Hamra area in the western region of Abu Dhabi. *mainland* for the last 6 years with a nominal production of 1364 M3/day. The plant was designed to produce potable water of less than 500 mg/l from raw water sources of high salinity sea water ranging from 49000 to 52000 mg/l using B-10 (6840T) permeators manufactured by **DuPont**, ***Permasep®*** **Products.**

PLANT DESCRIPTION

Al Hamra SWRO system consists of series of beach wells as the raw water intake system, adequate filtration & chemical pretreatment to properly condition the water prior to its feed to the membranes, the high pressure pumping system to obtain a feed water pressure of 82.7 bar (1200 psig) and membrane support racks. The product water is then posttreated to a pH of 8.0. A *simplified* schematic process flow is shown in **Figure 1** .

Since the plant location is in proximity to an *oil refinery* where the sea water can be *contaminated* with emulsified oil , and since oil , even in minute levels, can affect the plant performance, the raw water intake system had to be carefully selected to prevent rapid fouling of the membranes and a rapid decline in productivity . The obvious and most economical choice was to rely on natural filtration of oil and other suspended matters by investing in eighteen beach wells (at an average depth of ten meters), drilled a short distance from the shore line . The raw water , drawn from various wells, is then pumped to a primary collection tank at that same location . It is then pumped , through a 2 km , 8 inch pipe , to the settling tanks located in proximity of the R.O. plants , the SDI (Silt Density Index) from beach wells is always between 1.5 - 2.0 .

In the original design, the raw seawater was taken from an off shore intake point by laying PVC pipe over the sea bed. The intake pipe did not remain in tack for a long time due to storm clogging the intake and collapsing the PVC pipe. It was then decided to use natural filtration and draw water through beachwells, this decision was based on extensive studies of the geology nature of the soil.

The pretreatment consists of a sodium bisulfite shock dosing system (disinfection and prevention of bacteriological fouling inside the membranes) operating automatically for 20 minutes every day , a coagulant (Superfloc 573C), a high flow Multi Media filtration system, which reduces the raw water SDI down to 0.5 - 0.8 , an acid dosing system (*Sulfamic Acid)* which helps prevent Calcium Carbonate Scaling and helps maintain the proprietary DuPont *dynamic membrane (PT-B)* . As a final protection to the high pressure pump , and the membranes , the conditioned water is filtered through a 5 μ Cartridge filter before finally being fed to the high pressure pump. For energy conservation , the high pressure pumps are coupled with energy recovery using the Pelton Wheel technology savings amount to 30 % of the required power to the units.

The RO system in each unit consists of 44 B-10 (6840T) membranes in a single stage configuration. All high pressure pipes are made of high grade stainless steel, 316 Stainless Steel and all low pressure pipes are made of PVC. Each unit is provided with product drawback water tank , flushing water tank and cleaning tank .

Chlorine disinfection and pH adjustment are then made to the RO permeate water before it is forwarded to the product water tanks and then to the distribution networks.

PLANT PERFORMANCE

In order to evaluate the performance of the RO system, the operating data was normalized to the following conditions:

♦ Design Period	= 5 years
♦ Feed Temperature	= 30 °C
♦ Feed TDS	= 48000 mg/l
♦ Product Pressure	= 1.5 bar (21.8 psig)
♦ Recovery	= 30%
♦ Feed Pressure	= 77.2 bar (1119 psig)
♦ Number of B-10 Permeators per Unit	= 44
♦ Model Number	= 6840T

A- Productivity

The normalized product flow is shown in Figure 2. The normalized product flow for unit 2 after 6 years of operation was 29.45 M3/H versus 28.4 M3/H design. The normalized flow is thus 3.7% better than design.

B- Product Water Quality

Despite the wide range of feedwater salinity (49000-52000 mg/l) and many shutdowns of the RO system which had resulted in loosing the dynamic membrane (PT-B), the product water TDS has been maintained below 500 mg/l as shown in Figure 3.

THE TOTAL WATER COST

The total water cost of an RO system varies depending on location, system capacity, initial cost, land and construction cost, overall scope of project. In military projects some factors involved in cost calculation are not estimable (such as cost of land), consequently it is difficult to calculate precisely the exact total water cost per gallons per day.

The following study explores the different factors involved in cost evaluations.

1- Capital Cost:

The capital cost of the system can be divided into the following items:

- Cost of the equipment
- Cost of intake system
- Cost of external works

In view of the fact that the cost of land is not included, the total cost of the above mentioned items was 18 millions UAE Dirhams ($ 4.9 millions).

The total capital cost is annualized over a period of 15 years.

2- Operating Cost

About 30% of the total system energy is recovered with energy recovery turbine. Operating costs of chemicals, catridge filters and other consumable are calculated based on actual amount used during the 6 years of operation. Labor cost is divided into two parts, the cost of labor required for maintenance works and the cost of labor required for plant operation; plant operation is carried out by Armed Forces personnel. Membrane life was found to be more than expected. During the last 6 years of operation, only 22 membranes (out of 88) in both units were replaced/added making the replacement rate per year less than 5.0%.

3-Average Total Water Cost

a- Capital cost of the units, intake system, external works including maintenance
and spare parts in the first year of operation = $ 4,891,304
Capital cost/year = $326,086
The above annual ammortization amount does not include any interest charges.

b- Average cost of maintenance works including supply of chemicals, spare parts, membrane replacement, consumables, etc. in the first 2 years after maintenance period = $369,793
Average cost of maintenance works including supply of chemicals, spare parts, consumables, etc. in the next 2 years of operation = $ 710,138
Cost of maintenance works including supply of chemicals, spare parts, membrane replacement consumables, etc. till the end of the contract maintenance in May 1st, 1994= $ 207,124
Average total maintenance cost per year = $214,509

c- Energy Cost

Absorbed power	= 9744 kW per day
\$0.03/kWh , then 9744 x 0.03	= \$ 292.32 per day
	= \$ 96,027 per year

d- Labor Cost Per Month

Number of operators	= 6
Total Cost of operators	= \$ 1223 x 6 = \$ 7338
Total labor cost per year	= \$ 88,056

e- Summary of Annual Water Production Cost

	US\$	**U.A.E. DHS (000)**
Capital Cost (excluding interest charges)	326,086	1,199.5
Average Cost of Maintenance (including supply of chemicals, spare parts, membrane replacement, consumables, etc.)	214,509	789.1
Energy Cost (\$0.03/kWh)	96,072	353.4
Labor Cost	88,056	323.9
	724,723	2,665.9
Per 1000 US Gallons	6.13	22.55
Per M3	1.62	5.95

The above total cost was based on significantly higher capital cost of the plant, however with toady's RO technological improvements, the capital is about 30-40% less which will make the total water cost substantially lower.

CONCLUSIONS

Al Hamra seawater RO desalination plant performance has been very successful. The plant output has exceeded the projected and guaranteed values with an average permeator replacement /addition rate of less than 5.0% per year. Al Hamra SWRO plant produced over 3.0 million cubic meters of excellent quality product water.

Plant availability has been very satisfactory averaging about 90%.

The outstanding performance of Al Hamra plant has given the U.A.E. Armed Forces great confidence in the RO technology as an efficient and reliable process for seawater desalination.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the advise and co-operation of **MAJOR AHMED SAEED AL HAM AL DAHIRI**, Head of Design Department - Directorate of Military Works.

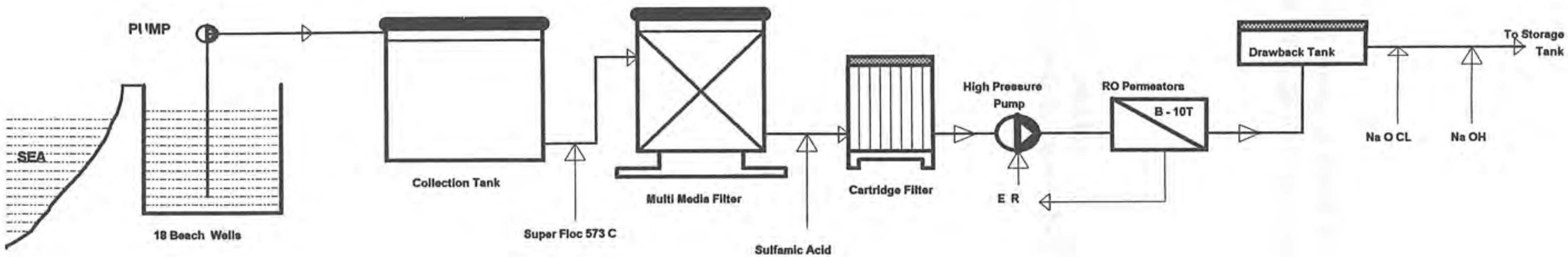

FIGURE 1 - PROCESS FLOW DIAGRAM

AL HAMRAH CAMP SWRO PLANT
UNIT 2

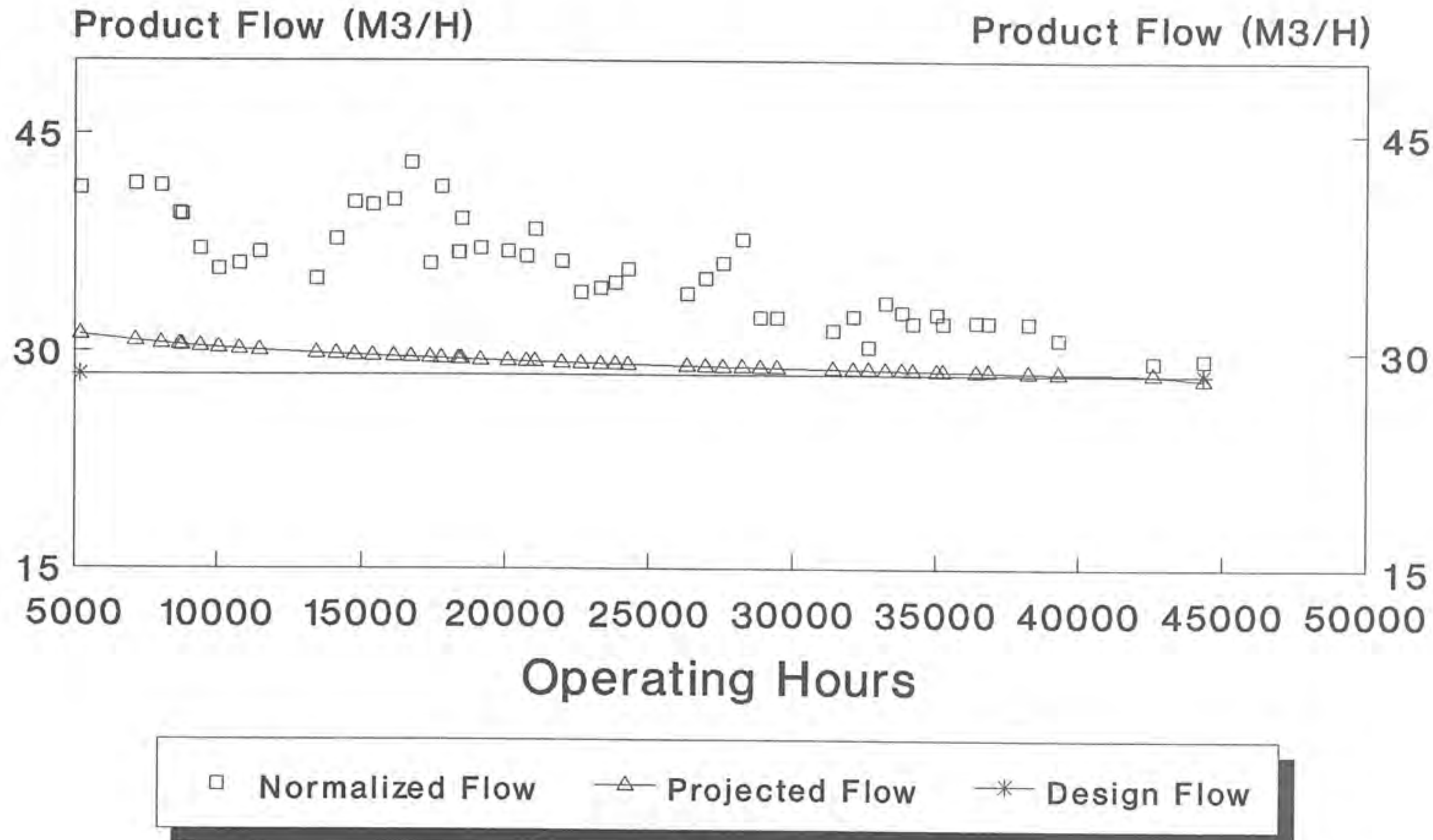

Figure 2 - Normalized Flow

AL HAMRAH CAMP SWRO PLANT

UNIT 2

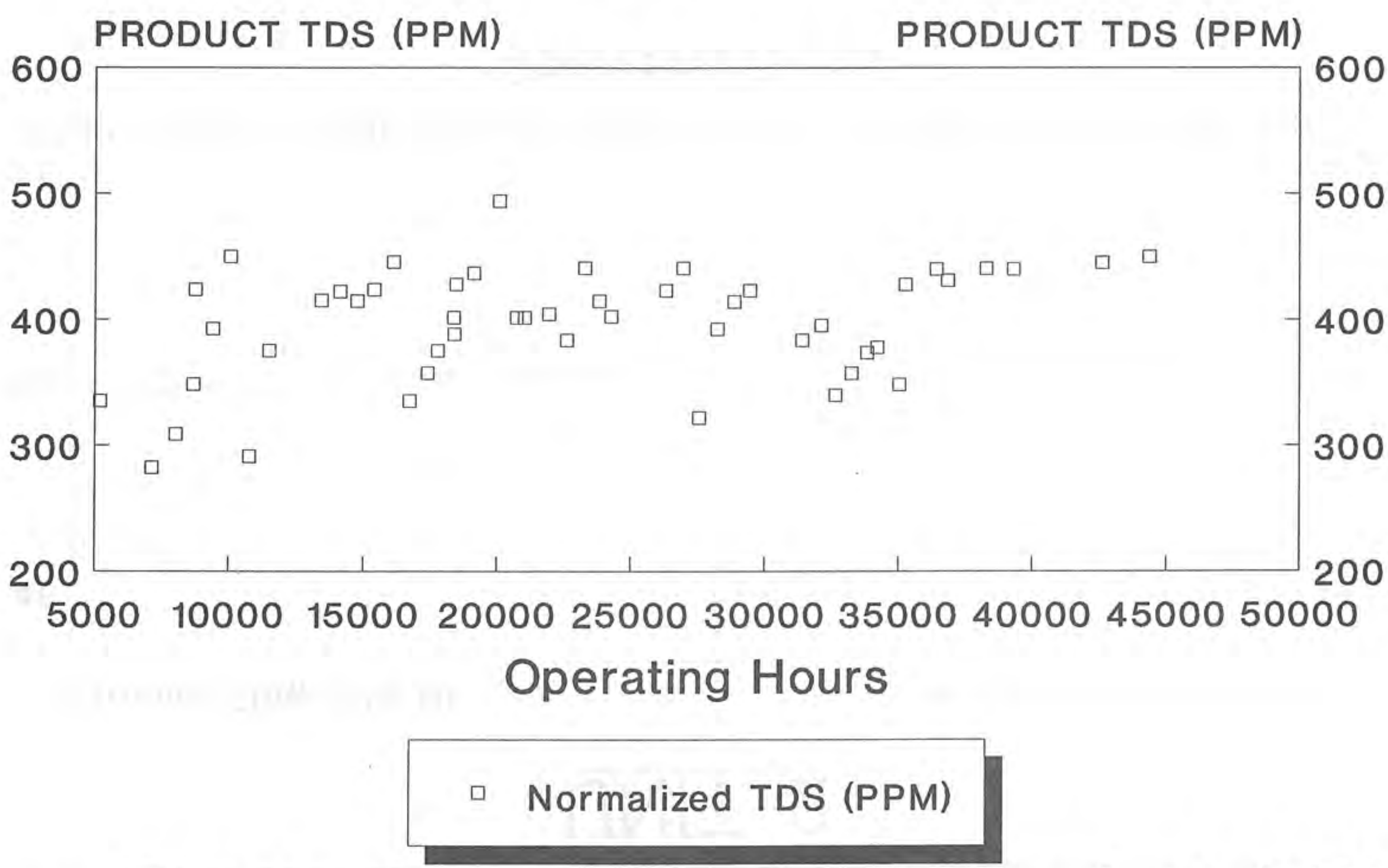

Figure 3 - Normalized Product TDS

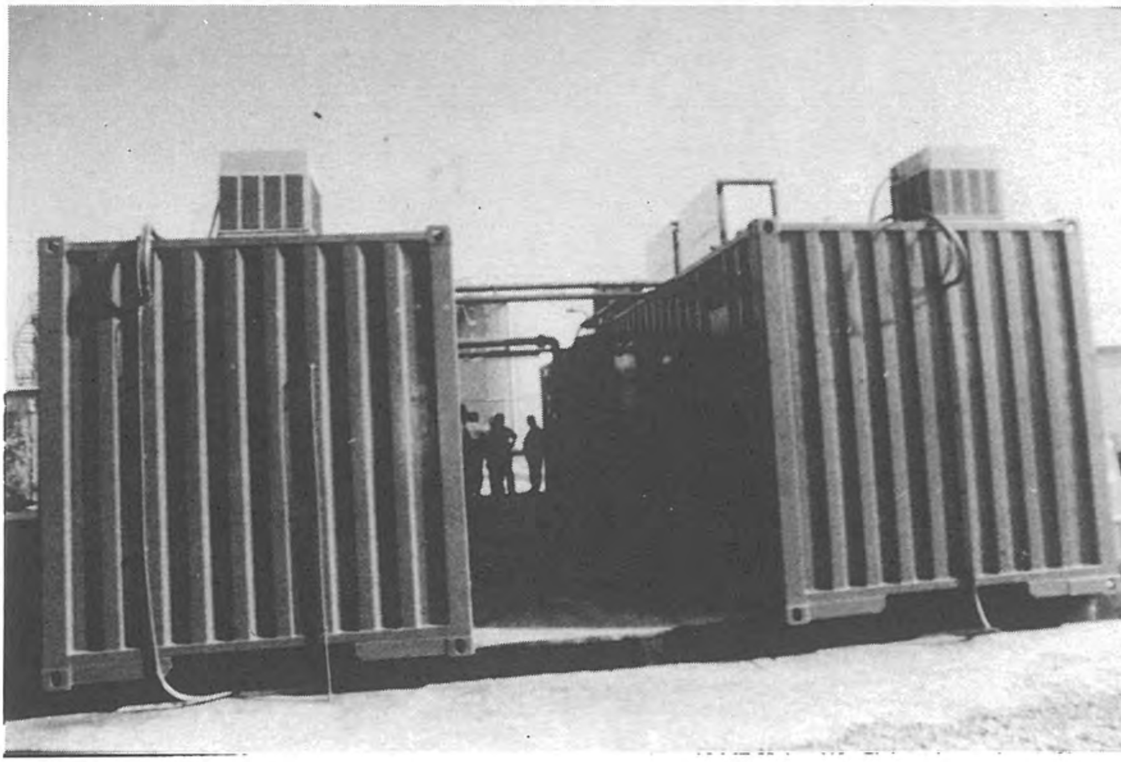

Figure 4 - General View

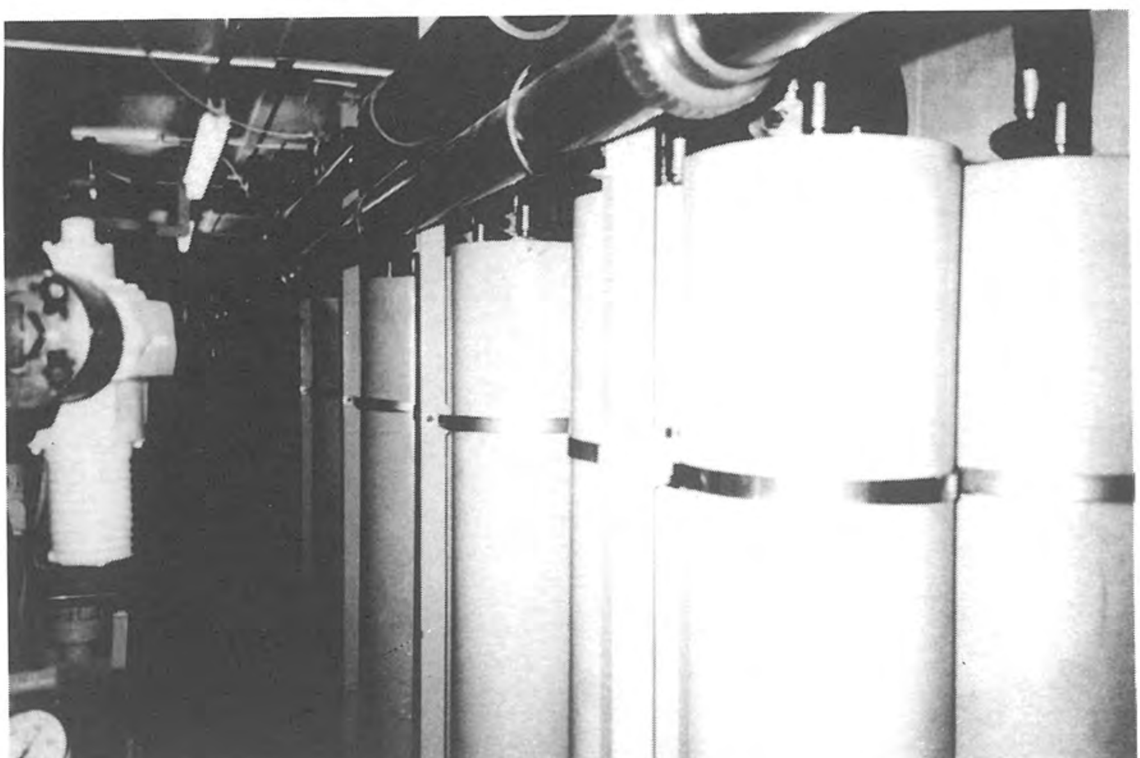

Figure 5 - B-10 Permasep ® Permeators

Figure 6 - Beachwells

The B-10 Twin Permeator

Ali Ben Hamida and Kamran Chida

THE B-10 TWIN™ PERMEATOR

by

Ali Ben Hamida
Technical Representative

&

Kamran Chida
Sr. Process Consultant

Permasep* Products
E.I. DuPont De Nemours Co., Inc.
Wilmington,De

ABSTRACT

In 1991, the DuPont Company commercialized its high capacity advanced permeator "B-10 Twin*for seawater and high brackish water RO desalination. Several installations using this technology, in the Middle East, Europe and the Americas have demonstrated the superior performance of the B-10 Twin* Permeator.

This paper discusses the design features of the B-10 Twin* permeators and the operating experience in several seawater RO plants

The improvements in the B-10 Twin* permeators have led to capital cost decreases and operation improvements. Such as;

* Improved tolerance to pretreatment upsets
* Reduced frequency of cleaning
* Improved overall plant availability
* Easier permeator troubleshooting

The B-10 Twin* has proved its reliability, superior salt rejection and stable production rate.

Nomenclature: Twin* = Twin™, B-10 & B-10T are type of DuPont membrane fibers.
Y= Recovery, T= Temperature, F= Feed water, P= Feed pressure
S= Salinity,

INTRODUCTION

DuPont recognised 25 years back, Desalination is one of the fastest growing technologies throughout the world and based on its specialised technology of polymer chemistry, invented and developed the hollow fine fiber and the single bundle permeator which became standards in the field for the production of high quality desalted water.

It is a well known fact that RO membranes are one of the most critical and vital component of a reverse osmosis plant. Though the RO membrane's share in the overall plant cost may be between 15-20%, it's reliable and satisfactory performance is key to the success of the reverse osmosis plant.

The current development uses the established second-generation sea water membrane, based on the same B-10 aramid polymer. This "B-10T" HFF membrane, introduced in 1984, has demonstrated long life with excellent salt rejection and has brought significant improvements to RO. By operating at 1200 psig (8273 kPa), a 20% higher pressure than its predecessor, B-10 membrane:

* increased membrane productivity, thus reducing membrane costs;
* increased salt rejection, enabling more efficient RO plant designs on the higher salinity Arabian Gulf, Red Sea and elsewhere;
* allowed operation at higher conversions, providing additional economic benefits;

The development programme addressed user goals for reduced capital and operating costs of large-scale RO plants, and for improved reliability of RO systems, especially with surface sea water feeds, DuPont tackled these goals by:

* Inventing a lower cost, double-bundle RO device with an innovative flow pattern;
* Redesigning the single,"B-10T" HFF membrane bundle to improve its performance
* Developing a compact, lightweight pressure vessel and assembly;

Understanding the working of Twin* Permeator

To understand the flow pattern and working of a Hollow fiber device, it is important to understand its construction which resembles a typical shell and tube heat exchanger. Millions of fibres are oriented in parallel and fixed in epoxy at both ends.

The fibres at one end of the bundle are precisely cut so that the product water can be discharged from the open bores of the fibres. The encasement of the fibres in epoxy at both ends gives the bundle mechanical stability. Typical example of flow pattern and the general construction of B-10 TWIN* hollow fiber (HF) Permeator is indicated in figure 1, 2 & 3.

Under pressure, when water is fed into the central distributor tube, it is forced out radially through the first bundle of fibres. the remaining portion of the feedwater travels through the interconnector to the feed tube of the second bundle. This feed tube contains, internally, a concentric brine tube. in the second bundle, like the first bundle, feedwater under pressure is forced out radially through the bundle of fibres. As pressurised feedwater contacts the outer surface of the hollow fibres, reverse osmosis occurs and permeate is forced through the centre of each hollow fiber and moves along towards the open end of the bore towards a collector (commonly known as tubesheet) from where it is sent out from both ends of the bundle into the permeate lines outside the device.

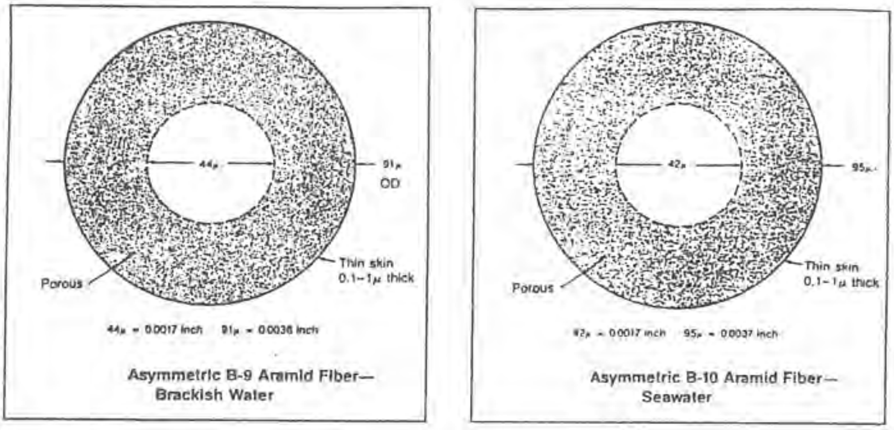

Figure 1 - Asymmetric "aramid" Polymer Hollow Fiber illustration

As permeation occurs due to RO, the brine generated is forced outside of each bundle, moves past the nub end of each bundle to the centre of the permeator into the interconnector opening. From there, the brine exits the Permeator through the concentric brine tube and is discharged into the brine manifold. O-Ring seals used between the feed, product and the brine sections prevent mixing. Thus, the feed brine flow pattern in the Twin* is similar to the single bundle hollow fiber permeator*, from the centre of the bundle to the periphery. This flow direction is very important to prevent nesting (Compaction) of the fibres which causes high pressure drop across the bundle resulting in reduced product flow and salt rejection.

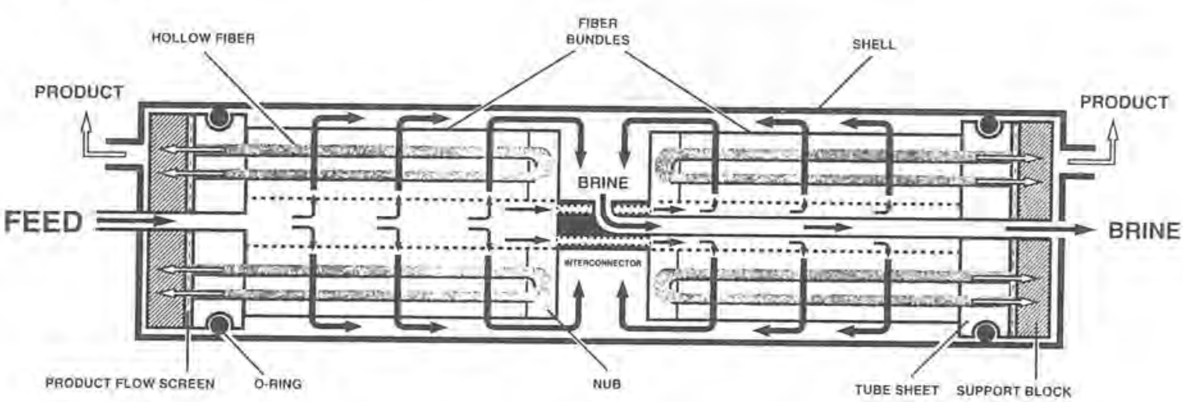

figure 2- B-10 Twin™ Permeator flow pattern

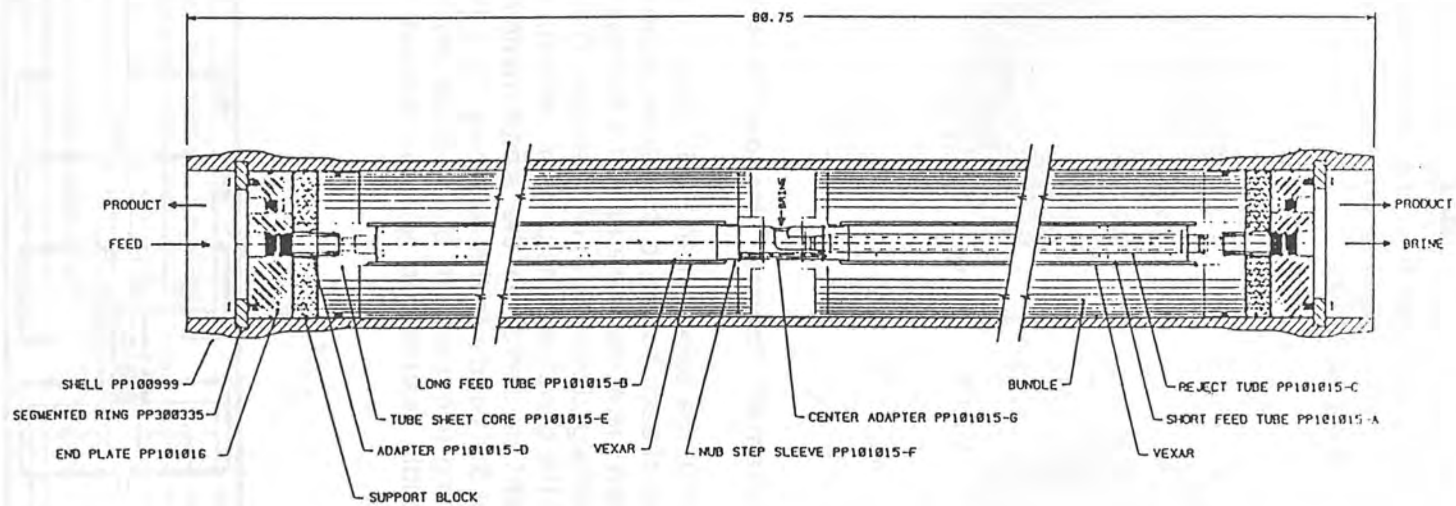

figure 3- B-10 Twin Permeator

Twin* Bundle Device Concepts

Double-bundle permeator design for a flow rate at a given feed pressure, significantly reduces hardware costs compared with single-bundle devices. A double-bundle design requires half as many vessel end closures and retaining rings per gallon of water produced. It also halves the number of high-pressure stainless steel fittings and support racks needed.

Design features and considerations:

In creating the Twin* device, a number of design concepts were evaluated. The key features which contributed in the final development of the B-10 & B-9 Twin* are discussed;

1) Effect of flow pattern:

Series staged twin (figure 4) and a parallel staged twin (figure 5) were modelled, prototypes developed and extensive tests were conducted to determine the optimal arrangement of the RO bundles configuration.

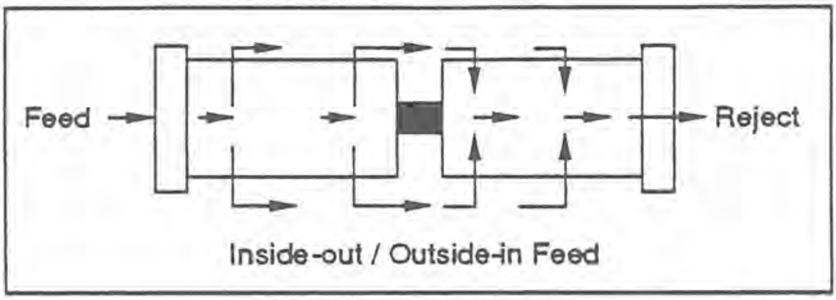

figure 4 - Series Staged Twin arrangement

In the series staged twin, feed in the first bundle flows radially from the inside of the bundle toward its outside, while feed in the second bundle contracts radially from the outside toward the inside. In the parallel staged twin, both bundles are fed from the inside to the outside.

Tests were performed to determine the relative performance of the outside-in and inside-out feed flow designs. Based on extensive testing carried out, at every value of percent conversion, the inside-out design demonstrated both higher flow and improved permeate quality as measured by lower conductivity.

To achieve inside-out flow in the new Twin* arrangement, a novel flow configuration was developed (figure 3) that enables the feed entering the centre of the both bundles from one end of the vessel to flow radially outward through each bundle. The concentrated brine flows between the outside of each bundle and the shell inner wall, collects in the space between the bundles, then flows through a centre interconnector and concentric brine tube out the opposite end of the vessel.

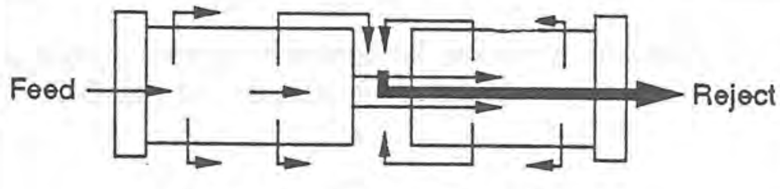

figure 5 - Parallel Staged Twin arrangement

The permeate travels down the hollow fibres and exits from both ends of the device.
Because the concentric brine tube in the inside-out device experiences low differential pressures, it can be fabricated from low-cost materials.

In addition, inside-out feed flow, which has been used successfully for over 23 years, stabilizes long-term performance by minimizing the nesting of fibres and any resulting increase in pressure drop across the bundle.

2) Fiber Length and Bundle flow Optimization

The " B-10T" bundle uses millions of tough, aramid hollow fibres oriented in parallel and mechanically anchored at both ends. Feed pressure forces permeate to the centre of each hollow fiber. The permeate moves along the hollow bore to the discharge point at the face of the tubesheet.

Average pressure drop associated with flow of the water down this bore was modelled, from closed end to open end. The pressure drop were used to calculate the net driving force across the membrane for various fiber active lengths. We then used values of net driving force to compute the permeate flows for single-bundle and Twin* designs were used.

The calculated permeate flows, plotted against fiber length with packing density held constant, for the single-bundle and Twin* devices. In both cases, permeate flow reaches a maximum value as active length increases, suggesting a preferred approach for optimization of device performance and cost. Shortening the length lowers costs, reduces salt passage, increases cross-flow velocity at a given conversion rate, and yields more uniform conversion along the length of the fiber.

3) Fiber spacing & packing density:

The relationship between packing density (cross-sectional area of the fibres divided by the area available) and fiber spacing for fibres having a circular cross section, suggesting further possibilities for optimization. Selection was based on a balance between a slightly higher packing density and an average fiber spacing of about 25 micron (0.001 in.), to allow the passage of the smaller colloidal particles that pass through a typical 5-10 micron (0.0002 - in.) cartridge filter.

To confirm the accuracy of the model and flow calculations, single-bundle permeators were fabricated and tested using a range of active lengths and packing densities. Table-1 shows that the model accurately predicts performance of the devices.

Table 1-Evaluating the Predictive Ability of the Flow Model

Active Length, inches	Packing Density, %	Experimental Flow, gpd	Predicted Flow, gpd	Surface area, sq.ft.
20	53	6300	6200	3900
26	53	7050	6900	4850
33	49	7000	6930	5750

4) Cost optimization:

Manufacturing costs of single bundle of Twin* devices, are influenced by fiber length at our standard test conditions. Based on test data, it was noted that both devices show optimal cost-performance at about the same fiber length, about 17.8 cm (7 inches) shorter active length yields a 22-percent increase in brine velocity compared with the standard device. The resulting combination of higher cross-flow velocity and more uniform conversion allows better performance at higher conversions.

The above hypothesis were evaluated by testing several compact and long-bundle permeators. Permeate conductivity obtained at conversion levels ranging from 20 to 45 percent proved that the compact bundle:

* *Yields a higher quality permeate at all levels of conversion*
* *Performs better at higher conversions.*

Based on the above, Twin* permeators were fabricated and tested and the results confirmed the improvement in performance. These results offer benefits for many applications, permitting operation at higher conversion levels, decreasing energy consumption and reducing the cost of surface sea water pretreatment equipment and consumption of chemicals.

5) Improved web for added robustness, cleanability & fouling resistance

The web used in DuPont permeators stabilizes the fiber array and ensures the uniformity of packing density and fiber spacing. It prevents channelling or short circuiting of the feed flow through the device. To improve the performance of the bundle during operation, we replaced the previous web material with a more open, woven monofilament mesh having regular openings between 50 microns (0.002 in.) to 150 microns (0.006 in.).

This web provides better flow distribution around the fibres and is less likely to trap materials which could foul the permeator and provide a source for biological growth, thus reducing the need for cleaning.

The new web is eight to ten times stronger than the earlier version and enables operation at higher pressure drops. The stronger web also improves the robustness and cleanability of the bundle under extreme conditions. These properties were evaluated by intentionally fouling a bundle with ferric chloride until the bundle's pressure drop exceeded 100 psi (689 kPa). After cleaning with citric acid, the performance of the bundle was completely restored to its original levels. Physical examination of the bundle after testing showed the web intact and dimensionally stable.

6) Device design and improvement:

Materials of construction for the Twin* were selected to minimize corrosion and withstand the extremes of pH and salt concentration encountered in sea water desalination. All parts (Hardware) were analysed using both finite element analysis and stress calculations, and were found to be acceptable for long-duration service (15-20 years).

All the Twin* parts were sized to minimize the resistance of flow through the device. Systems using the Twin* design can be run either single-staged or reject-staged with acceptable pressure drops.

Lighter and stronger pressure vessel for the Twin* permeator was developed using state-of-the-art technology. Prototype shells were cycle-tested 100,000 times from 0 to 8273 kPa (0-1200 psi) and showed a burst pressure greater than 6x maximum service pressure. The shell has been designed for easier end-plate and bundle removal and requires only 66% as much space as two single-bundle permeators with the same capacity.

The Twin* weighs 40% less than comparable single-bundle devices, resulting in a savings in racks and supporting materials.

Field Demonstrations

The product development reported here was very successful in translating computer modelling techniques into physical realities. To demonstrate the efficiency of "compact" bundles and Twin* permeators in field tests with a variety of sea waters. Two approaches were utilized. The first involved testing the new compact single bundles alone and examining their performance characteristics separate from the hardware durability issues. This was accomplished through side-by-side comparisons with the longer bundle of standard B-10 units. The second series of demonstrations involved the complete Twin* permeators operating in a variety of field tests and market development applications. Most of the field sites described below are still in operation. In some instances the product has accumulated nearly three years of on-line testing hours.

I) Single short bundles were tested at several sites including Bahrain, Grand Cayman and Canary islands. At all sites the performance exceeded expectations and was satisfactory.

II) B-10 Twin* Permeators were have been tested at several sites in the middle east and grand Cayman. Commercial plants using the B-10 twin* have been operational in the middle east;

a) **1325 cu.m/d, Das Island, UAE- started 1991**, [S = 44,000 ppm, F = open sea, Y = 35%, T = 30-35C(86-95F), P=7240 Kpa(1060psig)] The plant has operated for over 12000 hours and the first cleaning was performed after 7000 hours of operation.Performance of this single stage seawater RO plant is better than expected with stable permeate TDS averaging <400 ppm.

b) 1000 M3/d Ras Lafan, Qatar- started 1992, [S = 41,000 ppm , F = open sea, Y = 35%, T = (32), P = 7584Kpa (1100psig)].This plant was built as a temporary facility to provide potable water during construction of a new gigantic project. It is designed with minimal pretreatment. First cleaning was done after 5100 hours and the plant has operated for over 10000 hours with only one cleaning.

c) Abhur, Saudi Arabia [S = 41,000-45000ppm, F = open sea, Y = 35%, T = 22-35 deg.C, P = 900-1200psig]. Two B-10 Twin* have been under evaluation for over 18 months. Performance has been very satisfactory and only one cleaning has been done after about 7 months (5000+ hours).

d)Several small commercial plants with 15-20 B-10 Twin* membranes on the Red Sea and in the Gulf have also been operating for over a year and their qualitative as well as quantative performance is satisfactory.

e)Grand Cayman [S = 36,000 ppm, F = Beachwell water, Y = 45-50%, T = 82 (28), P = 8273KPa(1200psig)]. Testing started in 1990, Twin* permeators are 1:1 staged to achieve high conversions. Performance has mirrored .he compact, single-bundle permeators. Some units have operated for 14,000 hours with no requirement of cleanings.

f) Most recently, In the middle east, DuPont received a subcontract for supply of B-10 Twin* permeators to a 91,000 Cu.M /day (24 US MGD) single-pass RO plant at Al-Jubail, Saudi Arabia, now being built by a German consortium. Also, several smaller contracts have been awarded in Saudi region totalling about 13200 M3/day (5 US MGD) using the the new B-10 Twin*Permeators.

To summarize, these continuing field demonstrations have proven that the flow and salt passage of the compact bundle and the B-10 Twin* have performed as well as predicted from our computer modelling. In fact, the permeate quality appears somewhat superior to early predictions. Judging from the infrequent need for permeator cleaning, the 22% linear velocity increase of the compact bundles over the long bundles, along with other changes, appears decisive in reducing permeator fouling. Structural analysis and long-term system performance under a number of different operating conditions, including numerous plant start-ups and shutdowns, have demonstrated the mechanical integrity of the B-10 Twin* internal parts.

Summary and Conclusions

In continuation of our commitment to the RO industry, DuPont conceived, developed and successfully tested a new double-bundle RO device for sea water desalination. In-house testing, field trials and scaled-up manufacturing runs;

* Confirmed the predictions of computer modelling.
* Confirmed the robustness and mechanical integrity of the new device, and
* Met or exceeded production targets for improved flow and salt rejection.

Excellent progress was made towards long-term goals for lower RO plant costs and further improved reliability of large RO systems. Capital costs were reduced by:

* Providing a lower cost device per GPD of capacity;
* Reducing the number of expensive, high-pressure fittings, and
* Reducing RO system space and associated costs.

The programme also confirmed the potential for significantly reduced operating costs by:

* Demonstrating improved HFF bundle performance capabilities over a wide range of conversions and feed salinities.
* Reducing permeator fouling and cleaning frequency, thereby lowering maintenance labour and chemical handling and improving plant utility, and
* Permitting stronger membrane performance guarantees with expanded operating guidelines for surface water feeds.

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Water Supply Infrastructure Replacement, Awali, Bahrain

Nicolas Merrick

WATER SUPPLY INFRASTRUCTURE REPLACEMENT, AWALI, BAHRAIN

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Abstract

The town of Awali, meaning 'high place', developed in parallel with the Bahrain Petroleum Company (Bapco) and can trace its origins back to 1935 following the discovery of oil in Bahrain in 1932.

Since that time the infrastructure has developed and been modified in a variety of ways, the principal activities having taken place between 1935 and 1956. If Awali was to serve as a viable town for many years to come, then there was a clear need to upgrade many of its water related services.

The necessary development studies eventually led to the replacement of the groundwater abstraction facility, raw water delivery, water treatment, additional sweet water storage and distribution : i.e. the replacement of the sweet water infrastructure.

Key words

Abstraction, distribution, infrastructure, reverse osmosis, SCADA, storage.

Introduction

Those elements which were subject to study and eventual replacement are discussed in the following text. They are generally considered in process sequence commencing at abstraction and terminating with the distribution and control of the systems. Whilst this was not strictly the actual sequence in which the work was carried out, the author considers this to be the most appropriate form for discussion and this presentation.

Storage, which affects both raw and treated water, is considered as a single entity after the desalination stage. The generic terms of sweet and brackish water are applied to the potable and raw (aquifer B) waters.

Key features which required evaluation and inclusion in the works were :-

- Requirement for a highly reliable water supply system. There are no plans for sweet water to be supplied to Awali in the short or intermediate terms from sources outside the township.
- Previously, domestic and office consumers were supplied with both sweet and brackish water inside the properties. This was to change to 'full sweet water' and this single revision to the demand characteristics would itself generate a significant additional demand. Irrigation and fire fighting would continue as brackish water users.
- Power supply was to move away from the historical dependence upon the Bapco refinery at 110V, 60Hz system to meet the national standard of 220V, 50 Hz.

- Disposal of waste would need to be carried out in an acceptable and environmentally friendly way.
- The source water for the desalination plants, aquifer B within the Khobar formation at Zallaq, was deteriorating at an increasing rate. The primary indicator, the Total Dissolved Solids (TDS) had demonstrably increased within a short time period and will continue to do so.
- Conversion to metric units of measurement, in accordance with national standards.

The key features of the new works are illustrated on Figure 1.

Previous infrastructure

Aquifer B groundwater was abstracted from Zallaq using four shallow wells. It was boosted at surface level for delivery along three separate transmission lines of varying age but common poor condition, to Awali where it entered storage in three above ground steel tanks. Two of these tanks formed the supply to the irrigation and the majority of domestic water requirements in the houses and offices in Awali. The third tank formed the dedicated supply to the electrodialysis reversal plant. The waste from the plant was recirculated into the irrigation water and the product entered storage in two small above ground tanks. These feed Awali and a small supply to the Jebel for the Bahrain National Oil Company (Banoco). The brackish water distribution system had been replaced in 1986 but the separate system for supply of sweet water was old and prone to failures in terms of bursts and pressure drops.

Groundwater abstraction

The condition of the four wells drilled between 1935 and 1956 in Bapco's Zallaq well field had caused concern for some time. The most westerly, some 25 m from high tide level, had experienced hydrocarbon intrusion of unknown origin and was decommissioned in 1990. This left three ageing wells to serve the supply to Awali, and also a small brackish demand to the Jebel for Banoco.

Groundwater was lifted from the wells to the surface where it was boosted by one electrically driven duty pump with two diesel driven standby pump sets to Awali. The pumping station generated excessive noise and was a source of frequent maintenance. In addition, original or cost effective spare parts were particularly difficult to locate.

Preliminary studies on the wells and methods of renovation concluded that the cost of providing conclusive data with , based on visual surface inspection, the likelihood that the wells would require replacement, was not justified. Corrosion was readily apparent at the surface and would be present at depth. The more straightforward closed circuit television (CCTV) and calliper logs would provide information of limited benefit. More extensive investigations, such as gamma-log and electromagnetic casing inspection were costly to carry out and difficult to interpret.

Four additional wells were therefore drilled by the Water Resources Directorate (WRD) of the Ministry of Commerce and Agriculture. Well depth was typically 38 m, with the last 10m being 250 mm diameter open hole in the aquifer B Khobar limestone.

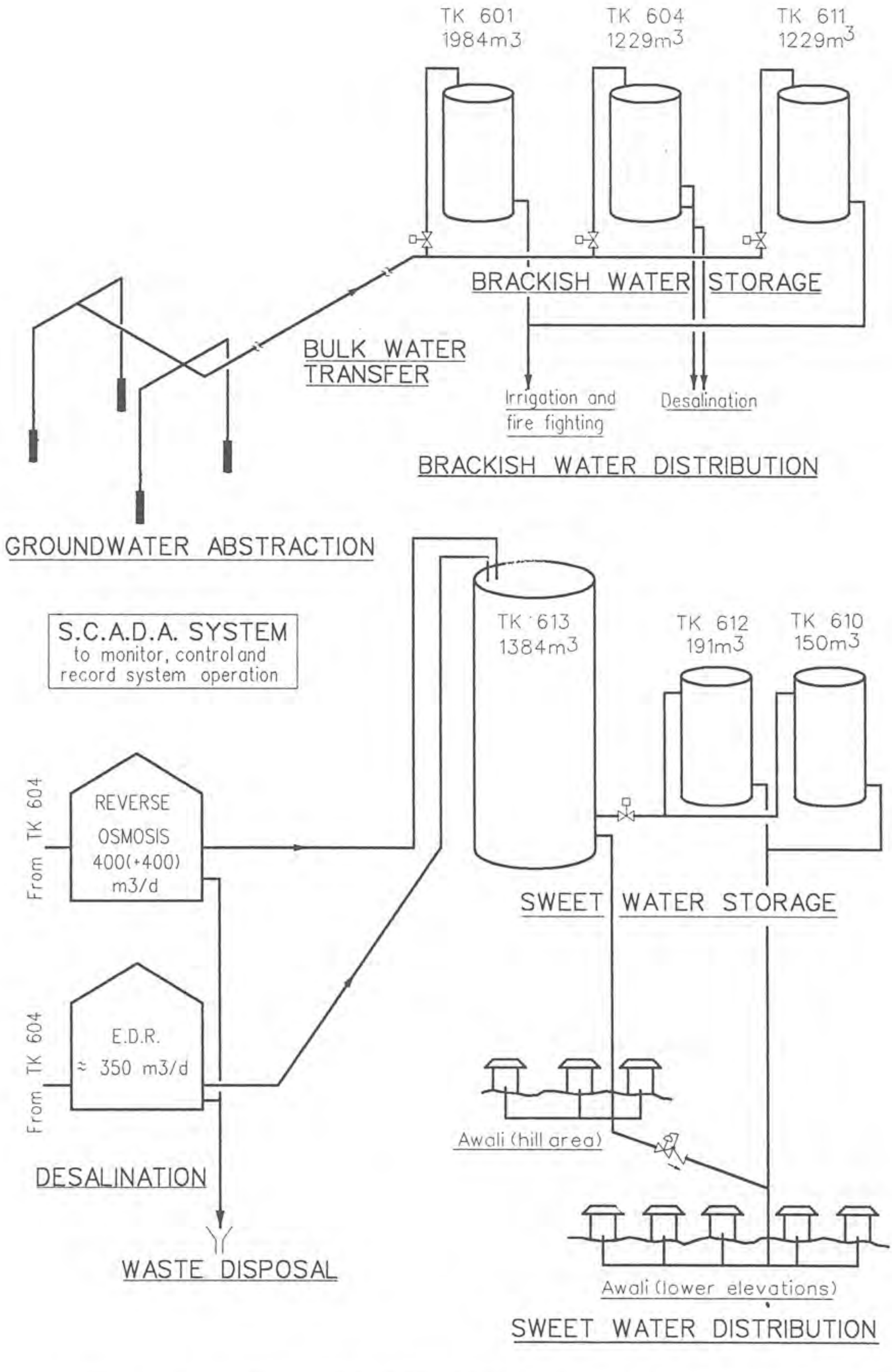

Figure 1: WATER INFRASTRUCTURE REPLACEMENT, AWALI

Further, the opportunity was taken to increase the discharge head of the pumps such that they were capable of delivering groundwater from the abstraction levels direct to Awali without the need for surface boosting. This removed a significant maintenance liability from the Bapco inventory and had the considerable benefit of increasing the aesthetics of the area and reducing the noise nuisance at Zallaq.

The specification for the multi stage borehole pumps was based on a remote offshore installation and made extensive use of nickel aluminium bronze in casing and impellers. Rising mains are of stainless steel. Duty point for the pumps is 150 m^3/h at a discharge pressure of 115 m. The configuration of pumps, anticipated demand and brackish water storage available allows for a more constant approach to the number of pumps in use at any one time. The selection of duty pumps can also be rotated to reduce localised aquifer draw down and distribute the usage of pumps. During peak periods, the hydraulic characteristics of the delivery pipeline to Awali means a significant reduction from the summated flow of the four individual pumps will actually be discharged at Awali.

Each well head has a buried reinforced concrete chamber positioned over it which contains flow meters and instrumentation.

New service roads and landscaping have been provided to enhance the area, which is adjacent to a beach and recreational facility.

Table 1 : Zallaq Well Field Data

Well Reference	Date Drilled	Depth, m	Notes
Bapco # 15	1936	36.0	Decommissioned 1994
Bapco # 32	1952	36.3	Decommissioned 1994
Bapco # 35	1953	43.6	Decommissioned 1990
Bapco # 41	1956	39.6	Decommissioned 1994
WRD 1490 (SW)	1993	38.1	Commissioned 1994
WRD 1491 (NE)	1993	37.5	Commissioned 1994
WRD 1492 (SE)	1993	36.1	Commissioned 1994
WRD 1493 (NW)	1993	37.8	Commissioned 1994

Zallaq to Awali delivery main

The original single above ground delivery line from Zallaq to Awali had been added to progressively, to form the three lines prior to replacement. These had themselves been subject to hydraulic inefficiency due to scaling and ageing. Excessive pressure loss was a frequent problem and bursts were not uncommon. Corrosion products also tended to collect in the tanks at Awali and debris entering the electrodialysis reversal (EDR) plant caused operational problems in addition to making it unpleasant for domestic use. A higher outlet had been formed on the desalination feed to permit settling of the solids prior to draw-off.

The new transmission line was designed to accommodate all likely flows for Awali and subsidiary supplies, together with an allowance for additional flows which may develop in the future. The line was of 500 mm cement mortar lined ductile iron and featured extensive washouts and isolating valves over it's entire 7.6 km length. Cathodic protection, using sacrificial magnesium anodes, was also installed in sections of the pipeline where there was a potential for corrosion.

Desalination

The demand for sweet water was almost directly dependent upon the resident population of Awali. The town had limited light service industries, offices, shops, a school, hospital and recreational facilities. It was the policy of Bapco to reduce the population to levels which generated an overall anticipated demand of 800 m^3/d. However, the final projected demand was dependent upon a series of long term proposals being carried out and so, in conjunction with budget considerations, the plant was designed and installed as an initial 400 m^3/d unit with the potential for extension to 800 m^3/d. The design of the plant, and the building which housed it, took full cognisance of this second stage of the development. Plinths were constructed for future equipment alongside plant to be installed in the initial phase. Access was allowed for, pipework connection points were made and the entire process fully considered.

Demands would progressively increase as consumers were connected to the new distribution system and their use of 'full sweet water' replaced the twin supply from before. In certain areas, the increase in consumption was 150 %.

In order to provide the required flexibility to assess the second stage of the reverse osmosis plant capacity, the EDR plant would continue to operate in parallel with the reverse osmosis first stage plant. After commissioning of the second phase of the RO plant, the EDR plant will be retired from service.

The isolated nature of Awali away from the national water infrastructure supply meant that any excess production could not be responsibly disposed of. Whilst discharge to brackish tanks for irrigation was possible, this was not considered a significant benefit. Excess production potential was essential in order to ensure that demand could be met, but had to be set at reasonable levels to avoid frequent shut down. Extended periods of plant shut down were to be avoided because of the operational problems this can generate. Intermediate short term plant shutdown is inevitable as final production could not accurately meet demand uncertainty.

Trending data for the source from Zallaq had shown , as is common throughout Bahrain, that the aquifer B quality was rapidly deteriorating. The EDR plant, installed in 1981, was ageing and whilst the product quality had been maintained by the extension of the stacks, the capacity and conversion efficiency had been reduced as a result. The plant was operating towards the upper limit of its feed water TDS range and this was not considered the optimum method for the future, when further degradation of the aquifer water would take place.

Table 2 indicates the water quality data which was used in the phase I design of the plant, and that which will form the basis for the phase II extension to be commissioned in 1996.

Table 2 : Water Quality Data

Parameter	Symbol	Unit	Sept 1988	Phase I Design	July 1994	Phase II Design
Total dissolved solids	TDS	mg/l	4,831	5,500	6,127	7,500
Conductivity		μS/cm	5,890 @ 20°	7,600 @ 20°	8,320 @ 25°	9,800
pH			7.4	7.5	7.2	7.2
Ammonia	NH_4^+	mg/l	0.05	0.05	< 0.1	0.1
Potassium	K^+	mg/l	40.8	54	50	55
Sodium	Na^+	mg/l	925	1226	1200	1265
Magnesium	Mg^{++}	mg/l	146	192	200	220
Calcium	Ca^{++}	mg/l	357	420	450	500
Strontium	Sr	mg/l	9.1	10		10
Barium	Ba^{++}	μg/l	16	20		23
Bicarbonate	HCO_3^-	mg/l		210	145	220
Carbonate	CO_3^{--}	mg/l			0	0
Nitrate	NO_3^-	mg/l		0.6	< 0.5	0.6
Chloride as Cl^-		mg/l	1,770	2,338	2,130	2,500
Chloride as NaCl		mg/l			3,800	4,460
Flouride	F^-	μg/l	693	800	<300	400
Sulphate	SO_4^{--}	mg/l	835	1,025	1,025	1,225
Silica	SiO_2	mg/l	20.3	24	22	24

Note : phase II design data is preliminary and subject to confirmation.

Following studies, it was decided that a reverse osmosis (RO) plant would be installed to meet the future supply demands. Key features of the plant include :-

- Design for 400 m^3/d of sweet water to World Health Organisation standards or better.
- Provision in the design configuration for the future plant extension to increase production to 800 m^3/d.
- Use of spiral wound polyamide sea water Filmtec SW30-8040 membranes.
- Provision of duty and standby plant on virtually all processes.
- Extensive instrumentation of the plant, with automatic operation the norm.
- Duty and standby plant for all pumps, filters and key items.
- High standard performance specification.
- Safety provisions.

The chemical treatment features gaseous chlorination of the raw water, ferric chloride coagulation, membrane anti scaling via sequestrant, sulphuric acid pH control and neutralisation of excess chlorine by sodium bisulphate. The permeate receives lime dosing for pH adjustment and inclusion of calcium as a health and taste consideration. Chlorination of the product water by gaseous chlorine is also included. A corrosion inhibitor was considered for protection of the existing pipework, but was not included.

Dosing pumps feature a duty and standby facility, with auto changeover in the event of tripping. Day and mixing tanks are provided on all chemicals except ferric chloride.

The schematic flow diagram, Figure 2, illustrates the primary process features on the plant.

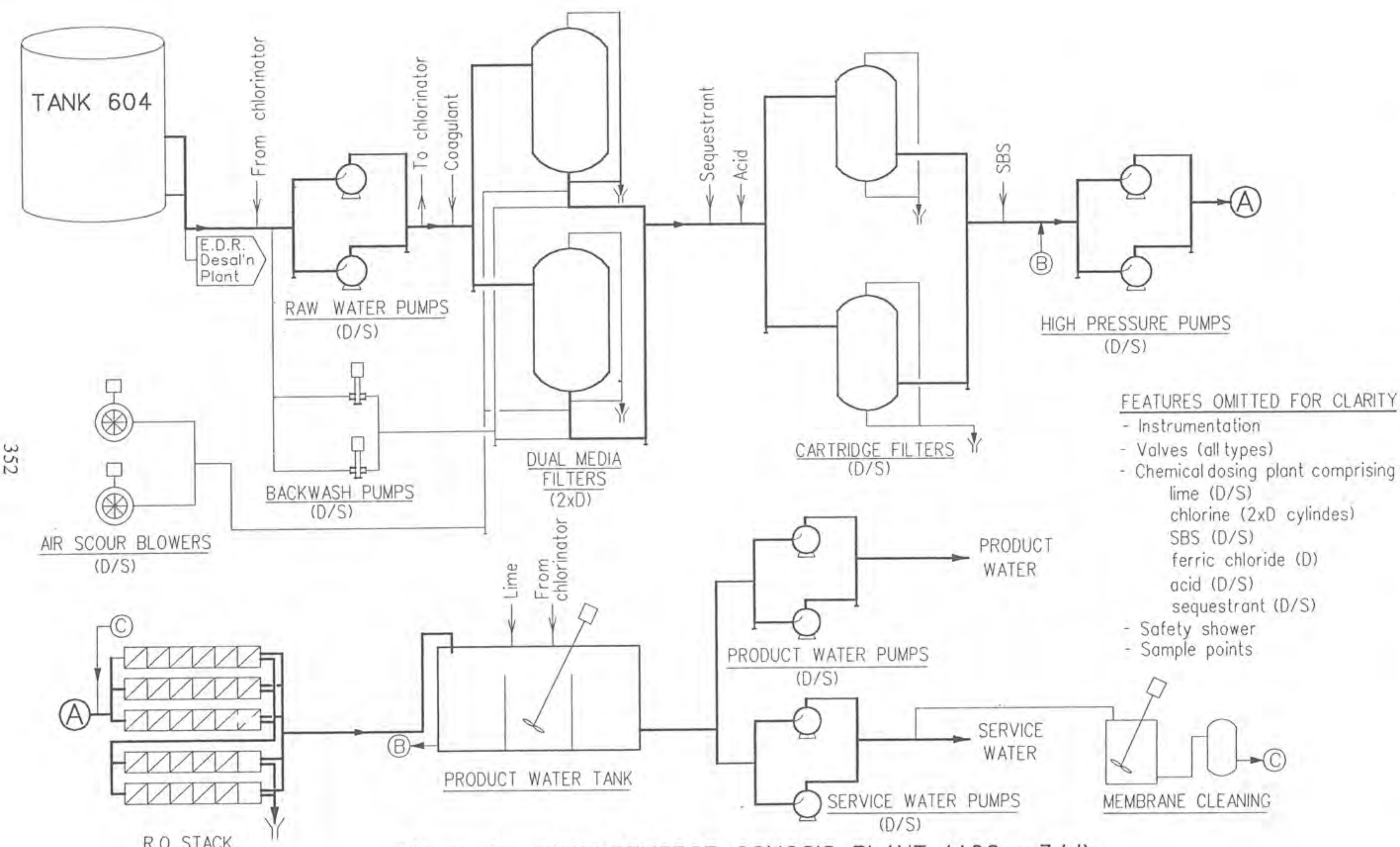

Figure 2 : AWALI REVERSE OSMOSIS PLANT (400 m3/d)

Safety features include an exhaust system for removal of fumes and powders in the chemical dosing area, a dedicated extraction system for the lime hoppers, safety shower and fire alarm system linked into the main government system.

The RO plant is housed in a purpose built 400 m^2 pre-engineered structure incorporating full allowance for the future extension of the plant. Pipework and cables are generally laid on trays or supports in trenches in the complex ground floor slab. Access is by removable stainless steel open gratings. The majority of plant items are on raised plinths, and plinths for the future extension have been constructed.

The generously sized building is divided into two main plant rooms, one containing the RO stacks and the other pumps and filters. Additional rooms provide storage and an air conditioned laboratory, a control room and a mess room. The two plant rooms feature an 'all air' central air conditioning system inducing positive pressure in the building, and designed to produce a controlled 30 $^{\circ}C$ during summer conditions. Full temperature reduction was not justified in all rooms, though it was considered a requirement for membrane protection and to afford protection to the instrumentation.

A new electrical sub-station has been incorporated into the building to serve the plant and adjacent developments.

Chemical storage, with safety shower, is provided in a fenced compound just outside the building.

Initial operation

The trials, commissioning and initial operation presented some unscheduled matters for resolution. These included :-

- An extended shut down which, despite the use of preservatives, may have been a contributory factor in establishing limited biological fouling on the membranes.
- Incorrect dosing of sequestrant and a potential for incompatibility between the anti scalant and the ferric chloride coagulant.
- Reduced production due to flow capacity tolerances of the membranes. Those installed were on the negative side of the permitted variation from the mean values. On a larger plant the number of elements would tend to balance this out, but this was not the case with the 25 installed. A further element was introduced into each of the first stage vessels to compensate for this, as indicated in Figure 2.
- Feed water quality had deteriorated more rapidly than envisaged and nessecitated changes to the feed flow rates and pressures.
- Insoluble material - silica - in the hydrated lime damaging dosing pumps when in suspension and clogging dosing lines by rapid settlement.

These issues have now been rectified, following tests which included destructive autopsies of selected elements, and the plant continues to operate well.

Waste disposal

The previous means of disposal of the brine from the EDR plant back into circulation further decreased the quality of water used for irrigation. The additional waste from the RO plant would exacerbate the situation further. Chemical waste from the EDR plant was discharged at depth but constant head tests carried out by the WRD to determine the potential disposal rate concluded that fissures had been opened up by the reaction of the hydrochloric acid and limestone. Further use of the disposal well for the RO was not carried out.

Alternative means of disposal were considered. These included evaporation lakes, existing sewers and drains, pipelines to the sea and aquifer recharge at depth.

The land area required for the evaporation lakes was considered too extensive to meet winter evaporation needs and had problems associated with reduced percolation rates, fouling, security of performance and safety. Disposal to existing sewers or drains was not considered a viable option due to subsequent treatment and the chemical content of the waste. The capital cost of piping the waste to the sea, some 8 km distant, was prohibitive.

In consultation with the WRD, together with Banoco, it was agreed that the waste would be discharged to the lower depths of the aquifer C via a disposal well to be drilled by the WRD in Awali close to the plant. The disposal well is 146 m deep and features a PVC lining to prevent the discharge of waste flow into the sub-surface strata.

Storage

Water storage in Awali was provided by five above ground steel tanks. Studies indicated that the existing brackish water storage was adequate for projected demands, and could be readily replenished with variable pumping operations at Zallaq. The more constant desalination production required a greater product storage capacity. With no additional storage, the reserves would have been sufficient for less than 12 hours at average flow conditions.

Additional storage therefore took the form of a new sweet water tank. The capacity was selected to provide some 60 hours storage overall. The site was restricted, but the tank had to be of sufficient volume to meet this criteria and allow the pressures within the gravity fed distribution to be attained during peak flows.

The volumetric ratio between the new and combined existing tanks are approaching a level of 6:1. In order to utilise the existing small tanks in a practical way, the pipework was rationalised and configured such that the new tank was filled from the desalination plants. This in turn supplies, in series, the existing tanks with a motorised valve controlling levels throughout. The higher areas of Awali are fed directly off the new tank to ensure a maximum available head. The lower elevations are fed from the two smaller tanks, acting in parallel,

The distribution system pipework is designed to allow for excess pressures on the hill area to be fed through pressure sustaining valves to the lower parts of Awali.

The new tank features a domed roof which removes the need for any internal supports, thus reducing maintenance and increasing the service life. The tank consists of seven courses, ranging in plate thickness from 8 mm for the floor and base course, up to 5 mm for the top course. External coating utilises a zinc phosphate primer, two pack micaceous iron oxide first coat and two pack polyurethane finish coat. Internal protection is afforded by two coats of a two part solvent free epoxy paint. Cathodic protection was not installed.

Pipework diversions were required to permit construction of the tank which took five months. Cathodic protection was not used, and instead coating systems were specified to afford the protection required. Externally, a two component polyamide cured epoxy paint base coat was applied on a primer, with a polyurethane top coat. Internally, two coats of solvent free two component polyamine cured epoxy were used.

The volumes and designations of the tanks are indicated in Table 3 below.

Table 3 : Water Storage Tank Designations and Volumes

Tank Number	Duty	Volume, m3	Notes
601	Brackish	1984	
604	Brackish	1229	Feed to desalination plants
610	Sweet	150	
611	Brackish	1229	
612	Sweet	191	
613	Sweet	1384	New tank

Distribution

Awali has two water supply systems. One supplies brackish water for certain 'domestic' use inside houses, offices, schools etc., and for irrigation and for fire fighting purposes. This system was renewed in 1986 using ductile iron and asbestos cement pipelines. It has been designed to convey all anticipated future demands, including sweet water. It is not feasible to produce sufficient sweet water within Awali to serve the full needs of the town : capital and operational costs are prohibitive by orders of magnitude.

Consequently, it was decided that the existing sweet water distribution system be replaced and that the use of two separate systems, unique in Bahrain, be continued. Computer modelling was used as a design verification tool to establish the optimum sizes of pipework based on hydraulic principles. Designs indicated the size of mains that could have been used were less than the standard minimum size adopted by the Directorate of Water Projects and Planning (DWPP). It was considered important that the minimum national standards be followed in Awali in the anticipation that in years to come, DWPP supplies would be available and the system handed over to the government. In such an eventuality, connection into the existing system would be straightforward and the pipework and fittings would meet DWPP standards. Hydraulically, the use of the larger diameter pipes meant that pressure loses were reduced, benefiting the consumers who are all supplied direct from the mains with no header tanks.

Material selection was based on national standards. Cement mortar lined ductile iron pipe and fittings are used for the mains. Service pipework is medium density polyethylene (MDPE) with pushfit fittings.

The distribution system uses cement mortar lined ductile iron pipework and fittings, in conjunction with medium density polyethylene consumer connections. All connection points have meters installed for monitoring and possible future charging. Zone meters are installed within the system, and provision is made for long term leak detection.

A key design feature, associated with the tank configuration and requirement for maximisation of storage, is the use of two sub-networks. That pipework serving the area of greater elevation, the hill area, is supplied direct from the new tank. The remainder of Awali is fed from the two existing small tanks. The two sub-networks are interlinked downstream of the hill area. Pressure sustaining valves will ensure that a back pressure exists on the hill area at all times. Excess pressure is relieved by the valves, which consequently increases the flow to the rest of Awali.

In total, some 10.2 km of ductile iron pipe and 16.4 km of MDPE pipe was laid and a total of 350 consumer connections made.

Control and monitoring

The overall water infrastructure comprises elements in Zallaq, and sub-groups in Awali for storage, treatment and distribution. It is important that these are monitored and controlled in such a way as to achieve peak performance of the system and maintain supplies. In this respect, a Supervisory, Control and Data Acquisition (SCADA) system was installed and relays data from key installations to a central control point in the RO plant building. This operates pumps at Zallaq, maintains brackish water levels in Awali on a tank 604 (desalination feed) priority basis via motor operated valves to direct flow into tanks, maintains levels in the smaller sweet water tanks by permitting gravity discharge of the new tank and serves to annunciate alarms on the system.

An important feature of this process is the establishment of historical data which demonstrates trending and enables operation outside normal parameters to be easily and readily identified. Flow and level data is recorded by electromagnetic flow meters and sensitive pressure transducers respectively. This facility maximises the flexibility of the system, and enhances the security of supply.

The system incorporates a colour monitor to enable operators, at various levels of authority, either to check the system visually or revise operational limits. Active mimics have been configured for ease of use. Operators are able to assess the functioning of the system as a whole or in detail, noting status, trends and highlighting potential problems. The default screen represents the primary operating data and indicates the tank levels, critical flows and the pump status at Zallaq.

The future

The works carried out to date have produced a successful and reliable water supply to Awali, but further works will be necessary to complete the scheme.

Policy modifications indicate that selected existing housing may now be retained and this, in conjunction with other previously undetermined demands, warrants a reassessment of the sizing for the second stage of the RO plant. Further groundwater quality deterioration will also need to be allowed for in phase II. Significant departure from a second stream of similar size would constitute re-engineering many aspects of the plant. Current indications are that the originally predicted 400 m^3/d will suffice, but this must be clarified.

The RO plant phase II is planned for installation in early 1996, but until this time the EDR plant will continue to operate, pending being mothballed or decommissioned.

There are thus two primary areas for future work. Firstly, the RO plant itself must be extended. In so doing, the opportunity will be taken to incorporate key alarms and controls from the plant into the SCADA system for greater flexibility and control. Secondly, the distribution system will need to be extended, commencing mid 1995, to supply those houses to be retained with sweet water to all outlets.

Summary

The works carried out to date have comprised an unusually complete opportunity to replace a complete water infrastructure.

The engineering studies carried out have achieved rewarding results in securing a system with the following key features :-

- Reliable groundwater abstraction at Zallaq, featuring submersible downhole pumps capable of delivering water direct to Awali without surface boosting.
- Raw water delivery main from Zallaq to Awali, able to meet the long term future requirements.
- Desalination by a highly specified reverse osmosis plant, implemented in two phases, with full consideration of projected raw water deterioration.
- Additional sweet water storage for security of supply and maintenance of pressure.
- Sweet water distribution using ductile iron and medium density polyethylene pipes and fittings to suit.
- A SCADA system to control, monitor and record the performance of the total system.

The effect of these works will be to enhance the living standards, and achieve a greater security of supply, to the residents and workers in Awali.

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Session - 8
Agricultural Irrigation

Irrigation Water Conservation By Automatic Scheduling

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Irrigation Water Conservation by Automatic Scheduling

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ABSTRACT

The effect of automatic irrigation scheduling on water conservation was demonstrated in a field experiment at Hail, Saudi Arabia, on a wheat crop during 1990 - 1993 using two automatic sheduling techniques, viz: the soil water monitoring and water budget based on continuous weather-parameter measurements. Overall water saving by the soil-water technique was 24 % while that for water budget treatment was 25 % as compared to control treatment. The average grain yield of wheat for the three seasons increased by 10 % and 8 % in the soil water and water budget treatments respectively.

INTRODUCTION

The Kingdom of Saudi Arabia has a modern agricultural sector with more than 1,200,000 hectares of irrigated area. Ground water is the only source of irrigation water for agricultural purposes in the country (Ministry of Planning, 1985). The water consumption for agriculture alone is more than 85% of total annual water use of the country. Sprinkler irrigation by center pivot systems is extensively used in irrigation in Saudi Arabia. Over-irrigate and waste water. Many users rely on past experience and personal judgment to schedule irrigation and either under-irrgate or in most cases scheduling strategy based on physically sound principles. Since Saudi Arabia is located in an arid region with limited water resources, it is necessary to optimize water use for agriculture. One of the most efficient techniques in optimizing water use is the technology of automatic irrigation scheduling. This technique allows farmers to obtain quick periodic soil, water and weather data from large fields remotely. Hence labor intensive manual measurements and human error in reading devices are eliminated. Such a system provides precise timing of water application for more efficient water management.

The objectives of this investigation were (1) to develop, implement and evaluate the use of soil water measured and water budget automatic scheduling systems under local field conditions, and (2) to compare the water saved and the grain yield from the application of the automatic irrigation scheduling techniques with that obtained by the common irrigation scheduling practice.

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BACKGROUND

Automatic Irrigation Scheduling based on Soil Water Measurement:
The introduction and use of microcomputers and telemetry system coupled with development of microprocessor technology have greatly expanded the options in implementing automation for irrigation. The advancement of computer has overcome many of the difficulties encountered in collecting and analyzing field data for soil water status.

During the last few years, various trials have been made, using automatic irrigation scheduling based on soil water measurement. A digital irrigation controller and gypsum blocks, soil water content were satisfactorily maintained within the desired levels for solid set sprinkler system (shull and Dylla, 1980). In a feasbility study of using tensiometers for scheduling of turf irrigation, substantial water savings were obtained in sensor controlled plots (Augustin and Snyder, 1984). Automatic scheduling, based on continuous monitoring of matric potential by tensiometers equipped with pressure transducers, has been successfully used in irrgiation scheduling (Feyen and Gilley, 1985; Merril et al, 1987; Grismer, 1987; Mclendon et al, 1983; and Thomson and Threadgill, 1987,). Recently, a new technique employing an infrared (IR) telemetry system was used to read tensiometers and send control signals to remote equipment such as pumps, irrigation valves and alarms (Pogue, 1987; Drum , 1988).

AUTOMATIC IRRIGATION SCHEDULING BASED ON WATER BUDGET

During the past twenty years, several models have been developed based on soil water, soil, plant and weather parameters for irrgation scheduling . A widely used model was developed by the U.S. Department of Agriculture - USDA (Jensen et al., 1970, 1971). In this model, the daily potential evapotranspiration (ETp) is an essential term which was usually obtained by one of several ET equations such as Penman, and Jensen and Haise methods. It also forecasts the timing and amount of irrigation water necessary for optimum crop production. The model was modified by Harrington and Heermann (1981) to make the interactive program more flexible in providing output to meet the needs of various irrigators and irrigation systems.

Fischbach (1980) described an " Irrigate" program which was part of the Nebraska agricultural network computer system (AGNET). This model used either the modified Blaney-Criddle or the modified Penman method to determine ETp. Weather data was collected from weather stations around the state. scheduling information for invidual fields is transmitted to the farmer by radio, telephone or newspapers.

Saxton, et al. (1974) developed a digital simulation model "SPAW" to compute daily ET from inputs of daily ETp, crop and soil moisture characteristics. The model was calibrated and verified by 3-4 years data from watersheds near Treynor, Iowa with corn and grass.

A soil water budge. model "SPAW-IRRIG" , a modification of the "SPAW" , was used for irrigation scheduling of corn grown in Washington's Columbia basin during 1984 and 1985 under a center pivot (CP) irrigation system (Field et al ., 1988). A finite difference routine was used to determine the re-distribution of soil water in the soil profile.

A computer-based water balance model (CBWB) was developed and evaluated for corn and soybean irrigated by a high pressure CP system in South Carolina (Camp et al., 1981). The model utilized the weather data to estimate ETp using the modified Jensen-Haise method.

A microcomputer based irrgation management and control system was developed by Vincent et al. (1986). In this irrigation system, real time data of weather and soil water conditions were used to initiate irrigation, estimate the required depth of water and to correct the calculated soil water level. A remotely controlled data logger was operated continuously to monitor and collect data throughout the season.

A computerized irrigation scheduling model was used for irrigated corn using center pivots and raingun systems in Michigan, USA (Harmsen et al., 1982). Modified Penman method based on long term averages of climatic data was used to estimate ETp for the next ten days. In this experiment the actual crop evapotranspiration, ETc (= ETp x crop coefficient) was determined from the estimates of ETp from FAO Penman (Doorenbos and Pruitt, 1977). Soil moisture contents and soil water potential were taken to check the accuracy of the model.

Heermann et al. (1984) developed an integrated water and energy management system which monitored, controlled pumps scheduled irrigations and controlled electrical load for 15 CP systems. Radio communication was used to transmit data and control signals. The management system was operated for three years in Colorado with corn grown on sandy soil (Heermann et al., 1985 & 1987). The results showed an improvement in the management of individual CP's, a reduction of water consumption, the number of inspection trips per day and a significant saving in electric energy cost.

MATERIALS AND METHODS

A field experiment was carried out for three growing seasons (1990 to 1993) to implement the automatic irrigation scheduling systems. The experiment was conducted at the farm of Hail Development Company (HADCO) in Hail region about 500 km north west of Riyadh, Saudi Arabia.

The HADCO's irrigation strategy is based on estimation of reference evapotranspiration (ETO) from California Irrigation Management Information Service (CIMIS) model based on Penman Equation and crop coefficients, Kc, from Al-Shamary (1985). He suggested a curve for wheat (Yecoro Rojo variety) with a 142 day growth cycle and Kc figures of 0.95 for Dec., 0.99 for Jan., 1.1 for Feb., March & April and 0.35 for May. Accordingly, the irrigation schedule consisted of 3 days on / 2 days off cycle in the early season, 4 days on / two days off around heading time and almost continuous operation in the filling and maturity stages upto 4 days prior to harvest at a pivot speed of 72%. The maximum depth of water applied during the season (Dec. 20 to May 10) with this strategy might reach 810 mm (including rain). Variations on this schedule are made to accomodate applications of area, herbicides and pesticides. Periodic adjustments were also made to make up for the changes in climatic conditions.

The experiment was conducted on three plots each consisting of two center pivots, one of the plots (circles H9E and H9W) was irrigated with direct sensing of the soil water tension using remotely controlled tensiometers (automatic tensiometers), while the second plot (circles G9E & G9W) was irrigated using an irrigation scheduling model based on measured weather data. The third plot (circles J9E & J9W) was irrigated according to the routine schedule followed by HADCO and used as a control treatment for comparison. Each pivot circle in the three treatments was further sub-divided into twenty strip plots measuring 1 x 2 m each across the diameter of each circle to measure crop yield variations due to the irrigation treatments.

Mechanical and chemical tests were carried out on soil samples from the six circles. The soil at the experimental site was dominantly a sandy loam soil. The field capacity and wilting points as determined by pressure plate apparatus were 11.9 % and 5.4 % (weight basis), respectively. The soil water characteristics curve for the soil was also established (Fig 1). The pH values of the soil samples ranged from 7.8 to 8.2, which indicated that the soil was slightly alkaline. Water samples were collected from wells G9, H9 and J9 and analyzed in HADCO's lab. The water pH values from all the wells ranged from 8 to 8.2, which were within the normal range. The EC values were 0.73 to 0.79 ds/m which are very close to 0.75 associated with a negligible degree of salinity problem. Also chloride was found to range from 3.3 to 3.6, i.e. less than 4, which is classified as negligible degree of hazard . The Sodium Adsorption Ratio (SAR) values ranged between 2.51 and 2.91 and were below the hazardous limit of 3.

The irrigation systems were low pressure Lindsay center pivots, powered by *CaterPiller 354 kw diesel engines. The pumps were *Pearless deep turbines connected to the engine through a right angle drive transmission system.

Software was developed to control the engine-pump system via a PC. An automatic controller was designed to allow interfacing and with necessary time delays in a sequence of operations to give additional control functions. Figure (2) shows a block diagram of the conceptual design of the automatic interface system which can control the pumping system.

Three turbine type electronic flowmeters were installed into each of the 0.25 m (10") pipe connecting pumps to the center pivot circles to measure the volume of water applied. In addition, the flow from all the nozzles from each pivot was measured twice (at the beginning and end of the experiment) and the average flow rate was multiplied by the hours of operation to obtain the total water applied over the application period. This was used in calibration of flow meters.

Uniformity evaluation catch can tests of center pivot systems were carried out and the average values of coefficient of uniformity (Cu) ranged from 73 to 84%.

CROP ESTABLISHMENT AND GROWTH

The experiment was conducted on wheat cultivar Yecora rojo and planting dates were January 7, 1990; Dec 30, 1991 and Dec 28, 1992 with seeding rate of 180 kg/ha. The seeds were drilled by machine in 0.20 m rows. Fertilizer was applied before planting at the rate of 50 N and 235 P_2O_5 kg / ha. Another 300 kg / ha N was added as a split application during the plant development stages throughout the growing season.

The crop responses evaluated were: grain yield (ton/ha), number of tillers per m, number of kernels per spike, 1000-kernels weight (g), biological yield (ton/ha) and harvest index (%).

The statistical analysis of variance for the data was done on an IBM compatible PC, using SAS (Statistical Analysis System) program. Analysis of variance was performed on each trial to determine significance of differences among the water treatments as outlined by Steel and Torrie (1980), on the basis of the Nested Classification design.

The details of installation of instruments and procedures for each treatment were as follows:

SOIL-WATER BASED EXPERIMENT: This experiment was carried out on plot consisting of two circles H9E and H9W. In this experiment, the irrigation scheduling was based on remote sensing of soil water through automatic tensiometers. The soil water data from these automatic tensiometers were transferred through infrared telemetry (IR) in a daisy chain closed loop system to a control room for making irrigation decisions. The experiment included the following main components: automatic tensiometers, base and field stations and control panel as shown in Fig. 3.

AUTOMATIC TENSIOMETERS: The automatic tensiometers were equipped with transducers to convert soil water potentials to voltage for transmission in the form of signals via an infrared telemetry communication system. In each circle, three sets of sensing devices were installed 20 m away from the access road for easy maintenance. Their distances from the pivot were 60, 200 and 340 m (Fig 3). Each set consisted of two automatic tensiometers at 0.3 m and 0.6 m depths from the soil surface. The data from the shallow automatic tensiometers for both circles (6 tensiometers) were utilized to initiate/terminate the irrigation cycle. These tensiometers were selected for decisions since the active root zone for wheat under center pivot irrigations ranges from 0.5 to 0.6 m (Hunting, 1987) and about 70% of the water used by plants is extracted from the top 50% depth (Hansen et al, 1980). In addition to automatic tensiometers, three sets of manual tensiometers were installed in each circle at depths 0.3, 0.6 and 0.9 m. The soil moisture contents were also determined by gravimetric method from each circle periodically.

BASE STATIONS: The Base Station is the control center of the system and consisted of an IBM compatible PC, IR transmitter, an IR receiver, a solar panel, a power supply and an RS232 connector for connection into the PC. The data from different field stations were collected and batched to the computer. The control commands, sent from the computer to any field station are passed through the base station. The microprocessor was an 80286 with 2MB RAM and a 40 MB hard drive.

FIELD STATIONS: The field station (FS) was capable of automatically reading up to eleven sensors and transmitting the data by IR telemetry link to the computer in the base station. The field station was also capable of receiving and executing commands to turn on/off engines, pumps and center pivots. The field station consisted of three major components; the microprocessor module, the IR receiver and IR transmitter. There were seven field stations in the system, one of them was used for operating pumping unit and the rest were used for transferring data from tensiometers to base stations. The field stations were connected to the base station in a daisy chain arrangement as shown in figure (3).

SYSTEM SOFTWARE AND OPERATION: To achieve a fully automatic operation, the software determines when to irrigate and the computer automatically turns on/off the irrigation system.

The software for irrigation management was menu-driven and user-friendly. It included four menus; field station (FS), base station (BS), reporting and system management in a logical sequence of actions. Each of these menus contained several functions. In the field station menu, it was possible to specify a field station, add/delete a sensor to a FS, and define the type of sensor and specify its range of operation. In the base station menu, the user is able to specify, update and control any device connected to it. In order to facilitate the operation of the system, a batch file was created, so the user only needs to execute this file to display the desired mode of operation (automatic or manual). If the automatic option is chosen, the current readings of tensiometers and the threshold limits for starting/stopping irrigation are displayed for user information. These limits may be ammended by the user. In this automatic mode, the decision when to start and stop pump operaion is software controlled without any human intervention. Messages from the FS's are received during the execution of any of the functions of the different menus. If the manual option is chosen, the pump may be started or stopped by entering a command on the keyboard of the computer manually.

The software for irrigation decisions were based on two main factors; the water status in the soil profile and the soil type. Since the experimental soils were mainly sandy loam, the corresponding threshold suctions at which irrigation should start/stop were obtained as 35 kpa ad 15 kpa ($\pm$ 5) from water retention curves for different soils as given by Haise & Hagan (1967). The measured tension values by tensiometers were compared to the threshold values using a voting mechanism (i.e. the decision was made when the majority of tensiometers read within the threshold limit value $\pm$ 5 kpa). Therefore when the condition of "start irrigation" was met, the computer automatically sent a command signal to start the engine-pump system. On the other hand, when the condition of "stop irrigation" was met, the computer sent a similar command in order to stop the system. The software provided two maintenance signals for each irrometer making the monitoring of the tensiometers automatic and easier.

THE WATER BUDGET EXPERIMENT: This experiment was carried out on two circles G9E and G9W at the same site. An automatic weather station (Campbell Scientific*) with 21X micrologger was installed at a site near the experimental location. It was connected to the PC via a twisted pair shielded cable for data transfer. The micrologger was programmed to collect data every minute and averaged every hour and twenty four hours. The measured parameters were: air temperature, relative humidity, shortwave radiation, net radiation, soil temperature (0.1 m & 0.2 m depths), wind speed and wind direction at 2 m height.

The software developed for the water budget scheduling was menu-driven and utilized ETO from the modified Penman method, FAO version as given by Doorenbos and Pruitt (1977). The previous day weather data were read from the micrologger and used to calculate the current depletion. The total depletion on any day after an irrigation was determined by adding the depletion forcasted for the day to the sum of previous depletions since irrigation.

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* The use of names does not imply an endorsement of the product. It is mentioned only for the benefit of the reader.

This total was compared to the readily available water in the soil. If the decision was to start an irrigation, the number of complete pivot circles at a selected speed was determined to add the required volume of water.

The soil water status in the water budget method (circles G9E & G9W) and control circles (J9E & J9W) was monitored by gravimetric method and manual tensiometers. The gravimetric data were measured at 0.15, 0.25, 0.50, 0.75 and 1 m depth from three different locations in each circle.

RESULTS AND DISCUSSIONS

Since irrigation in both circles for each treatment was initiated at the same time, only the data from one circle was presented and discussed. The soil water tension for the circle H9W at 0.3 m and 0.6 m depths at one location in the soil profile of the soil-water-based irrigation scheduling system for the third season is shown in figures (4) and (5). These data represent the 24 hours average of the soil water matric potential by automatic tensiometers taken every 29.4 minutes. From figure 4 the soil moisture at 0.3 m depth shows that the tension in circle H9W lies most of the times between 15 and 30 kPa. This indicates that the soil profile in general had enough surplus water. The moisture tension at 0.6m depth from this circle also varies from 15 to 30 kPa. Hence at this depth also enough surplus water was available. Long irrigation cycles with large depths of application may cause some water to accumulate on the soil surface due to limitation in soil infiltration capacity. However, when an irrigation is terminated, some of the accumulated water continues to infiltrate, and can increase the soil water level below the threshold limit (15 kPa). From this it can be concluded that further saving of water could be achieved by changing the upper threshold limit. Since similar fluctuations of soil matric potential took place in the previous seasons, i.e. 1990-91 and 1991-92, therefore, data were not presented.

The monitoring of tensiometers was performed automatically through software. Some air bubbles were entrapped from time to time and subsequently maintenance was needed. Their effect on irrigation scheduling was insignificant as the irrigation decision was made on voting mechanism.

The tensiometers were reliable in sensing and monitoring the soil water status at all locations and were very successful in the automation of irrigation. They needed only bubble evacuation by adding water. Automatic monitoring of tensiometer failure through IR system made servicing easier.

The soil-water matric potential at one location as measured by manual tensiometers is plotted vs time since planting for the same season in the water budget experiment (G9W) in figure 6. The moisture tension at 0.3 m ranges from 10-16 kPa during the first 70 days and increases during the rest of the season. The lower tensiometers (0.6 m and 0.9 m depths) show that the potentials are within the threshold limit during the entire season. This indicated that water was available (at deeper level) which could be extracted by the roots as they are fully developed at this stage (after full cover at 55 days).

The soil-water potential in control circles at one location for the same season (3rd season) are presented figure 7. The potentials ranged from 10 to 20 kPa for the first 55 days while during the rest of the season the tension was a little higher, but below field capacity. This shows that more water is applied in the control treatment during the entire season.

From the gravimetric water content data, the variation of the water distribution in the soil profile was studied. The moisture content values after irrigation at each depth were averaged for the three locations in each circle under study. These averages were plotted versus the depth at three different time intervals for the circles H9E, G9E and J9E for the third season in Figs. 8,9,10. From these figures it is seen that the moisture distribution in circles H9E and G9E throughout the entire profle tends to remain constant during the entire season. Mostly, the values are near the field capacity. This implies that the water was distributed uniformly in the profile and that some deep percolation might be taking place in the lower part and hence more water could be saved by proper water management.

The average water applied for all the treatments during the three seasons are presented in Table (1). The total amount of water applied for the soil water based experiment during the three seasons were 550, 404 and 386 mm while the water applied for the water budget experiment were 496, 407 and 412 mm as compared to the control in which the water applied was 651, 487 and 633 mm. This shows that the overall water saving percentages during the three seasons were 24% and 25% for the soil-water based and water budget techniques as compared to control.

The irrigation dates and the corresponding amounts of water applied for all the three treatments during the growing season 1990-1991 are presented in Table 2 as an example.

A summary of the analysis of variance combined for the three water regimes is given in Table (3). The differences between the two means of the grain yield and yield components for soil water based treatment and water budget were not significant, although the means of the total amount of water used in these two treatments were not significantly different, whereas it was significant as compared to the control.

The average grain yield over the three seasons was 7.06 tons/ha in the water sensed experiment and 7.17 ton/ha in the water budget experiment as compared to 6.54 ton/ha in the control experiment as shown in Table 4. This indicates that the soil water sensed experiment and water budget techniques resulted in greater yield per hectare than the control experiment.

Generally, communication and interrogation were not interrupted significantly by dust storms or fogs during the experiment. However, Visibility during severe dust storms (<10 m visibility) might prevent communications.

Proper grounding eliminated the interference on the IR links and the noise on the IR signal did not affect the reporting of the IR system. However, the telemetry failed to communicate when the sun became nearly in line with the receiver sight. Hoods were provided to prevent the sun from interfering with the IR signal. The lenses of the transmitter/receiver as well as the solar cells, needed regular (at least weekly) cleaning of dust. Also, birds fences decreased the efficiency of the solar cells. A rail fence was then installed to prevent birds from using the solar cells as a stand.

CONCLUSIONS

The performance of the two field automatic irrigation scheduling systems was satisfactory. Favorable soil water conditions required for crop growth maintained.

Automatic scheduling systems optimized irrigation water use with 24% and 25% of water savings and resulted in increased crop yield of 10% and 8% in the soil water sensed and water budget experiments respectively.

The automatic system for irrigation scheduling was adaptable to large center pivot systems. The installation and maintenance of the IR system did not present any major problems.

On the whole, the system was found to work satisfactorily. No failures occurred under the existing field conditions during the entire growth period. The system was not completely free from malfunctions, but did result in a satisfactory performance during this experiment and was practical to implement.

The system is flexible enough and can be tailored to meet the needs of individual users, companies and/or groups of users. Costs for farmers are reasonable justifying the investment in the advanced technology of irrigation scheduling. This will enable them to make the best decisions under existing constraints. The resulting benefits are increased net returns, reduced labour input and increased agricultural production from a limited water supply.

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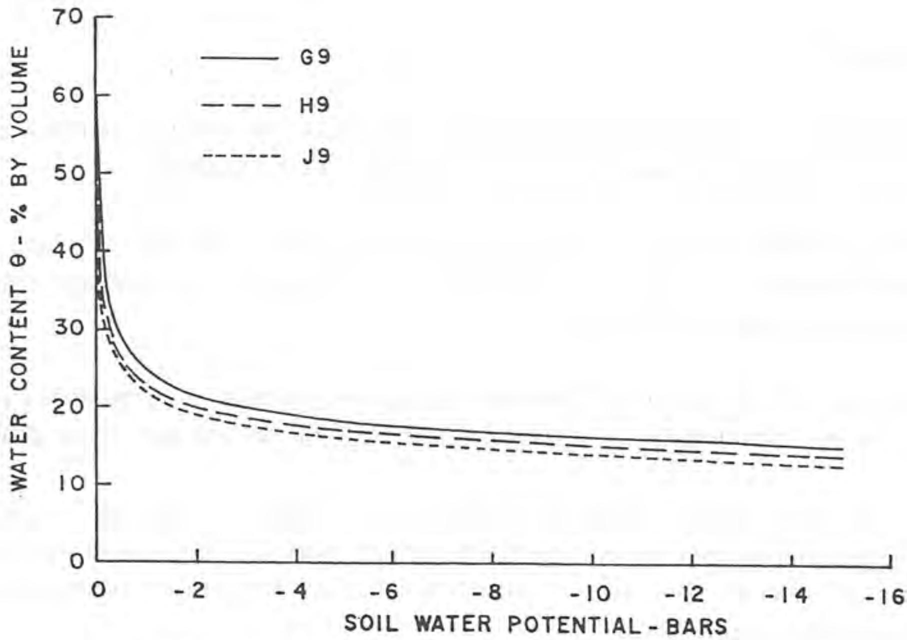

Fig 1 . Soil water characteristic curves for the dominant soils in the experimental field .

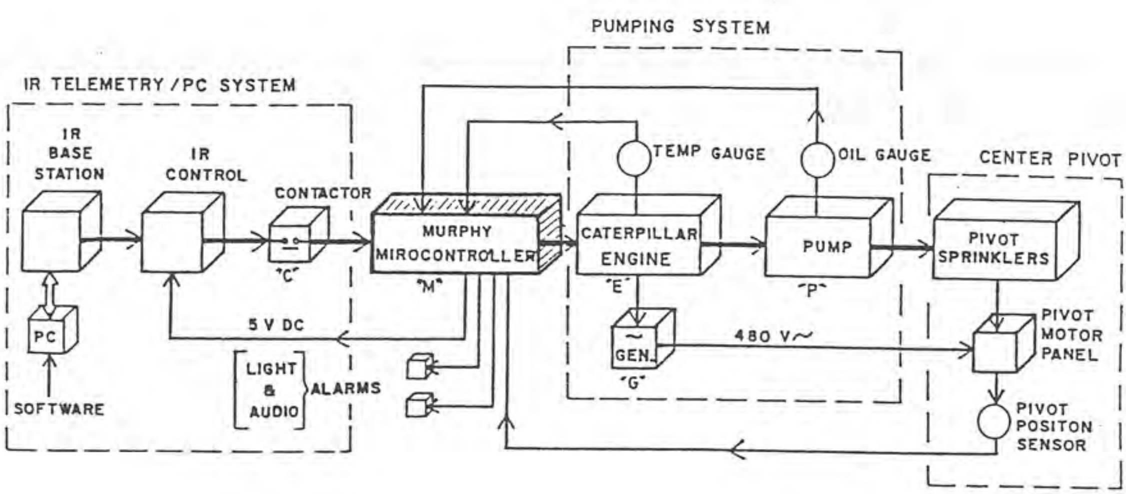

Fig 2 . Design concept of an automatic system to interface the IR telemetry / PC system to the pumping unit .

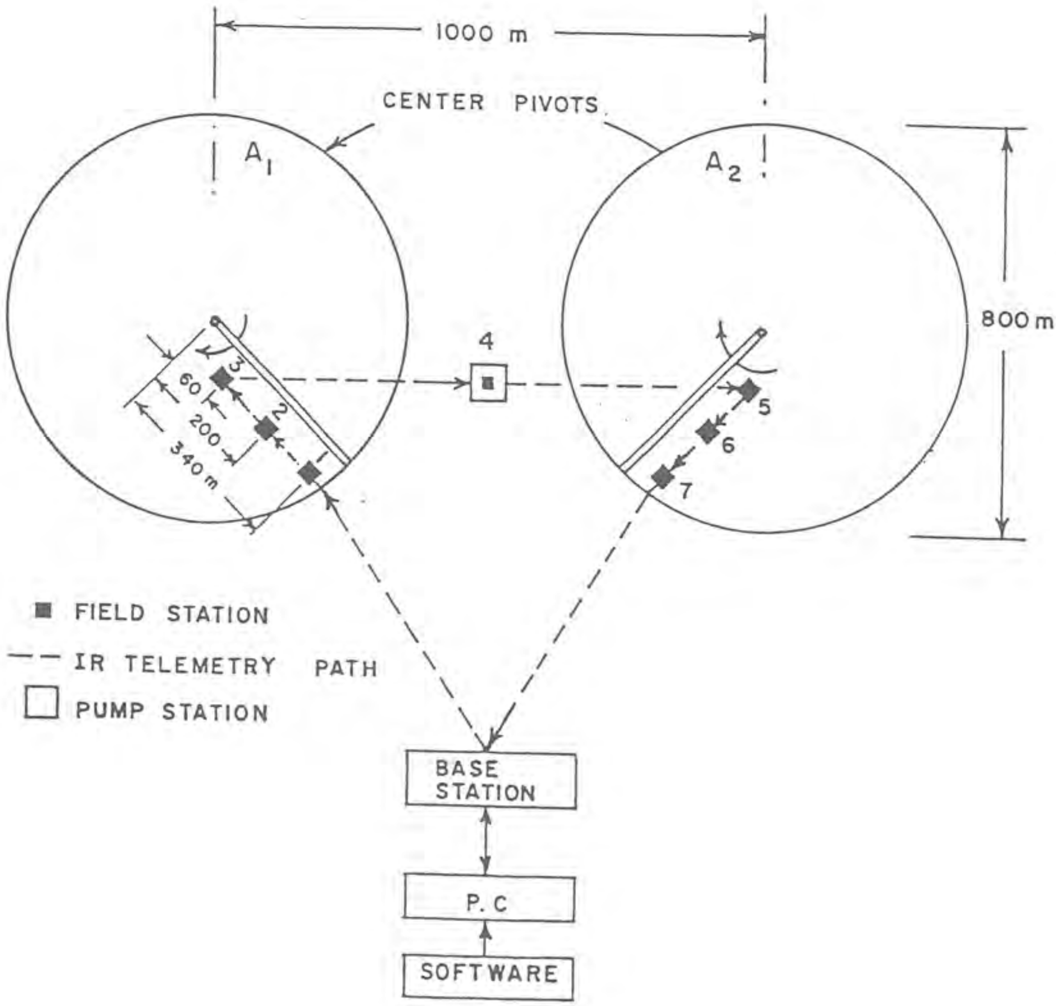

Fig. (3): Schematic of IR Telemtry and Control Systems for Automatic Scheduling via Soil Moisture Sensing

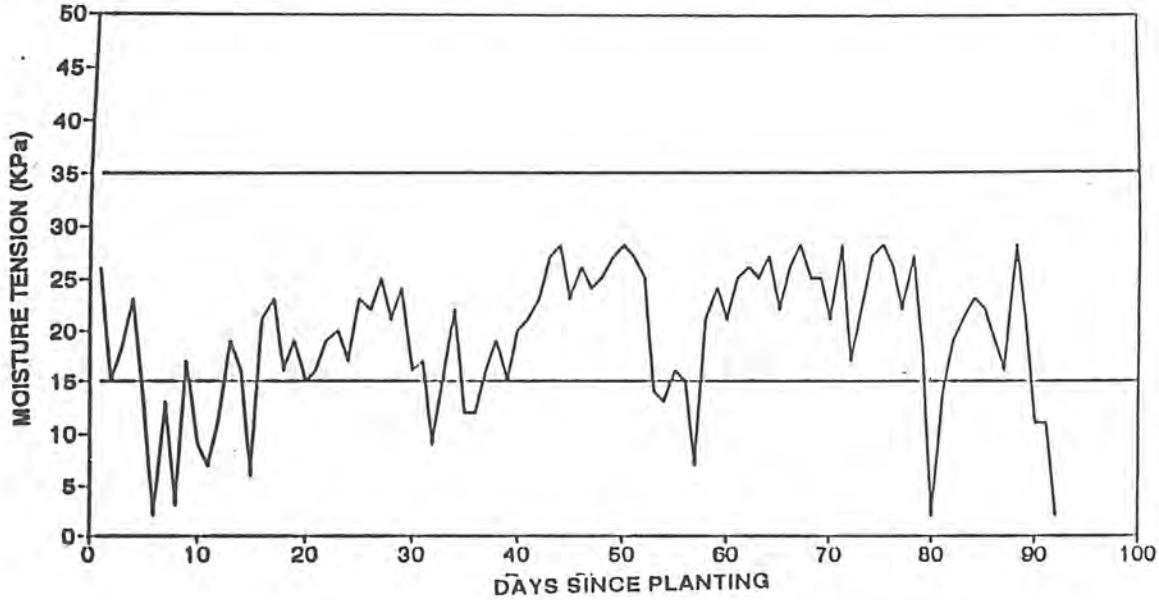

Fig 4 . Soil moisture tension by automatic tensiometers during the third growing season for circle H9E (0.30 m).

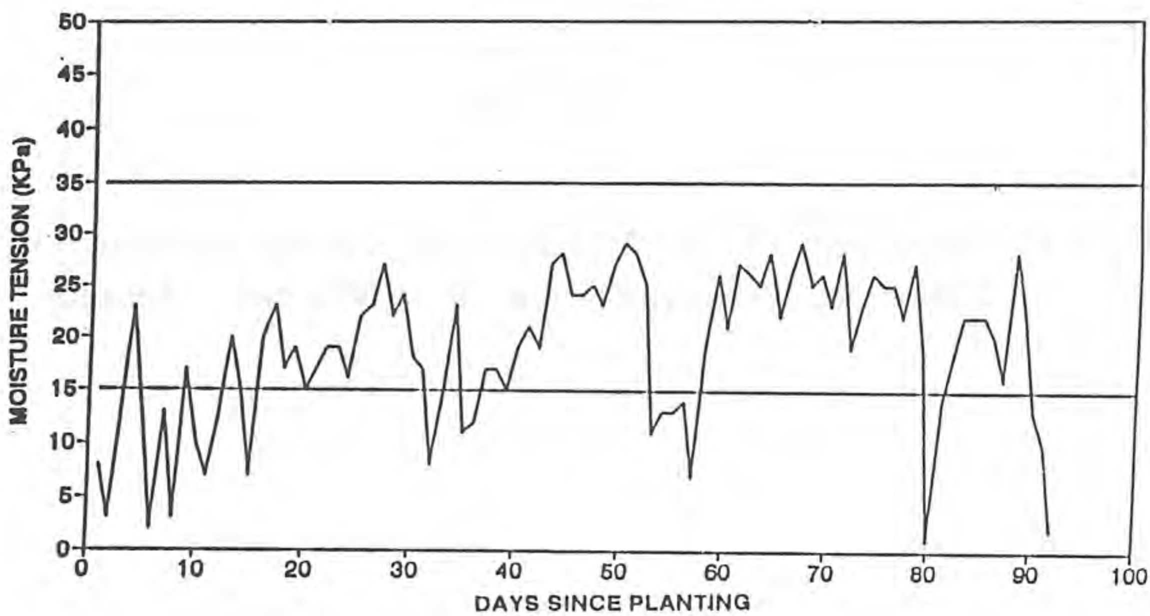

Fig 5 . Soil moisture tension by automatic tensiometers during the third growing season for circle H9E (0.60 m).

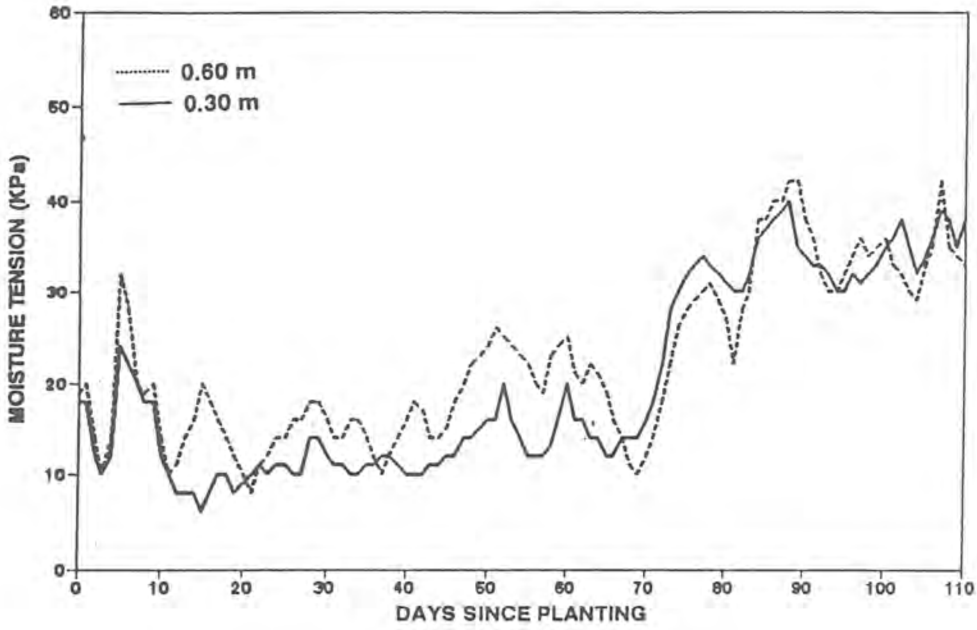

Fig 6 . Soil moisture tension by automatic tensiometers during the third growing season for circle G9E (0.30 & 0.60 m).

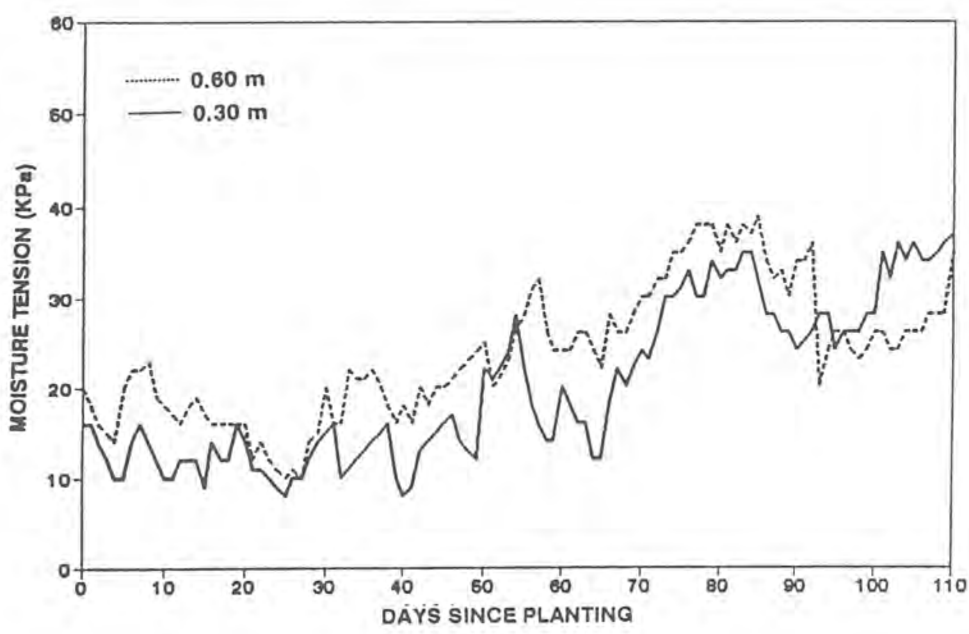

Fig 7 . Soil moisture tension by automatic tensiometers during the third growing season for circle J9E (0.30 & 0.60 m).

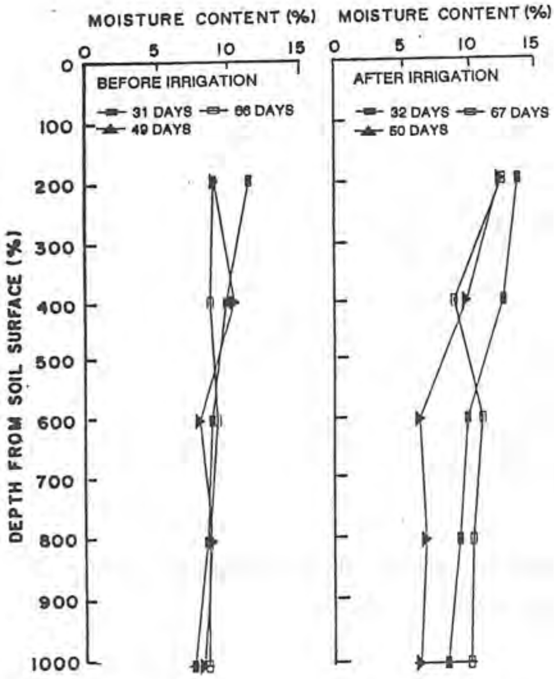

Fig 8 . Moisture content distribution in the soil profile as measured by gravimetric method for circle H9E befor and after irrigation (3rd.season)

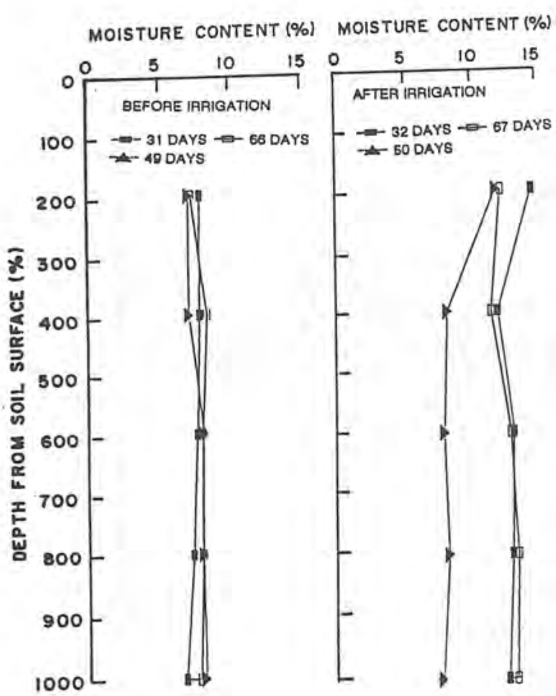

Fig 9 . Moisture content distribution in the soil profile as measured by gravimetric method for circle G9E befor and after irrigation (3rd.season).

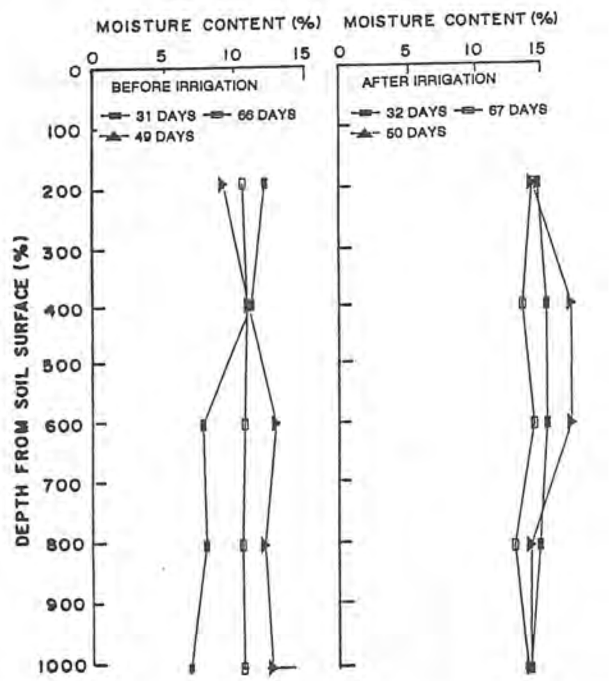

Fig 10 . Moisture content distribution in the soil profile as measured by gravimetric method for circle J9E befor and after irrigation (3rd.season).

Table 1 . Water Applied and Water Savings During the Three Growing Seasons for the Different Treatments.

Pivot	Season 1990-91					Season 1991-92					Season 1992-93				
	Total Oper. hrs	Flow Rate L/S	Water Appl. mm	Ave. Water mm	Sav-ing %	Total Oper. Hrs	Flow Rate L/S	Water Appl. mm	Ave. Water mm	Sav-ing %	Total Oper. hrs	Flow Rate L/S	Water Appl. mm	Ave Water mm	Sav-ing %
H9-E	1413	57.67	546	550	15	1051	57.67	401	404	17	1029	57.84	397	386	24
H9-W	1413	58.43	553			1051	58.43	406			1029	54.69	375		
G9-E	1463	50.64	494	496	24	1211	50.64	406	407	16	1107	57.80	427	412	25
G9-W	1463	50.97	497			1211	50.97	408			1107	53.83	397		
J9-E	1818	54.03	655	651		1369	54.03	490	487		1587	58.10	615	633	
J9-W	1818	53.35	647			1369	53.35	484			1587	61.59	652		

Table 2: Irrigation scheduling for the three treatments at Hail for growing season 1990-91

H9-E (A1)				G9-E (B1)				J9-E (C1)			
Irrigation Date	Start Time	Irr. Hrs.	Irr. Vol. cu.m	Irrigation Date	Start Time	Irr. Hrs.	Irr. Vol. cu.m	Irrigation Date	Start Time	Irr. Hrs.	Irr. Vol. cu. m
Jan 07	14.54	144.4	29987	Jan 12	07:55	53.0	9662	Jan 11	08:30	192.0	37346
Jan 16	9.15	38.2	7920	Jan 15	08:10	24.5	4466	Jan 21	16:20	20.0	3890
Jan 19	15.10	19.1	3959	Jan 16	09:10	32.0	5834	Jan 26	13:25	61.0	11865
Jan 22	7.30	37.3	7744	Jan 19	14:10	27.0	4922	Jan 30	14:25	64.2	12487
Jan 24	14.42	21.4	4439	Jan 23	10:20	49.0	8933	Feb 04	08:18	76.8	14938
Jan 30	11.30	9.0	1869	Jan 27	16:35	14.4	2625	Feb 09	09:27	75.2	14627
Jan 31	11.30	20.2	4196	Jan 31	10:30	16.3	2972	Feb 16	09:40	72.3	14063
Feb 04	8.45	23.2	4815	Feb 03	10:30	52.2	9516	Feb 25	10:20	21.2	4124
Feb 05	9.43	22.3	4621	Feb 06	08:20	23.0	4193	Mar 01	08:08	95.1	18498
Feb 09	9.26	46.5	9652	Feb 09	16:20	40.2	7329	Mar 07	08:11	29.3	5699
Feb 12	10.39	27.5	5709	Feb 12	11:15	26.5	4831	Mar 11	09:25	142.6	27737
Feb 18	16.00	2.2	457	Feb 16	14:10	0.8	146	Mar 18	14:30	47.0	9142
Feb 19	8.46	8.6	1783	Feb 20	08:20	25.2	4594	Mar 25	10:10	202.5	39388
Feb 20	11.45	16.4	3405	Feb 25	10:23	21.7	3956	Apr 04	08:40	62.0	12059
Mar 03	9.53	52.1	10819	Mar 02	10:05	54.4	9917	Apr 09	09:32	8.6	1673
Mar 08	9.28	10.3	2143	Mar 07	10:36	24.5	4466	Apr 11	08:07	68.0	13227
Mar 11	15.38	20.0	4161	Mar 11	12:00	123.2	22460	Apr 15	11:20	88.1	17136
Mar 12	14.34	45.3	9397	Mar 21	16:20	40.0	7292	Apr 23	08:08	22.0	4279
Mar 16	9.19	8.1	1684	Mar 25	11:35	7.8	1422	Apr 25	08:52	161.2	31355
Mar 17	8.57	75.0	15571	Mar 27	11:03	0.5	91	May 04	10:30	22.1	4299
Mar 20	15.34	16.4	3394	Mar 28	14:18	60.6	11048	May 05	12:45	58.3	11340
Mar 25	10.10	49.5	10277	Apr 02	08:35	101.0	18413	May 09	08:00	49.9	9706
Mar 31	14.59	170.5	35400	Apr 07	15:45	22.5	4102	May 13	09:05	170.8	33222
Apr 11	14.17	95.1	19740	Apr 10	15:10	184.5	33635	May 20	15:45	7.6	1478
Apr 23	8.21	28.5	5915	Apr 23	08:55	19.3	3518				
Apr 24	14.28	191.2	39697	Apr 25	09:35	175.3	31958				
May 04	8.10	71.4	14817	May 04	09:05	9.5	1732				
May 07	12.20	25.3	5261	May 05	15:05	42.7	7784				
May 11	8.20	20.0	4152	May 08	10:05	10.7	1951				
May 12	9.45	16.2	3353	May 12	07:30	17.9	3263				
May 16	8.35	45.6	9457	May 13	13:15	4.9	893				
May 18	11.20	54.1	11232	May 14	08:45	47.9	8732				
				May 16	17:15	86.8	15824				
				May 20	08:10	22.9	4175				
TOTAL		1430.7	297024			1462.7	266656			1817.8	353577

TABLE 3. Analysis of variance combined over 3 years for different crop parameters.

SOURCE OF VARIATION	D.F.	MEAN SQUARES						
		Yield t /ha	Tillers per sq.m	Kernels per spike	Kernels wt. gm	Bio.Yield t /ha)	Harvest index	Water Applied cu.m
YR.	2	52.8**	61140.9**	879.7**	168.9**	6.5**	0.215**	52340068**
TRT.	2	13.2**	104444.0**	16.3	3.4	11.5	0.022**	86536275**
YR/TRT	4	11.5**	59710.7**	28.3	109.6**	66.3**	0.015**	10183358**
PIVOT	1	0.2	1013.4	5.7	7.6	3.9	0.013	11175**
YR/PIVOT	2	1.6	2323.2	15.2	46.8*	16.3*	0.011	28410**
TRT/PIVOT	2	1.3	2149.9	47.7	51.8**	24.0**	0.004	184730**
YR/TRT/PIVOT	4	1.4	22854.6*	48.6*	15.9	8.6	0.002	612772**

* Significant at the .05 level of probability .
** Significant at the .01 level of probability .

TABLE 4. The effect of three water treatments on different croop parameters during the three growing seasons.

TREATMENTS	Grain yield t /ha	Tillers per sq.m	Kernels per spike	Kernel wt. gm	Bio. Yield t /ha	Harvest index	Water Applied Cu.m/Ha
1	7.1 A	718.7 A	24.5 A	41.1 A	14.9 AB	0.477 A	4464 B
2	7.2 A	708.2 A	25.0 A	41.4 A	15.4 A	0.471 A	4369 B
3	6.5 B	663.1 B	24.3 A	41.5 A	14.8 A	0.451 B	5885 A
MEAN	6.9	696.6	24.6	41.3	15.0	0.446	4906
C.V	11.3	13.9	16.9	7.9	12.8	14.64	0
L.S.D	0.2	24.2	1.1	0.8	1.1	0.022	165

Means followed by the same letter are not significantly different at the .05 level of probability .

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ECONOMICS OF SPRINKLER IRRIGATION SYSTEMS IN THE GULF REGION

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ABSTRACT

Four computer programs were developed to design and analyze sprinkler irrigation systems. The first program estimates the reference crop evapotranspiration using the site climate data. The second program calculates the design daily irrigation requirement, the capacity, the operating pressure, and the spacing of sprinklers. The third program designs the laterals, the submains, and the mains for the solid and move sets systems. The fourth program evaluates the different alternatives plans economically and select the most economic plan.

INTRODUCTION

Sprinkler irrigation systems are widely used for irrigating the crops in Gulf countries. These systems suit the Gulf countries because of the water shortage, the texture of soil, and climate conditions. The design of a sprinkler irrigation system includes, the selection of the sprinklers to suite the crops and agriculture land, and the selection of pipes to carry the water from the source to the point of application. The design also includes the selection of pumps and water storage tanks. If the groundwater is the source of irrigation the designer should

define the location and depth of wells, the quantity and the quality of available water from wells.

In the early years of developing the sprinkler irrigation systems the main concerns of the designers were to estimate the head loss in the pipes and to identify their diameters. But now a days the main concern is to select the optimum design of the irrigation system, which meets the technical specifications with the minimum costs.

Taylor et al. (1985) developed an irrigation cost models to calculate the annual costs of irrigation for portable pipe, traveling gun(hose pull), and center-pivot systems. These cost models were used to generate data for econometric models, which were then used to evaluate the feasibility of large-scale

riparian irrigation expansion in Virginia. The cost models size the irrigation components based on simplifying assumptions about field shape, pipes orientation, and sprinklers layout in the field. Thus, detailed topographic data and information for considering different pipes or sprinklers geometries are not required. However, because of simplifying assumptions, the models cannot be used to obtained actual design specifications of irrigation systems. Kumar et al. (1992) modified Taylor's models for implementation as a component of knowledge-based system for preliminary selection and economic evaluation of sprinkler irrigation systems in Virginia.
The present research includes four computer programs written by Fortran and Quickbasic computer languages. These programs are developed to help the designers in selecting the best planning and design of an irrigation system. The programs also evaluate the systems economically and select the most economic sprinkler irrigation system.

DEVELOPMENT OF COMPUTER PROGRAMS

Four computer programs were developed to design, analyze and evaluate the sprinkler irrigation systems. These programs are explained in details in the following paragraphs:

THE FIRST PROGRAM

The first program estimates the reference crop evapotranspiration, ET_o, from the climate data using the Blany Criddel and Penman methods as given by Addink et al.(1980).
The input data for program 1 are: the average daily temperature during the time period, T, the number of days in the time period, N, mean daily percentage of annual daytime hours for the time period, P, minimum relative humidity, RH_{min}, in percent, ratio of actual to maximum sunshine hours, n/N, in decimal, observed solar radiation in evaporation equivalents, R_s, in mm./day and extraterrestrial solar radiation, R_a, in mm./day.
The preparation stage: based on the above data the program prepares, the saturated vapor pressure, e_{sa}, in mbar, at air temperature, T_a, in C°, the actual vapor pressure of air, e_a, in mbar, and the air pressure, P_a, in mbar.
The calculations stage: the program calculates the slope of the saturation vapor pressure versus temperature curve,Δ, at air temperature, T_a in mbar/C°, the psychrometric constant, γ, in mbar/C°, the aerodynamic term, E_a, in mm./day, and the net radiation, Q_n, in mm./day. The following equations are used to estimate the reference crop evapotranspiration using the Blany Criddel and Penman methods:
Equation for the modified Blany Criddel method is:

$$ET_0=N\left[\frac{a}{K_5}+bP\left(\frac{\overline{T}}{K_1}+K_2\right)\right] \qquad (1)$$

where,

a = $0.0043(RH_{min})-n/N-1.41$

b = $0.81917-0.0040922(RH_{min}+1.0705(n/N)+0.065649(U_{day}) - 0.0059684(RH_{min})n/N-0.0005967(RH_{min})\ U_{day}$

RH_{min} = minimum relative humidity, in percent;

n/N = ratio of actual to maximum sunshine hours, in decimal; and

U_{day} = daytime wind speed at 2 m height in m/s.

Equation for Penman method is:

$$ET_o=\frac{\Delta Q_n+\gamma E_a}{\Delta+\gamma} \tag{2}$$

where,

$$\Delta=\frac{4098\ e_{sa}}{(T_a+237.3)^2} \tag{3}$$

$$e_{sa}=Exp(\frac{19.08\ T_a+429.4}{T_a+237.3}) \tag{4}$$

$$\gamma=\frac{(1615\ P_a)}{2.49\ (10)^6\ -\ 2.31\ (10)^3\ T_a} \tag{5}$$

$$P_a=1013-0.1152h+5.44(10)^{-6}h^2 \tag{6}$$

$$E_a=(0.27+0.2333u)(e_{sa}-e_a) \tag{7}$$

$$Q_n=.75R_s-2.00(10)^{-9}(T_a+273.16)^4(0.34-0.044\sqrt{e_a})(-.35+1.8\frac{R_s}{R_a}) \tag{8}$$

$$R_a=1.2614(\frac{h_{do}}{(r_{ve}^2)}[h_s\frac{\Pi}{180}\sin(\Phi)\sin(\delta)+\cos(\Phi)\cos(\delta)\sin(h_s)] \tag{9}$$

$$h_{do}=12.126-1.85191(10)^{-3}ABS(\Phi)+7.61048(10)^{-5}(\Phi)^2 \tag{10}$$

$$r_{ve}=0.98387-1.11403(10)^{-4}(J)+5.2774(10)^{-6}J^2 - 2.68285(10)^{-8}(J)^3+3.6163(10)^{-11}(J)^4 \tag{11}$$

$$h_s = \cos^{-1}(-\tan(\Phi)\tan(\delta)) \tag{12}$$

$$\theta=.986(J-1) \tag{13}$$

$$\delta = \frac{180}{\Pi}(0.006918-0.399912\cos\theta+0.070257\sin\theta-0.006758\cos2\theta + 0.000907\sin2\theta-0.00697\cos3\theta-0.001480\sin3\theta) \tag{14}$$

where,

R_a = extraterrestrial radiation in mm./day;
h_{do} = daytime hours at zero declination in hr.;
r_{ve} = radius vector of earth;
h_s = sunrise to sunset hour angle in degrees;

Φ = location latitude in degrees (Φ is positive for north latitudes and negative for south latitudes);

δ = declination of sun in degrees;

J = days from Jan. 1 (e.g., J = 1 for Jan. 1, J = 2 for Jan. 2,...,J=365 for Dec.31);

θ = day of year expressed in degrees, (i.e.θ = 0 is Jan. 1,

θ = 90° is Apr. 2,θ = 180° is July 2,...)

The output of this program is a print of the reference crop evapotranspiration, ET_o, from the two methods the modified Blany Criddel and the Penman. The flow chart of program 1 is shown in Fig.1.

THE SECOND PROGRAM

The second program calculates the design daily requirement, (DDIR) based on the type of soil, the type of crops, and the climate data. Subsequently, from the wind speed, the selection of operating, and maintenance time the program sets the spacing between sprinklers and laterals. It also calculates the required sprinkler capacity. So that the suitable sprinklers can be selected from manufacture catalogues. The program has the ability to change the spacing between sprinklers and laterals and the operating time, till a suitable design is reached. Then it prints the output which are the sprinkler flow rate, the spacing between sprinklers, the spacing between laterals, and the sprinkler operating pressure. The flow chart of the program is shown in Fig. 2.

The input data, the calculation procedure, and the equations used in program 2 are summarized as follows:

The input data for program 2 are: sprinkler capacity, Q_s, in l/min., the soil texture, the type of crops, and the climate data.

Preparation stage: based on the soil texture the program selects the field capacity, F_c, permanent wilting point, PWP, and the infiltration rate, inf, as listed by Larry (1988). For each crop type the program selects the maximum root depth, D_{rz}, the maximum allowable depletion, MAD, and the crop coefficient, K_c, as listed by Larry (1988)

Calculations stage: the second program calculates the crop evapotranspiration using the following equation:

$$ET = K_c\ ET_o \tag{15}$$

where,

ET = crop evapotranspiration;

ET_o = reference crop evapotranspiration; and

K_c = crop coefficient.

The program calculates the readily available water, RAW, from the following equation:

$$RAW=MAD*D_{rz}\frac{(F_c-PWP)}{100} \tag{16}$$

where,

MAD = maximum allowable depletion in decimal = .65 for most of the crops;

D_{rz} = maximum root depth in mm.;

F_c = field capacity in percent by volume; and

PWP = permanent wilting point in percent by volume.

Following that the program uses the water budget technique Eqn.

Input data :

T_{mean}	= mean temperature in C^{o}.
$R.H._{min}$	= minimum relative humidity in %.
U_{dry}	= daily wind speed in m/s.
n	= sunshine hours.
N	= mean daily duration of maximum possible sunshine hours.
P	= mean daily percentage of annual day time hours for the time period.
T_a	= air temperature in C^{o}.
P_a	= atmospheric pressure in mbar (Eqn. 6).
ϕ	= latitude in degree.

Calculations of M.B.C. :

a	= Eqn. 1
b	= Eqn. 1
ET_o	= Eqn. 2

Calculations of Penman method :

e_{sa}	= Eqn. 4
Δ	= Eqn. 3
γ	= Eqn. 5
ζ	= Eqn. 13
δ	= Eqn. 14
h_s	= Eqn. 12
r_{ve}	= Eqn. 11
h_{do}	= Eqn. 10
R_a	= Eqn. 9
R_s	= Eqn. 8
e_a	= Eqn. 7
Q_n	= Eqn. 8
E_a	= Eqn. 7
ET_o	= Eqn. 2

Results :

Print ET_o = mm/day

Fig. 1 Flow Chart for Program No. 1.

17 to obtain the time between two successive irrigations, assuming that at the starting day the soil water contents equal to the field capacity, F_c. Then the program uses the same equation to estimate the water budget (water content) at the end of each day till the critical water content as given by Eqn. 18 is reached. So the time required to change the soil water content from the field capacity to the critical water content is equal to the minimum time between two successive irrigations.

$$\theta_i=\theta_{i-1}-\frac{100\,(ET-P_e)}{D_{rz}} \tag{17}$$

$$\theta_c=F_c-RAW*\frac{100}{D_{rz}} \tag{18}$$

where,
ET = crop evapotranspiration in mm.;
P_e = rain in mm.;
θ_i = water content at day, i in percent by volume; and
θ_c = critical water content.
Once the minimum time between two successive irrigation is defined the design daily irrigation requirement can be obtained as follows:

$$DDIR=\frac{RAW}{T_{min}} \tag{19}$$

where,
DDIR = design daily irrigation requirement in mm./day;
RAW = readily available water in mm.; and
T_{min} = minimum irrigation time interval during the irrigation season in days.
The time interval between the beginning of two successive irrigations of a given set in hours, can be obtained from equation 20. as follows:

$$H\leq\frac{0.24P_f*D}{DDIR} \tag{20}$$

where,
D = desired depth of irrigation in mm., (the water depth required to increase the soil water content from F_c to θ_c) usually it's equal to the readily available water, RAW;
DDIR = design daily irrigation requirement; and
P_f = percent of total field irrigated when the system is operated, equal to 100 for solid set system.
Once the time between to successive irrigations, H_1, is selected according to equation 20, the required depth of application for each irrigation, D_a, can be obtained as follows:

$$D_a=\frac{H_1*DDIR}{0.24*P_f} \tag{21}$$

where,
H_1 = the selected time between two successive irrigations, should be equal or less than H, in hr.
The sprinkler must have sufficient capacity to supply, D_a, in the irrigation time, H_2. Thus sprinkler capacity can be obtained as follows:

$$Q_s=\frac{K\ D_a L\ S}{(H_2-T_m)\,E_e} \tag{22}$$

where,
Q_s = sprinkler capacity in l/min.;
D_a = depth to be applied in each irrigation in mm.;
L = spacing between laterals in m.;
S = spacing between sprinklers in m.;
H_2 = irrigation time for a solid set and equal to time between successive two irrigations for set move system in hr.;
T_m = downtime for maintenance in hr.; and
E_e = application efficiency in percent, normally equal to 80%.
From the standard manufacture catalog a suitable sprinkler can be selected to supply discharge, Q_s, with wetted diameter, D_w, to satisfy the following formulae:

$$L \le K_L D_w \tag{23}$$

$$S \le K_S D_w \tag{24}$$

where, K_L and K_S are coefficients depend on the wind speed and the spacing patterns as given by Larry (1988). The second program has the ability to perform iterations procedure starting with different values of H_1 and goes through Eqn. 21 to 24 till acceptable values of Q_s, L, and S are reached and agreed with the manufacture catalogs. The required sprinkler pressure can be obtained from manufacture catalogs for the selected sprinkler. The last step in the sprinkler selection is to check the rate of application, with the soil infiltration rate to avoid the runoff. This check can be done as follows:

$$A=\frac{KQ_s}{LS}<infiltration\ rate \tag{25}$$

where,
A = application rate in mm/h;
Q_s = sprinkler flow rate in l/min.;
L, S = as defined before; and
K = constant = 60.
If the application rate, A, is greater than the infiltration rate then the time of irrigation, H_1, should be increased

consequently, the discharge, Q_s, will be reduced, so that the application rate, A, reaches a value less than the infiltration rate.
The output of the second program is the sprinkler capacity, Q_s, in l/min., sprinkler operating pressure, P_s, in KN/m^2, spacing between sprinklers, S, in m, and spacing between laterals, L, in m.

THE THIRD PROGRAM

The third program designs the pipelines for the irrigation system. The pipelines are classified as laterals, submains and mains. The mainlines convey water from the source and distribute it to the submains consequently to the laterals. The lengths of mains, submains and laterals are obtained from the planning which depends on the shape of the land as explained by Anderson (1980).
The input data for program 3 are: the sprinkler discharge, Q_s, in l/min., the operating pressure, P_s, in KN/m^2, the ground level along the pipelines, the type of pipelines, and a suggestion for the pipeline diameters. The program has the ability to use three different diameters along the lateral and ten different diameters along the mains and the submains.
Preparation stage: based on the above data the program selects the coefficient of Hazen and Williams, C, for each pipe as listed in Streater (1983), and the minimum pressure required to operate the sprinkler.
Calculations stage: the program calculates the total flow rate required for the system, the friction loss and the pressure required at the upstream of each pipeline (mains, submains, laterals). The calculations procedure is summarized in the following paragraphs:
For one lateral, the total flow rate is calculated as follows:

$$Q_l = N_s * Q_s \tag{26}$$

where,
Q_s = sprinkler flow rate in l/min as given from program 2; and
N_s = number of sprinkler along the lateral, as given from the layout.
The head loss along the lateral is calculated by dividing the length of the lateral into number of reaches equal to number of sprinklers minus one, N_s-1. The head loss of each reach can be calculated as follows:

$$H_{L_i} = \frac{K_1 S\,(iQ_s)^{1.85}}{C^{1.852} D_i^{4.87}} \tag{27}$$

where,
K_1 = constant = 589.414 x 10^3;
C = Hazen and Williams coefficient;
D_i = lateral diameter for the reach, i in mm.; and
Q_s = sprinkler flow rate in l/min.
The head loss in the reach next to the submain equal to:

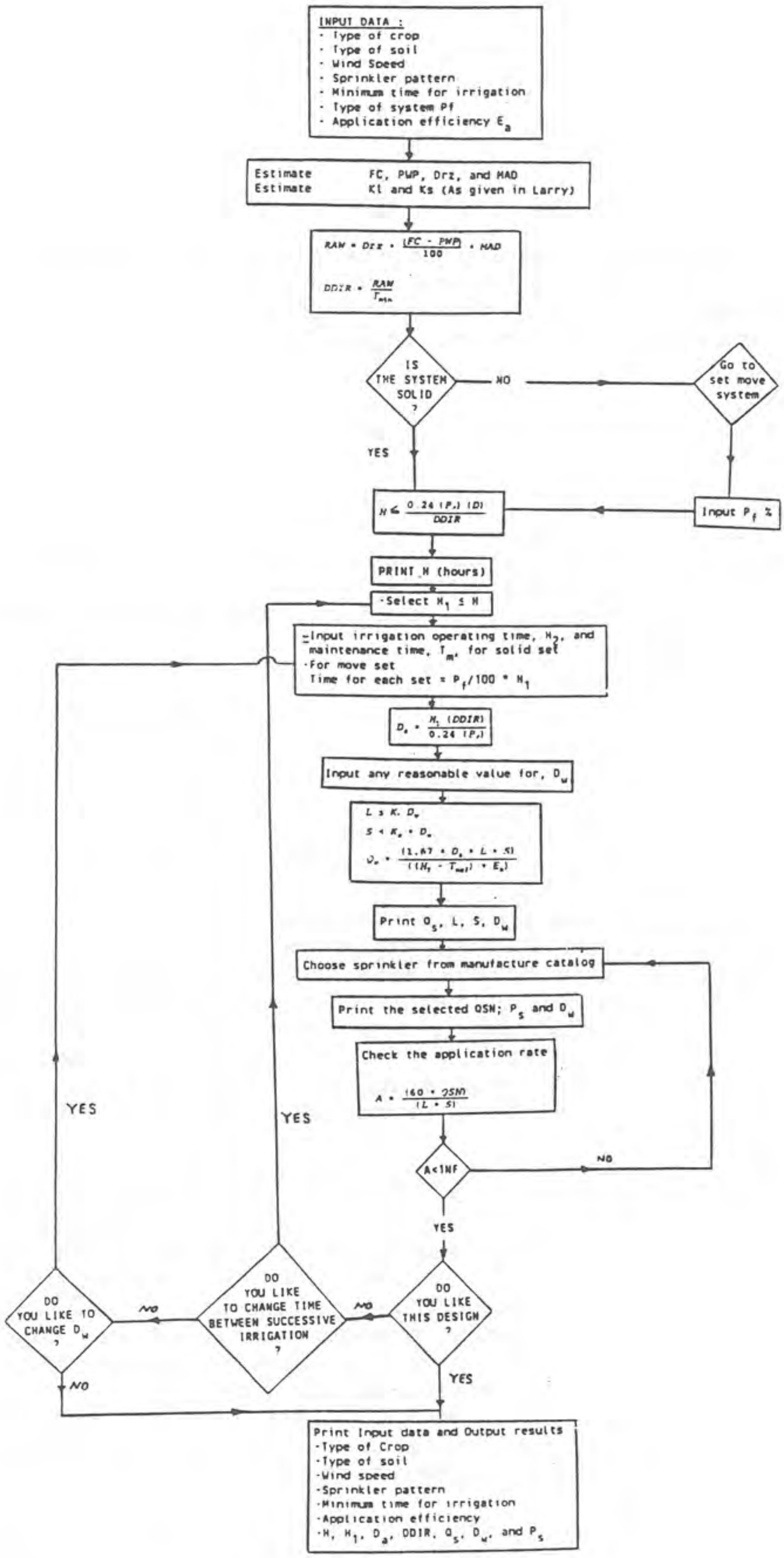

Fig. 2 Flow Chart for Program No. 2.

$$H_{L_s}=\frac{K_1 L_{ls} Q_l^{1.85}}{C^{1.852} D_1^{4.87}} \tag{28}$$

where,
L_{ls} = pipe length from submain to the first sprinkler in m.;
Q_l = lateral flow rate in l/min; and
D_1 = the diameter of the first reach from the lateral.
The total head loss though the lateral is:

$$H_{L_{total}}=H_{L_s}+\sum_{i=1}^{i=N_s-1} H_{L_i} \tag{29}$$

where,
N_s = number of sprinklers.
The head loss calculated from equation 29 is increased by 10 % to account for the secondary loss.
The pressure along the lateral is calculated for each reach using Eqn. 30 as follows:

$$P_i=P_{i-1}+\gamma H_{L_i}+(E(i-1)-E(i))*\gamma \tag{30}$$

where,
P_i = pressure at point, i on the lateral, at i = 1 , P_i = sprinkler operating pressure, this value is increased by 10 % to account for the riser losses;
P_{i-1} = pressure at the point downstream, i in Kw/m2;
E_i = elevation of point, i in m;
E_{i-1} = elevation of the point downstream point, i in m; and
γ = specific weight of water in KN/m^3.
During the calculation of the head loss the program checks the velocity and the pressure difference for each reach. The velocity was restricted to a maximum of 2.0 m/sec. through laterals and 2.5 m/sec. through submains and mains. Also the pressure difference between the beginning and the end of laterals, submains and mains were restricted to 20 % to reach a uniform distribution of water.
The program calculates the pressure at each sprinkler along the lateral till it reaches the submain. The same procedure in calculating the flow rate, the head loss and the pressure for laterals is followed for the submain and main till the locations of pumps and water source are reached. At this stage the program prints the required discharge and pressure at the pumps. It also prints discharges, head losses, pressures, diameters, lengths and elevations along the mains, submains, and laterals.
In the design of submains and mains the program has the ability to consider laterals operated from one or two sides of the submains or mains, which can be used for the case of set move systems. The flow chart of program No. 3 is shown in Fig. 3.

ECONOMIC EVALUATION:

The most important part of the design of an irrigation system is the selection of the most economic system. This required the determination of the ownership costs, the operating costs and the total costs, for each feasible design so that the most suitable system can be selected. The fourth program calculates the

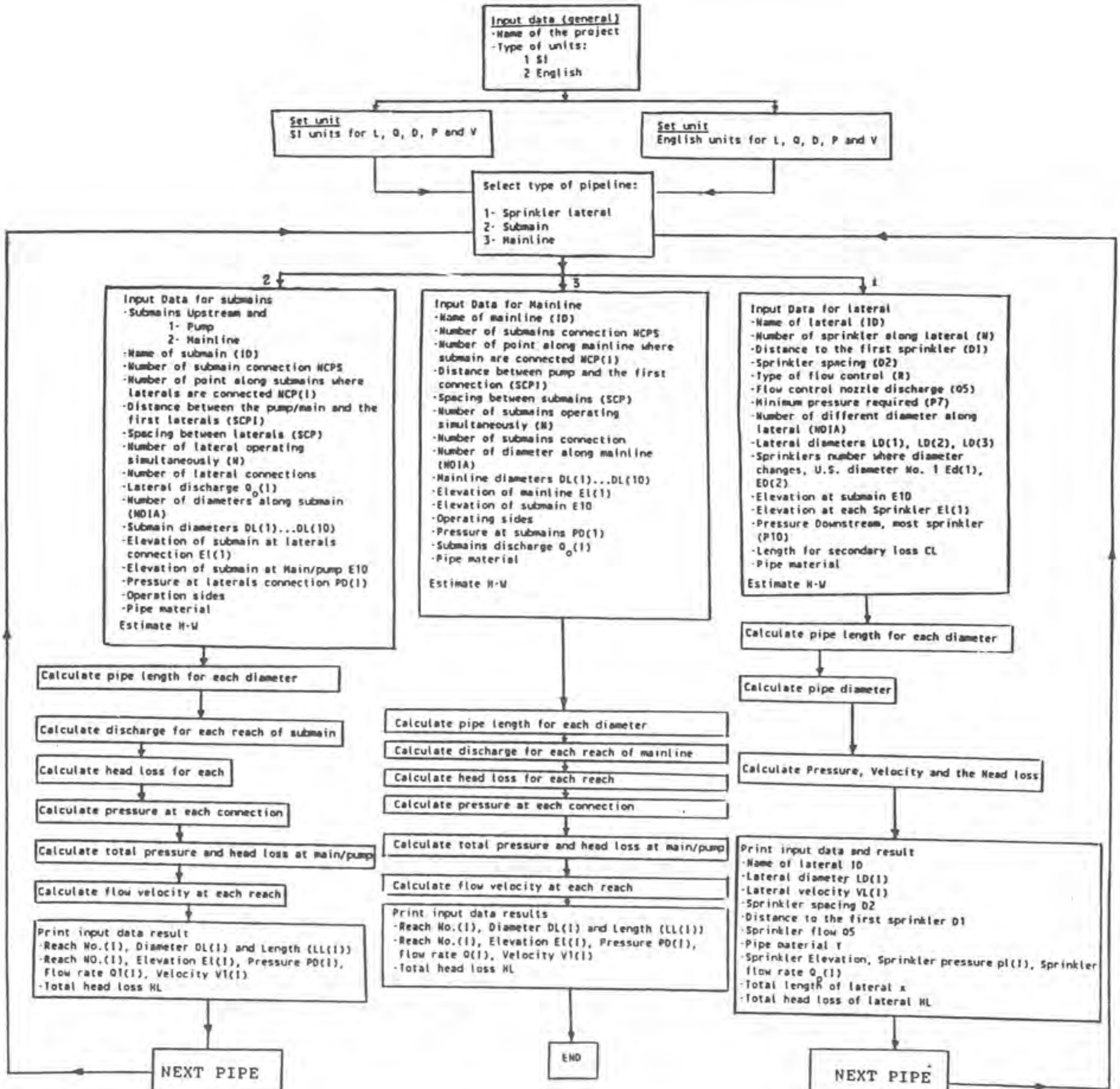

Fig. 3 Flow Chart for Program No. 3.

ownership costs, the operating costs and the total costs based on the above design information.

The fourth program input is: the output of the third program which includes the design data (sprinklers, pipelines diameters and lengths, pumps discharge, head and power, ...etc.). With these information the program calculates the initial costs for the system based on U.A.E. market as follows:

Initial costs of sprinklers in DH = Number of sprinklers x Cost per sprinkler in DH (31)

Initial costs of pipeline = Length of pipeline x Cost per m length of pipe for specific diameter in DH (32)

Initial costs of pump in DH = 51.269 + Pump power in Kw x Cost of pump per Kw (33)

where,

Pump power in Kw = Q P / C

C = constant = 49.2 x 10^3, with pump efficiency equal to 80%;

Q = pump flow rate in l/min.;

P = total pressure at pump in KN/m^2; and

the cost of pump/Kw in U.A.E = 300 DH for the year 1993.

The annual ownership costs include the depreciation costs and the interest costs for sprinklers, pipelines, fittings, valves, and pumps. They are calculated as follows:

$$ADIC = CRF\sum_{j=1}^{j=N_c} PW_j \qquad (34)$$

$$CRF = \frac{i(1+i)^{AP}}{(1+i)^{AP}-1} \qquad (35)$$

$$PW = IC \quad -SV\left(\frac{1+r}{1+i}\right)^{AP} \qquad (36)$$

where,

AP = analysis period years;

ADIC = annual depreciation and interest costs;

CRF = capital recovery factor;

SV = salvage values of components in DH; it is taken about 10% of the initial costs;

r = expected annual rate of cost escalation in decimal;

NC = number of system components (sprinklers, pipes, ..etc.);

PW = present worth of component in DH;

IC = initial cost of component in DH; and

i = annual interest rate in decimal.

When the analysis period, AP, is shorter than the components useful life, UL, the component will not be fully depreciated at the end of the analysis period. In this case, the final salvage value (at the end of the analysis period) is the sum of the un depreciated and salvage value. Therefore, the salvage value, SV_f equal to:

$$SV_f = IC - (IC - SV)\ \frac{AP}{UL}$$

and, SV_f should replace, SV, in equation (36).

In situations where analysis period, AP, exceeds the component useful life, UL, the component will need to be replaced one or more time during the analysis period. Therefore, the present worth of component is obtained as follows:

$$PW = IC + (IC - SV) \left[\sum_{J=1}^{N} \left(\frac{1+r}{1+i} \right)^{J(UL)} \right] - Z \left(\frac{1+r}{1+i} \right)^{AP} \tag{37}$$

where,

$$N = integer\ portion\ of = \frac{AP - 1}{UL}$$

$$Z = IC - (IC - SV) \left(\frac{AP - N(UL)}{UL} \right)$$

N = number of component replacement during the analysis period.

The analysis period for the study is 20 years, which is reasonable age for the pipes and pumps, but the age of sprinklers is taken 5 years. The annual operating costs include the costs of energy, maintenance, repair and labor. The cost of energy is calculated based on the pumps power in Kilowatt and number of operating hours per year. The cost of energy in U.A.E is taken 0.07 DH/Kwh for the year 1993. So the following formula is use to calculate the energy cost:

Energy cost = Pump power in Kw x Number of operating hours/year x cost of Kwh in Dh. (38)

The maintenance and repair costs for different components are obtained from standard tables as given in Larry (1988). The effect of the escalating costs can be included by multiplying estimated annual operating costs for the initial year of operation by the equivalent annual cost factor(EACF). EACF is defined by the following equation:

$$EACF = \frac{(1+r)^{AP} - (1+i)^{AP}}{(r-1)} \left(\frac{i}{(1+i)^{AP} - 1} \right) \tag{39}$$

The above equations give a complete economic analysis for the irrigation system. The flow chart for program No. 4 is shown in Fig. 4 this Fig. presents the calculations procedure for the fixed costs (ownership costs), operating and total costs.

APPLICATION

The four computer programs have been applied on a newly reclaimed land on U.A.E. The land is located about 70 Km. south of Abu-Dhabi. The area of land is about 13 hectare. Five different solid sets and one move set system plannings were proposed for this area. Figs. 5 to 9 show the four solid set plannings for the irrigation systems. Fig 10 shows the move set planning. The four computer programs were applied to design the irrigation system for this area. The outputs of the first three programs are the complete design of the six systems (capacity and pressure of sprinklers, diameter and length of laterals, submains and mains, and discharge, pressure and power of pumps).

The capacity, pressure of sprinklers and the diameters and lengths of pipes are shown in Figs. 5 to 10 for the six

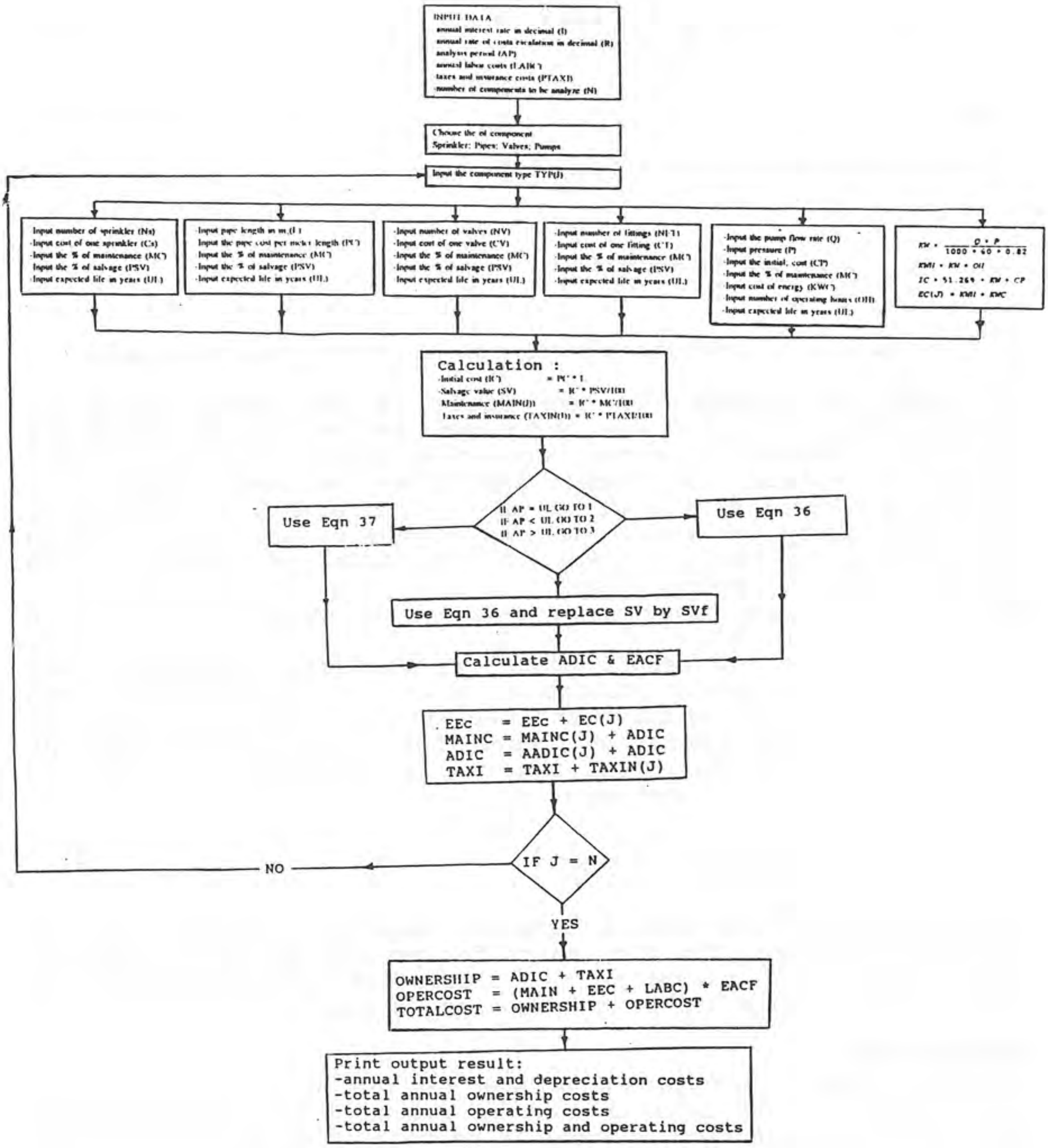

Fig. 4 Flow Chart for Program No. 4.

plannings.
These data are used as an input for the fourth program which goes through the economic analysis for each system and draw a graph as shown in Fig.11 between the annual costs, the operating costs and the total costs for each system. The program carries the economic analysis based on the cost of energy, pipes, sprinklers, pumps, labors, and maintenance in U.A.E. The labor costs for this site were assumed 4 works with salary 25 DH/day. As shown in Fig.11 system No. (5), gives the minimum cost for the solid set systems. This planning has been redesign as a set move system as shown in Fig. 10. The cost of this system has been reduce by 6.5% because the reduction in pipe diameters and pumps size. But the pump operating time has been increased instead of using 920 hours per year for the set solid system, to 3680 hours for the set move system. But still the move set system more economic than the solid set.

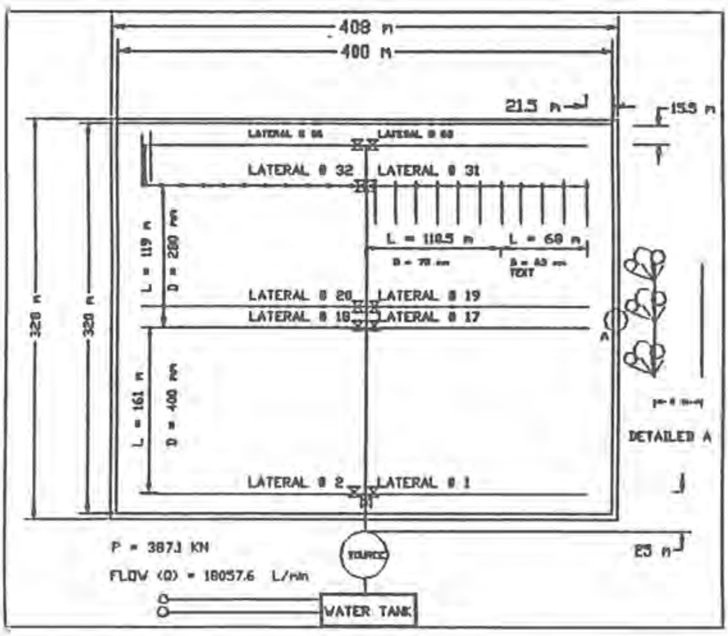

Fig. 5 Planning No. 1

CONCLUSION

Four computer programs were developed to design and evaluate the sprinkler irrigation systems economically that, the most economic sprinkler irrigation system can be selected. The first program calculates the reference crop evapotranspiration using the modified Blany Criddel and the Penman formulae based on the site meteorologic data. The second program calculates the capacity and operating pressure of sprinklers, and the spacing between sprinklers and laterals. The third program calculates the diameters and lengths of the laterals, the submains and the mains. It also calculates the head loss through the pipes and the pump head, discharge and power. The fourth program performs the economic and cost analyses for the different alternatives system design, and select the most economic one.

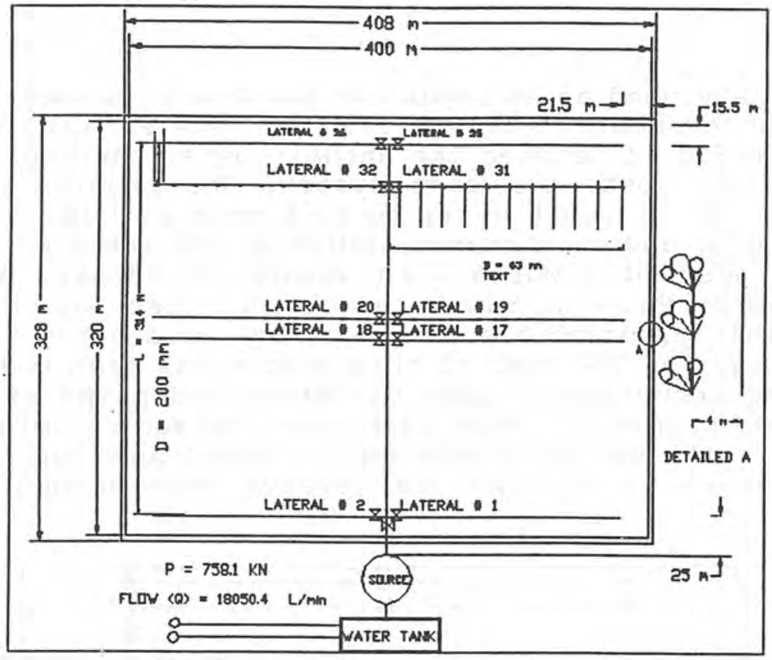

Fig. 6 Planning No. 2

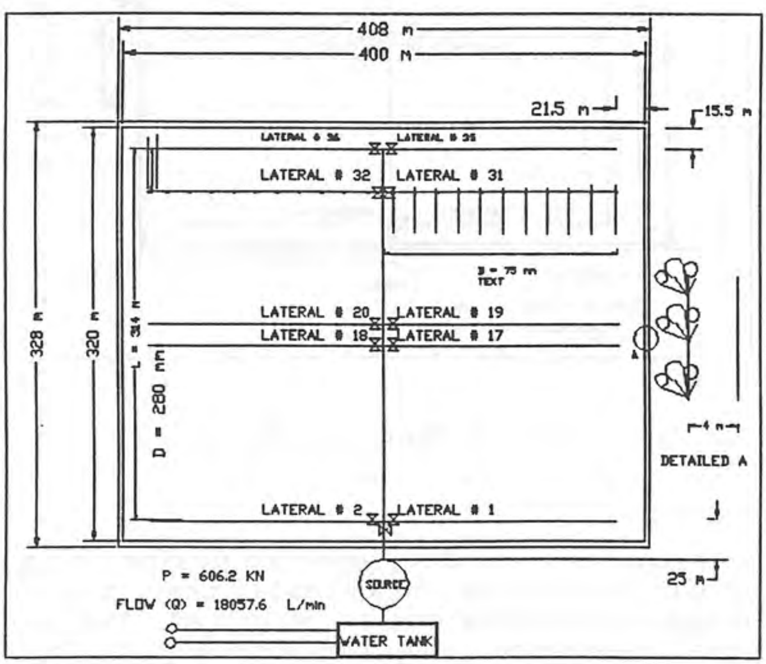

Fig. 7 Planning No. 3

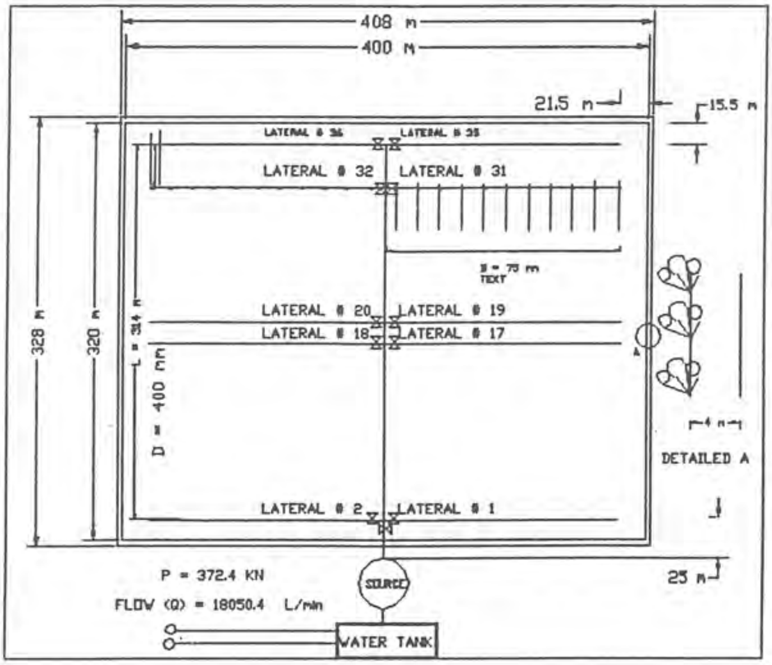

Fig. 8 Planning No. 4

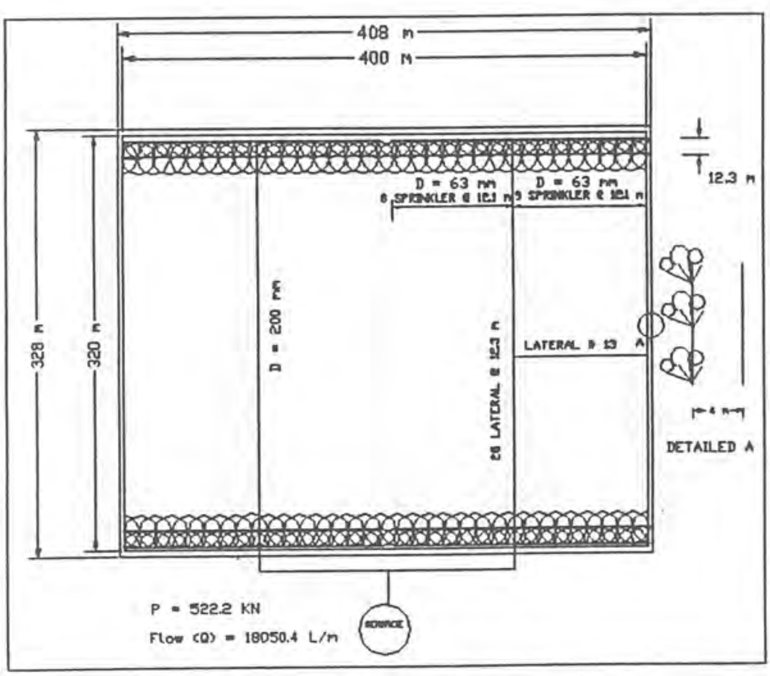

Fig. 9 Planning No. 5

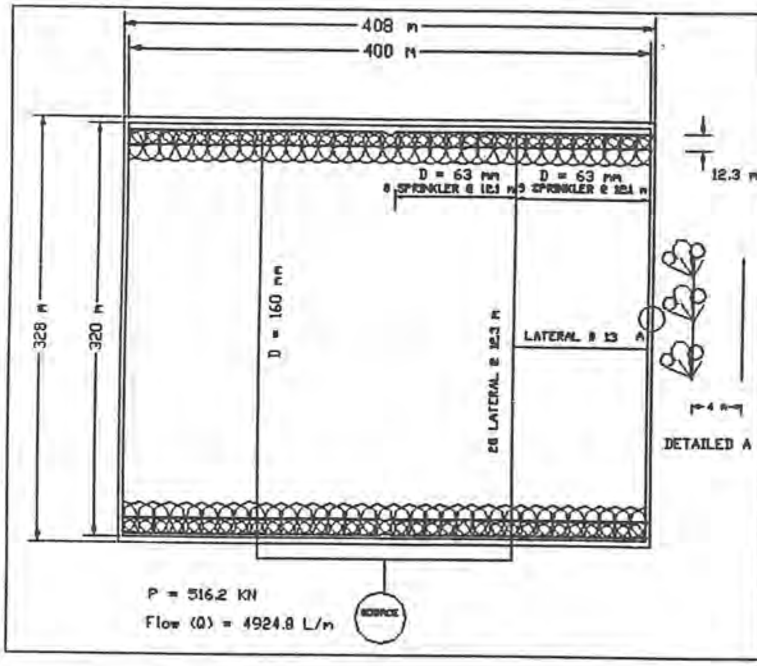

Fig. 10 Planning No. 6

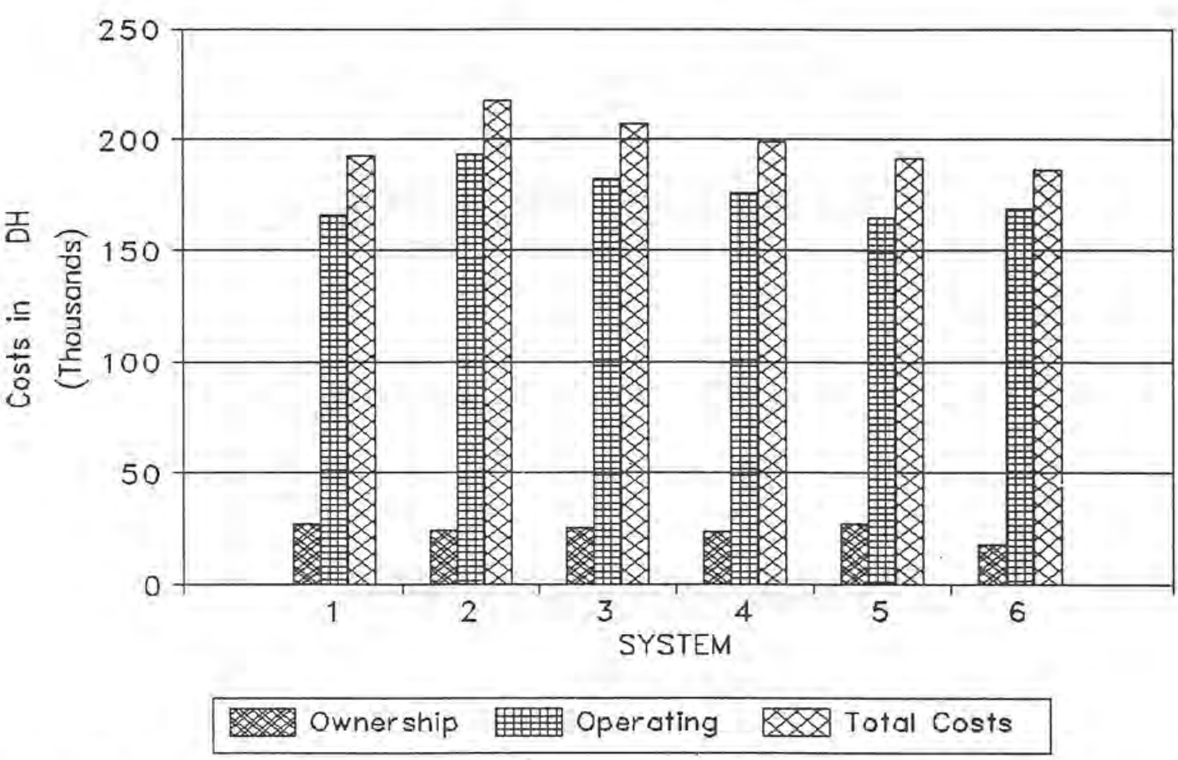

Fig. 11 Relation Between Annual Ownership, Operating and Total Costs.

APPENDIX 1 REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

ADIC = annual depreciation and interest costs;
AP = analysis period years;
C = constant = 589.414 x 10^3;
C = coefficient of Hazen and Williams;
CRF = capital recovery factor;
D = desired depth of irrigation in mm.;
D_a = depth to be applied in each irrigation in mm.;
DDIR = design daily irrigation requirement mm./day;
D_{rz} = maximum root depth in mm.;
E_a = aerodynamic term in mm./day;
E_e = application efficiency in (percent);
E_i = elevation of point i;
E_{i-1} = elevation of the point downstream point i;
ET = crop evapotranspiration in mm.;
ET_o = reference crop evapotranspiration in;
F_c = field capacity in percent by volume;
H = time interval between the beginnings of two successive irrigations of a given set in hr;
H_1 = the selected time between two successive irrigations in hr;
H_2 = irrigation time for a solid set and equal to time between successive two irrigations for set move system in hr;
i = annual interest rate in decimal;
IC = initial cost of component in DH;
L = spacing between laterals in m.;
MAD = maximum allowable depletion in decimal;
N = number of days in time period;
NC = number of system components (sprinklers, pipes, ..etc.);
n/N = actual maximum sunshine hours;
P = total pressure at pump in KN/m^2;
P_e = the rain in mm.;
P_f = percent of total field irrigated;
P_i = pressure at point i;
P_{i-1} = pressure at the point downstream i;

PW = present worth of component in DH;
PWP = permanent wilting point in percent by volume;
Q = pump flow rate in l/min;
Q_s = sprinkler capacity in l/min;
r = expected annual rate of cost escalation in decimal;
RAW = readily available water in mm.;
S = spacing between sprinklers on a lateral in m.;
SV = salvage values of components in DH;
T_m = downtime for maintenance in hr;
T_{min} = minimum irrigation time interval in days;
γ = specific weight of water KN/m^3;
Θ_c = the critical water content; and
Θ_i = the water content at day i in percent by volume.

Using Lysimeters To Develop Evapotranspiration Crop Coefficient's Under Arid Climatic Conditions

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Using Lysimeters to Develop Evapotranspiration Crop Coefficients under Arid Climatic Conditions

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Abstract: Crop coefficient curves and equations have been developed under arid climatic conditions for Riyadh area. The crop coefficients were based on evaporation from A pan, Penman and J-H methods. Three drainage type lysimeters were installed at the educational farm of the College of Agriculture and planted with wheat for one growing season in 1992. The weather data were collected from the Research Station of Agriculture College at Dirab and Ministry of Agriculture and Water, Riyadh. Crop coefficients for different locations were determined and presented in the form of equations and graphs. The study showed that a significant variation occured due to location, climate and topographic conditions for different methods. For proper planning and design of irrigation projects, it is necessary to develop local crop coefficients.

Introduction

Estimates of crop evapotranspiration have practical application in the scheduling of irrigation, the modelling of crop yield in relation to crop water use, and in irrigation project planning and management.

In practice, the estimation of evapotranspiration (ET) for a certain crop involves first calculating the potential or reference ET and then applying suitable crop coefficient.

A number, of approaches are available for determining crop coefficients. Usually crop coefficients are based on either actual measurements of ET from a reference crop or that estimated from equations and from some standard evaporating devices such as evaporation pans. A number of equations are available for estimating reference evapotranspiration (ET_r), such as modified Penman equation including FAO Penman (Doorenbos and Pruitt, 1977), Kimberly-Penman (Jensen, 1990) and Penman- Monteith as presented by Jensen, et. al, 1990), Jensen-Haise equation as given by Jensen (1990), Thornthwaite (1948) and others. Either the grass based evapotranspiration (ET_0) or alfalfa based ET (ET_r) may be estimated.

The most reliable approach for estimating crop ET (ET_c) or crop coefficient (K_c) is the direct measurement of ET by using lysimeters. Lysimeters are also useful in the development or verification of reference ET techniques and in the development of specific crop coefficient curves (Wright, 1982).

The reference crop (either grass or alfalfa) is grown in lysimeters and the measured ET is used in developing equations for estimating reference ET or calibrating the existing equations. Other techniques used may be neutron scattering or gravimetric methods.

Crop evapotranspiration is related to reference evapotranspiration by a dimensionless crop coefficient for the particular crop at the estimating growth stage and surface soil water conditions (Wright, 1982) such that

$$ET_c = K_c \cdot ET_r \quad \text{...} (1)$$

where K_c is the dimensionless daily crop coefficient, ET_c is the daily crop ET (mm/day) and ET_r is the daily reference ET (mm/day). The actual evapotranspiration data obtained from a certain crop grown in lysimeters could

be used in this equation for obtaining improved estimates and hence developing K_c charts. Lysimeters have been used in developing functional relationships and crop coefficients to estimate crop ET (Wright, 1991 and Pruitt,1991).

Daily values of K_c are obtained either using the generalized basal crop coefficients K_{cb} given by Wright (1979) or mean crop coefficients. The basal crop coefficients represent conditions when the soil surface is dry and the evaporation from the surface is minimal but the availability of soil water does not limit plant growth. To account for the effects of surface soil wetness, difference in soil drying properties, and available soil water, the following equation is used:

$$K_c = K_{cb} \, K_a + K_s \quad \text{.........(2)}$$

in which K_c is the daily crop coefficient, K_{cb} is the daily basal crop coefficient, K_a is a coefficient dependant upon available soil moisture and taken as 1 unless available soil water limits transpiration in which case it has a value less than 1 and K_s is a coefficient to allow for increased evaporation from the soil surface occuring after rain or irrigation. When soil surface is wet, $K_s > 0$ and $K_c > K_{cb}$ and when the soil surface is dry with minimal evaporation but the available soil water does not limit plant growth or transpiration, $K_c = K_{cb}$ since $K_a = 1$ and $K_s = 0$. When available water within the root zone limits growth and evapotranspiration K_a is < 1 and $K_c < K_{cb}$ when $K_s = 0$. (Jensen, 1990).

The functional relationship of K_a and available soil water is as follows:

$$K_a = \ln(A_w + 1) / \ln(101) \quad \text{.........(3)}$$

in which A_w is the percentage of available soil water. When soil is at field capacity, $A_w = 100$ and $K_a=1$; but as A_w approaches 0, K_a goes to 0.

The above relationship depends on soil water properties and crop rooting patterns. Therefore suitable relationships should be used for each soil. Based on field measurements, Boonyatharokol and Walker (1979) recommended the following variations of K_a :

$K_a = 1$ for $A_w > 50\ \%$

$K_a = A_w/50$ for $A_w \leq 50\ \%$

Jensen, et al. (1971) suggested the following equation to approximate K_s under Kimberly (Idaho) conditions:

$$K_s = (K_1 . K_{ci})\ e^{-\xi t} \quad \text{for } K_1 > K_{ci} \text{ ,} \qquad (4)$$

in which t is the number of days after a rain or irrigation, ξ represents the combined effects of soil characteristics and evaporation demand, and K_{ci} is the value of K_c at the time the rain or irrigation occured. The value of K_1 will vary for various soils and locations.

Wright (1981) developed the following general equation for K_c based on K_{cb} and the effects of surface soil wetness, soil drying properties and irrigation methods:

$$K_c = K_a K_{cb} + (K_1 - K_a K_{cb}) \left[1 - (t / t_d)^{1/2} \right] f_w \quad \text{for } t \leq t_d \qquad (5)$$

where t is the number of days after major rain or irrigation, t_d is the usual number of days required for the soil surface to visually appear dry, t_d is about 5 or more after thorough wetting of the soil, 3 days ar less for sandy soils and 7 days or more for clay loam soils and f_w is the relative portion of the soil surface originally wetted by furrow irrigation.

For $K_a = 1$ the equation will be as follows:

$$K_c = K_{cb} + (1 - K_{cb}) [1 - (t / t_d)^{1/2}] f_w \text{ ....(6)}$$

for surface irrigation, and sprinkler irrigation which wets the entire surface f_w = 1 and for furrow irrigation $f_w = 0.5$.

A mean crop coefficient may be more useful than a basal crop coefficient for estimating daily crop ET. This is due to the impracticality to estimate wet soil effects or when seasonal estimates of water requirements are needed. When a mean crop coefficient is used, no adjustment is needed for the effects of surface soil wetness or increased evaporation. When soil moisture availability is limited, a correction to K_c can be made, using K_a as follows:

$$K_c = K_a \, K_{cm} \text{ ....(7)}$$

Mean crop coefficients can be determined from the lysimeter ET data that were used to derive the basal crop coefficients. Also soil water balance data can be used to develop K_{cm} values for time periods of several days if daily lysimeter data are unavailable.

In practice reference ET for a certain locality is usually estimated from suitable equations based on meteorological data. Alfalfa reference ET_r has been suggested to be used under arid conditions (Jensen, et. al 1971, 1990; Wright and Jensen 1972, and Wright 1979, 1981,1982).

The modified form of Penman combination equation (Jensen et. al. 1990) used is:

$$ET_r = [\frac{\Delta}{\Delta + \gamma}(R_n - G) + \frac{\gamma}{\Delta + \gamma} W_f (e_s - e_d)] \text{ ....(8)}$$

where Δ is the slope of the saturation vapour pressure and temperature curve, γ is the psychrometric constant , R_n is net radiation (mm/day), G is soil heat

flux (mm/day), W_f is wind function dependant on daily wind travel (U in km/day) and the nature of the selected reference surface (alfalfa or grass), and $e_s - e_d$ is the mean daily saturation vapor-pressure deficit.

Another procedure for estimating ET_r from climatic data is the Jensen-Haise (J-H) Method. The modified J-H method is as follows:

$$ET_r = C_T \ (T - T_x) \ R_s \ \text{..........} (9)$$

$$C_T = \frac{1}{C_1 + 7.3C_H} \ ; \quad C_1 = 38 - \frac{2E}{305} \ ; \quad C_H = \frac{50\text{mb}}{e_2 - e_1} \qquad \text{and}$$

$$T_x = -2.5 - (e_2 - e_1) - E/550$$

where ET_r is the alfalfa reference ET in mm/day, E site elevation in meters, e_2 the saturation vapour pressure of water in mb at the mean monthly maximum air temperature of the warmest month in the year (long term climatic data), and e_1 is the saturation vapour pressure of water in mb at the mean monthly minimum air temperature of the warmest month in the year (long term climatic data), T is the temperature in C° , and R_s is solar radiation (mm/day) measured or estimated.

The reference crop ET may also be estimated with reference to evaporation from class A pan by using the following equation:

$$ET = K_p \ E_p \qquad \text{..........} (10)$$

where K_p is the dimensionless pan coefficient, E_p is the evaporation from class A pan and ET is the reference crop ET with respect to grass or alfalfa in the same units as E_p.

The main objective of this paper was to develop crop coefficients for wheat crop under local conditions to predict crop water requirements for proper irrigation scheduling and efficient use of irrigation water.

Materials and Methods

Three identical drainage type lysimeters were used in this study. These lysimeters were installed at the educational farm of the College of Agriculture, King Saud University. They were located in the middle of a half hectare experimental field planted with the same crop, i.e. wheat. Each lysimeter had a surface area of 4 m^2 (2x2 m) and a total depth of 2 m. The cross section is shown in fig. 1. Each lysimeter was provided with a drainage system to collect the excess water. The lysimeters were refilled with soil in 15 cm layers

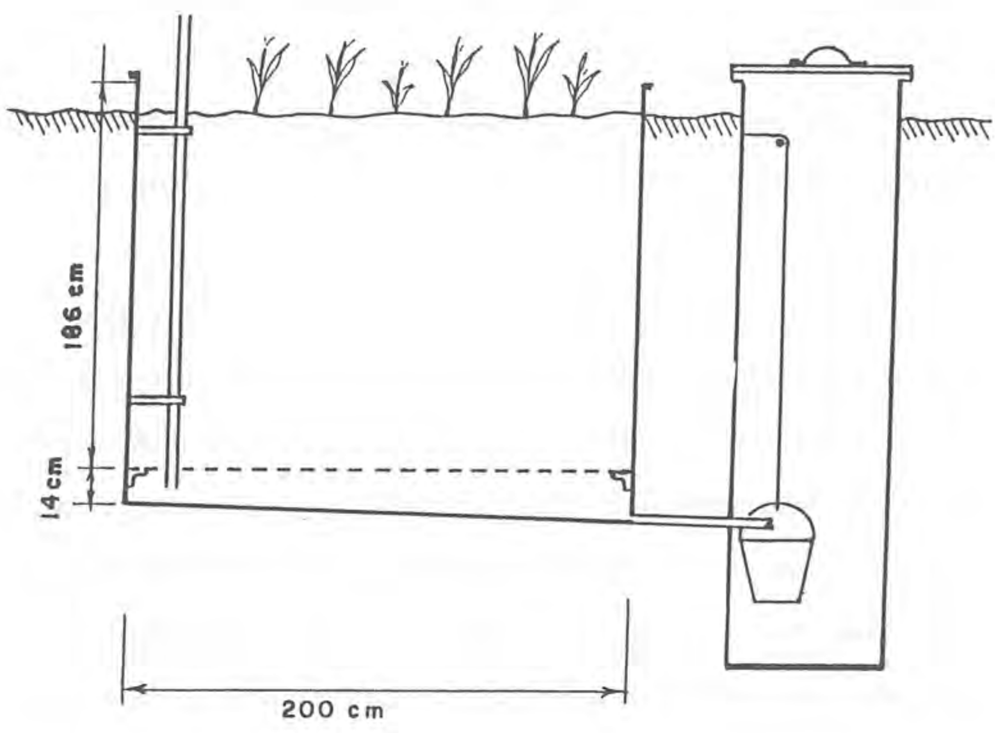

Fig. 1: Sectional view of the drainage type lysimeter for measuring ET of crops.

successively and carefully compacted. The soil was sandy loam and the final bulk density obtained was 1.55 gm/cm^3. Pre-irrigation was carried out after reconstructing the profile to ensure proper settlement and arrive at approximate natural conditions and to facilitate seed bed preparations for planting. The lysimeters and the surrounding plots were planted with a wheat variety of Yecora Rojo having four growth stages, namely early, development, mid-season and maturity of about 15, 40, 60 and 20 days respectively and commonly grown in the area. The planting was carried out on Dec 20, 1992 at a seeding rate of 180 kg/ha. The seeds were drilled by hand in rows 150 mm apart and were carefully covered with soil. Fertilizer was applied before planting at the rate of 100 N, 300 P_2O_5 and 30 K_2O kg/ha. Irrigation was carried out immediately after planting to get good germination. A good germination stand was obtained in the lysimeters and the surrounding fields after 10 days. Tensiometers were provided in each lysimeter and irrigation was carried out when the suction reached about 30-35 kPa. The water was applied through conventional surface irrigation method and measured with pre-calibrated flow meters. The surrounding fields were irrigated with the same method at the same time.

Evaporation data were collected daily from U.S. Weather Bureau class A pan situated about 100 m from the lysimeters. Other meteorological data were collected from the weather station of the Agricultural Research Station of K.S.U. in Dirab about 25 km South of Riyadh. Data were also obtained for Riyadh from the Ministry of Agriculture and Water (Hydrology Section). The data consisted of daily temperatures, relative humidity, wind speed and direction, solar radiation, vapor pressure and evaporation from class A pan.

The measaured ET from lysimeters were obtained by using the following water balance equation:

$$ET = I + R - D \pm \Delta W \quad \text{...(11)}$$

where ET was the measured ET in mm/day, I the amount of irrigation water applied during the period of observation, R the rainfall in mm, if any during the period, D the drainage water during the same period and ΔW is the change in stored water.

Initially, the water content of the profile were raised to field capacity and ET was calculated between two successive drainage events, i.e. when drainage water was obtained after irrigation. Under these conditions the change in stored water or ΔW is negligible.

The alfalfa reference ET or ET_r was estimated with Penman equation (i.e. equation 8) using wind function as given by Wright (1982). The site elevation was 650 m m.s.l. and e_2 and e_1 corresponding to the mean normal monthly maximum and minimum temperatures were 84.17 and 36.93 mb (max. normal temperature of hottest month during July was 42.5 °C and minimum normal temperasture of July was 27.6 °C) respectively. These values were used in J-H equation to obtain C_T and T_X . Since the Penman and J-H equations were calibrated earlier for the same site with measured values of ET_{20} or ET_r, crop coefficients were also obtaied with these calibrated equations in the analysis.

Results and Discussions

The crop coefficients for wheat crop, with respect to class A pan evaporation data taken at the educational farm, were determined and plotted in Fig. 2. The

four growth stages are distinctly clear on this figure. The crop coefficient during the initial stage (germination and early growth) is constant and equals 0.3. From the end of this stage, the coefficient increases almost linearly until full cover is attained. During this stage, the crop coefficient reaches the value of 0.8. and is given by the following equation:

$$K_c = 0.013\ N + 0.08 \qquad (R^2 = 0.97) \text{...} (12)$$

where N represents the number of days since planting. During the mid-season and maturity stages the crop coefficient remains constant at 0.81. At the end of this stage, a drop takes place in K_c and continues to reach a value of 0.12 at the time of harvest. In general, the obtained K_c curve followed the same pattern and the values were close to those presented by Doorenbos and Pruitt (1977). The lower values of pan crop coefficients could be due to the interaction of several factors affecting evaporation. These factors include the response of pan to sensible heat advection as compared to crops, the size of pan, the pan envirnment in relation to nearby surface, and the climate. Under dry and windy conditions, similar results have been reported by Jensen (1990).

The ET_r values were estimated from Penman and J-H equations using climatic data from Riyadh. With the measured values of crop ET for wheat, the crop coefficients were determined and presented in fig. 3. The general trend is the same for both. Crop coefficients in the first stage are constant and equal 0.28 and 0.5 for Penman and J-H equations, respectively. For the second stage, the crop coefficients are represented by the following equations:

$$K_c = 0.014\ N; \quad (R^2 = 0.98) \qquad \text{for Penman ....} (13)$$

$$K_c = 0.022\ N; \quad (R^2 = 0.97) \qquad \text{for J-H equation ....} (14)$$

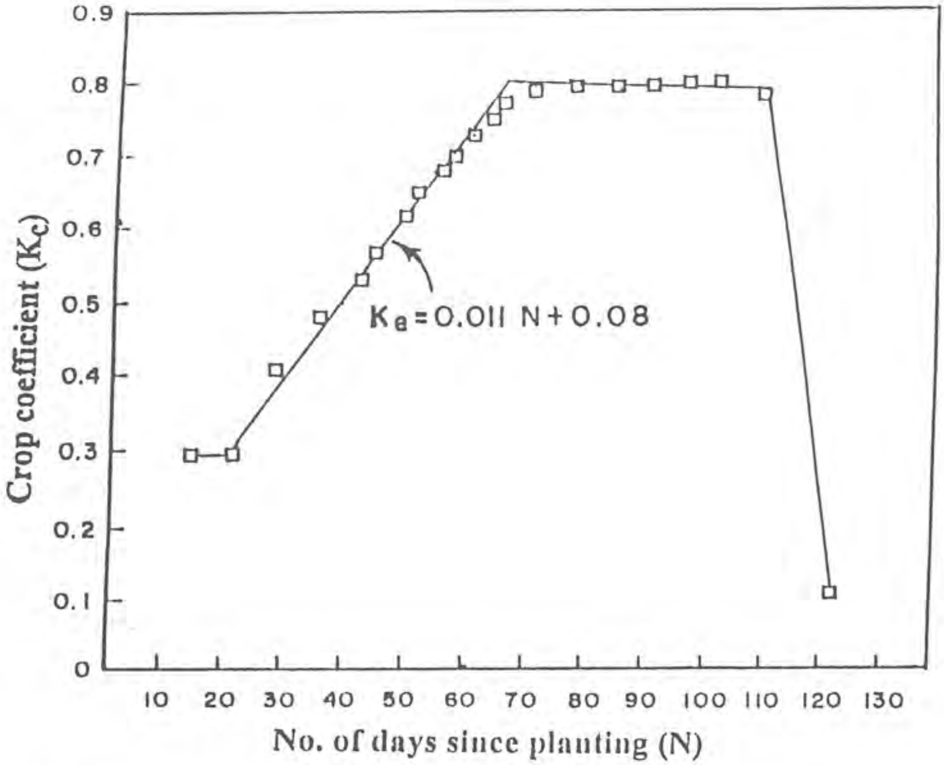

Fig. 2: Crop coefficients with respect to evaporation from class A pan at educational farm of College of Agriculture, Riyadh.

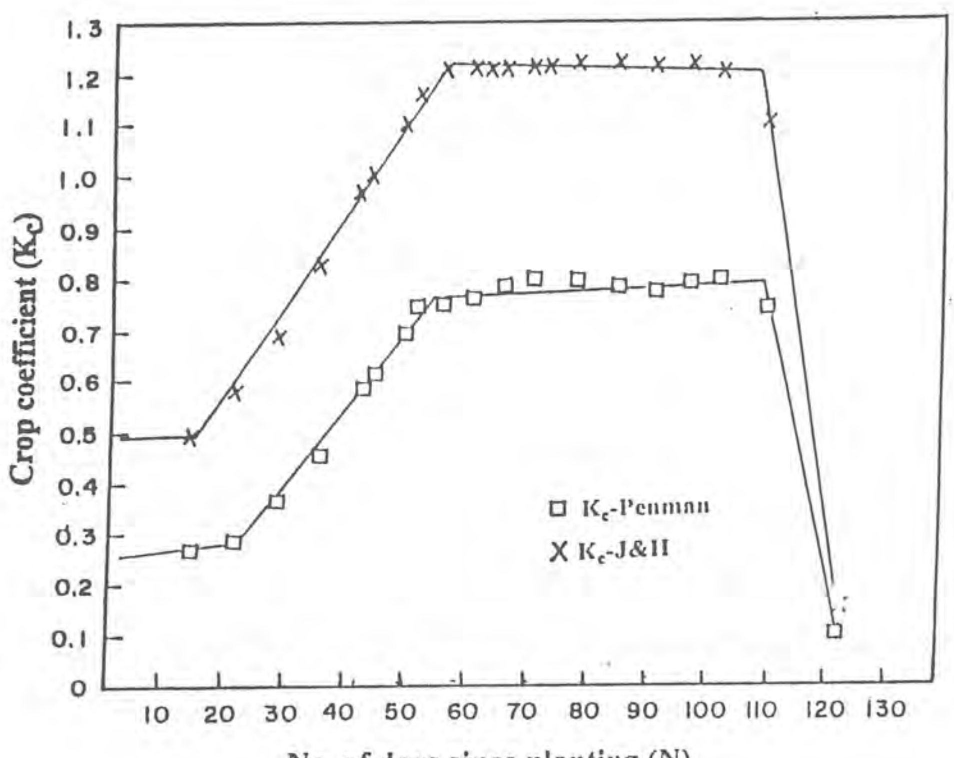

Fig. 3: Crop coefficients with respect to ET estimated from Penman and J-H equations using Riyadh weather data.

The coefficients remained constant at 0.78 and 1.2 for the mid-season and maturity stages for Penman and J-H respectively.

The crop coefficients as given by J-H equation are higher than those given by Penman equation for all stages. This means that the crop coefficients obtained with the J-H are overestimated. This is because the determined ET_r is underestimated by this equation as it is mainly based on solar radiation and does not involve the advective energy which is important in evapotranspiration process. The crop coefficients obtained by Penman and J-H methods during the entire growth period are greater than those obtained from pan evaporation.

The crop coefficients obtained with the Penman and J-H equations under local conditions during the first and second stages are less than those obtained by Wright (1979) as reported by Jensen (1983) under arid conditions at Kimberly, Idaho, U.S.A. In the third and fourth stages the crop coefficients obtained by the J-H method are more and those by Penman are less. Therefore, it is evident that crop coefficients must be developed to suit local conditions rather than using published data. Similar findings are obtained for Dirab location. The crop coefficients with Dirab data are shown in fig. 4. The crop coefficients during the initial stage are 0.22 and 0.42 for Penman and J-H, respectively. In the second stage, the cropefficients are given by:

$K_c = 0.008\ N + 0.125 \quad (R^2 = 0.987)$ for Penman ....(15)

$K_c = 0.018\ N + 0.216 \quad (R^2 = 0.99)$ for J-H equation ....(16)

The maximum difference of J-H is about 120 % from Penman. This vast difference is due to the topography of the location. This area is situated in a valley surrounded on all sides by hills. The crop coefficients at Dirab and Riyadh are almost the same because they receive almost the same amount of radiation. The crop coefficients with Penman equation at Riyadh site are greater than those obtained at Dirab. The maximum difference is during the third stage and is about 60%. This may be due to the effect of the difference in advective energy at both locations.

Crop coefficient values were determined using calibrated Penman and J-H equations for the site, i.e. the educational farm of the College of Agriculture. The equations were based on one year data for alfalfa crop and were as follows:

$ET_{20} = 0.96\ ET_{Penman}$ $(R^2 = 0.58)$ for Penman

$ET_{20} = 1.16\ ET_{J\text{-}H}$ $(R^2 = 0.69)$ for J-H equation

where ET_{20} is the ET from 20 cm tall alfalfa stand or reference ET.

The crop coefficients were plotted alongwith Wright's values (as given by Jensen, 1983). As seen from fig. 5 the Penman equation gave closer values to that from Wright. The small deviations from Wright during all the stages is due to the differences in locations and length of growing seasons. The crop coefficient value with Penman during the initial stage was 0.7 and with J-H as 0.5. During the second stage the following crop coefficients could be used to arrive at a good estimate of crop ET (wheat only).

$Kc = 0.018\ N$ $(R = 0.92)$ for Penman ....(17)

$K_c = 0.021\ N$ $(R^2 = 0.97)$ for J-H equation ....(18)

During the mid-season and maturity stages the crop coefficients remain constant and are given as 0.85 and 1.25 for Penman and J-H equations, respectively.

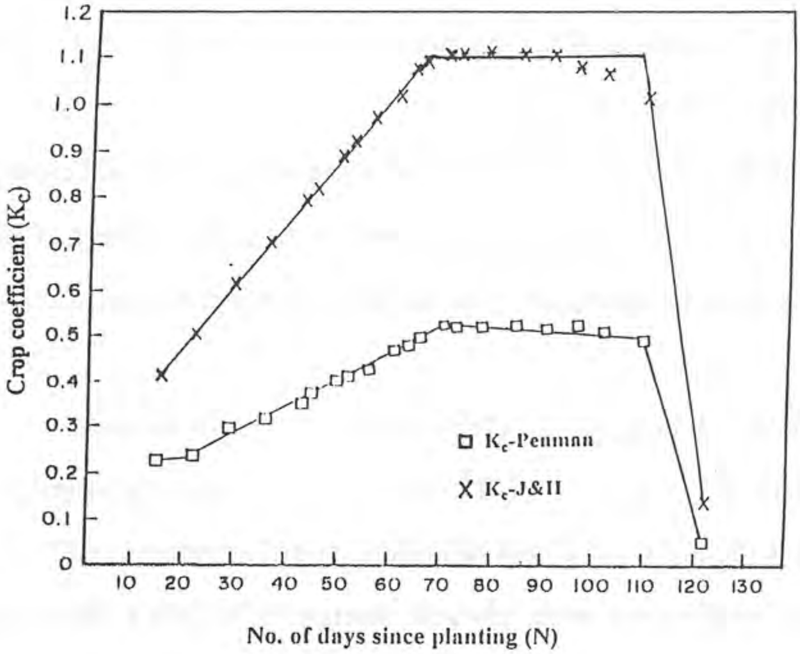

Fig. 4: Crop coefficients with respect to ET estimated from Penman and J-H equations using Dirab weather data.

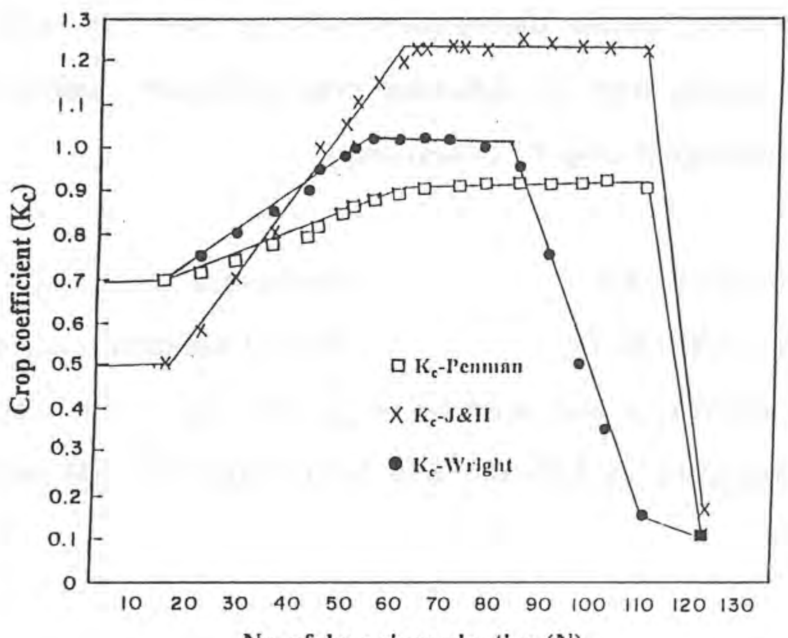

Fig. 5: Crop coefficients with respect to ET estimated from Penman and J-H equations calibrated for the site.

The crop coefficients given above are related only to the Penman and J-H equations, respectively and should not be used interchangeably.

Conclusion

A set of equations and crop coefficient curves for wheat under arid conditions were presented. These equations could be used in estimating more accurately the crop evapotranspiration under arid climatic conditions. A significant variation in crop coefficient values was found between the methods and location.

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Session - 9
Waste Water

High Efficient Process For Ammonia Removal From Wastewaters and Industrial Effluents

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ABSTRACT

It is usually desirable to have low ammonia concentration in the final effluent streams of the treatment processes due to its harmful toxicity even in the low concentration level.

Various methods are available for ammonia removal like air stripping, ion exchange and chlorination. Among all these processes, air stripping is the simplest, more economic and the easier to control. It is also the most suitable technology for efficient removal in hot countries like the Gulf area. Hence, it is applied in the present work for ammonia removal in packed strippers.

In this investigation, an aqueous solution of ammonia is used to simulate the waste stream. This solution was then stripped in a packed column using two different types of packing. Air was blown counter currently through the stripping column. The exit ammonia concentration was measured electrochemically by a special selective electrode. Very high removal efficiencies, more than 90%, were achieved during the experimental work.

Various operating parameters like liquid flow rate, gas flow rate and ammonia influent concentrations were extensively studied.

Results obtained were also discussed and analyzed in terms of the design parameters like the height of transfer unit (H.T.U) and the number of transfer units (N.T.U).

A general design correlation was also obtained using a computer program and regression analysis including the whole collected data. The achieved design equaiton is :

$$H.T.U_{og} = \alpha\, G^{\beta}\, L^{\gamma}\, C^{\theta}\, D^{\mu}$$

Recommendations for future work are finally given.

Introduction

Among various nitrogen compounds, (nitrates, nitrites and ammonia), ammonia is the most important one to be controlled because of its abundance in domestic and industrial wastewaters. Furthermore, urea and certain complex organic compounds such as proteins or free amino acids may ultimately yield ammonia upon hydrolysis.

Processes for the removal of these nitrogen compounds are not as well established as those of phosphates. Among the applied processes which are being used on a plant scale for nitrogen removal, one can mention the following (1):

a) Ammonia stripping
b) Selective ion exchange
c) Biological nitrification and de-nitrification
d) Break-point chlorination

These processes are usually applied in the tertiary treatment steps of waste water and stripping is still the simplest one and the easiest to control (1,2).

Packed towers have been used for gas-liquid absorption and stripping for many decades in the chemical process industries, but their application in water treatment is relatively recent(3).

Originally, the concept of removal of volatile contaminants from water dates back to the beginning of this century. Hazen(4) indicated that aeration is one of the cheapest methods for removing volatile contaminants from water.

Analysis of the literature studies revealed also that most of research work in this area concentrates on the stripping efficiency of various stripping contactors like spray columns, packed columns and diffused air equipment. In addition, effects of various operating parameters like gas and liquid flow rates, influent concentration and temperature are considered in these studies. Some examples of these studies are given and discussed here below:

Shpint (5) analyzed ammonia removal from water in diffused aeration towers. The analysis is performed as a function of water and air flow rates as well as influent concentration. An empirical formula was derived using the dimensional analysis technique to determine the dependence of the mass transfer coefficient on gas loading rate, concentration and column dimensions.

Prather (6,7,8) reported several studies in which ammonia was removed from petroleum refinery wastewater by stripping with air. The influent concentration of ammonia in his work was around 100 mg/liter and a 85% removal efficiency was achieved at pH values of 10.5. Slechta and Culp (9), Stander and Van Vuuren (10), Culp (11) and Cooper (12) reported various studies carried out at the South Lake Tahoe Reclamation plant (California) in order to investigate the potential of using packed column air stripping for ammonia removal. Removal efficiencies of about 95% were obtained in these studies either in the small scale experiments or in pilot plant investigations. However, some operational problems like calcium carbonate scales depositions and icing in cold conditions were reported for packed tower ammonia stripping.

Apart from experimental studies mentioned earlier, limited theoretical work on the prediction of ammonia removal in packed columns by air stripping was found in the literature. Recently, Standinger and co-workers (13) investigated the application of mass transfer correlations on the experimental data of air stripping of volatile organic compounds from wastewater. Their experimental data involved 4 types of packing used in the air strippers and more than 10 types of volatile organic contaminants. Their developed correlations could predict mass transfer values within 17% deviation from the actual experimental data.

The analysis presented in this study focuses on the determination of the effects of the operating parameters on the stripping of ammonia from water in a counter-current packed column. The analysis is carried out over a wide range of ammonia concentrations and liquid and gas flow rates. Results are presented in terms of ammonia removal efficiency, height and number of transfer units. A general correlation is also given for the whole experimental results, collected in the present work.

Experimental System

The stripping column used in the present work is made of plexiglass tube with an inner diameter of 0.075 m, a total height of 1.2 m and a packing height of 1.0 m. The column is fitted with four lines for gas and liquid streams. A schematic diagram of the apparatus is shown in figure (1).

Two types of packing material are used in the present experimental runs; the first is a metallic Raschig rings made of steel wire (as shown in Fig. 1.A) while the second is a ceramic Raschig rings (Fig. 1.B). The metallic packing has a diameter of 0.008 m and a height of 0.008 m while the ceramic packing is 0.02 m diameter and 0.02 m height.

The column porosity was also measured and its average value was 0.8.

In all experimental runs, ammonia concentration in the liquid stream was measured by an electrochemical technique in which a special ion selective electrode is applied for this purpose (14).

Operating Parameters

The stripping column is operated at various conditions in order to study the factors affecting the stripping process. These conditions include (1) gas flow rate, (2) liquid flow rate and (3) liquid influent concentration. The analysis is performed at room temperature, where both liquid and gas temperatures are kept at $25 + 1^oC$.

The parameters range used in the experiments include:

- liquid flow rates of $(0.4 — 1.4) \times 10^{-6}\ m^3/s$
- gas flow rates of $(0.5 — 2.5) \times 10^{-3}\ m^3/s$
- ammonia concentrations of 100 - 1000 ppm

Model Equations

Analysis of the stripping process is achieved through calculations of the removal efficiency, which is defined by the relation:

$$\eta = [\frac{X_i - X_o}{Xi}] * 100 \qquad (1)$$

The above equation assumes that the overall liquid mass flow rate does not vary between inlet and exit points of the tower. This assumption is realistic because rates of water evaporation to the air stream are negligible at low operating temperature, i.e. 25^oC.

In addition to calculations of the removal efficiency, the separation process is also characterized by calculations of the number of transfer units, height of a transfer unit and the overall mass transfer coefficients. The number of transfer units for gaseous stripping of a single component in a dilute system is determined from the equation (13,15,16):

$$N.T.U_{og} = (\frac{1}{S-1}) \ln [1 + (S-1) \frac{y_o}{y_1^*}] \qquad (2)$$

The height of the transfer unit is directly obtained by dividing the column height by the number of the transfer units, i.e.,

$$H.T.U_{og} = \frac{Z}{N.T.U_g} \qquad (3)$$

The overall mass transfer coefficient based on the gas phase is determined from the relation (15,16):

$$K_g\ a = \frac{V/A}{H.T.U_{og}\ P} \qquad (4)$$

similarly, the overall mass transfer coefficient based on the liquid phase is determined from (15,16):

$$K_L\, a = \frac{L/A}{H.T.U_{oL}\ \rho L} \qquad (5)$$

Results and Discussion

Due to the significant effects of the operating parameters like gas flow rate, liquid flow rates and ammonia concentration on the stripping operation, more attention is given to these parameters in the present work. The collected data are presented in terms of the dependence of ammonia removal efficiency, number of the transfer units and height of the transfer units on the above mentioned parameters.

The data displayed in Figs. (2), (3) show variation in ammonia removal efficiency with the gas and liquid flow rates at inlet ammonia concentration of 100 ppm for both metallic and ceramic packing respectively.

As shown in these figures the column removal efficiency increases with the increase of the gas flow rate and the decease of the liquid flow rate for both types of packing.

Very high removal efficiencies (> 90%) are achieved for the smallest liquid flow rate investigated ($0.4 \times 10^{-6}\ m^3/s$) in the case of metallic packing while for ceramic packing the obtained efficiency is (> 80%) at the same liquid flow rate. This relatively drop in efficiency for ceramic packing is expected due to its large dimensions and lower specific surface area.

The higher removal efficiency obtained in the present work (> 90%) especially for metallic packing, at the lowest liquid rate and highest gas rate investigated, is in reasonable agreement with the similar studies appeared in the literature of this field (6-12).

It is worth mentioning that, the same behaviour of the dependence of the removal efficiency on both gas and liquid flow rates (as given by Figs. 2,3) is also obtained for the higher influent ammonia concentrations. Although a slight reduction in the removal efficiency was detected at higher concentration levels due to the limited height of the present column (1.2 m for the column and 1.0 m for the packing) and also to expected increase in mass transfer resistance of both phases at that high levels of ammonia concentrations.

From the practical point of view, the flow rates ratio (G/L) is more important for the designer to have a reasonable operating

(G/L) and only one sample of these plots is displayed in figure (4) for both the metalic and ceramic packing. Fig (4) indicates the direct increase in the removal efficiency with increasing the ratio (G/L) which was also concluded from previous plots more over the highest efficiency (> 95%) values are achieved at (G/L) > 4000 for both metalic and ceramic packing of the present stripper.

It is worth mentioning that; at lower values of (G/L), the metalic packing has higher values of the removal efficiency (as shown in Fig. 4) than the ceramic packing which is expeced due to the lower specific surface area of the larger ceramic packing compared to the smaller metalic type. But at higher (G/L) ratios this effect is reduced due to high air flow rates in the stripper which has stronger stripping effect on ammonia removal.

In order to find the sensitivity of the design parameter $H.T.U_{og}$ on the operating parameter (G/L) and the influent concentration level of ammonia, Figs. (5) and (6) are plotted for both types of packing applied. It is clear from these figures that $H.T.U_{og}$ increases with increasing the (G/L) ratio for all concentration levels investigated and also for the two types of packing used. The increase of $H.T.U_{og}$ by increasing the gas flow rate is expected according to the following equaiton:

$$H.T.U_{og} = \frac{G/A}{K_g.a.p.}$$

In which direct proportionality between $H.T.U._{og}$ and G is prevailing.

On the other hand, decreasing the liquid flow rate may reduce the contact mass transfer area (9) in the previous equation and hence causes the expected increas in $(H.T.U_{og})$ with decreasing the liquid rate L in the ratio (G/L).

Due to the fixed height of the present stripper and according to equation (3),the $N.T.U_{og}$ varies in the opposite direction to the $H.T.U_{og}$ and hence its sensitivity curves W.R.T. (G/L) and concentration have the reverse trends of figs. (5 and 6). The $(N.T.U)_{og}$ data are displayed in Figs. (7) and (8) as samples only for the 100 ppm concentration. These figures indicate the expected effect of direct decrease of $N.T.U_{og}$ with increasing the ratio (G/L) which is opposite to the given trends of Figs. (5) and (6) at the same concentration level of ammonia (100 ppm).

In order to obtain a fruitful design correlation from the present work in a more general and useful expression for stripping column design, the whole data collected are fed to the computer and the following model was suggested for regression analysis and fitting:

$$H.T.U_{og} = \alpha\, G^{\beta}\, L^{\gamma}\, C^{\theta}\, D^{\mu}$$

where G,L, C are the investigated operating parameters and D is the equivalent characteristic length of the type of packing used. But $(H.T.U)_{og}$ is selected as the main design parameter of the stripping column as ususally considered by the designers in this field (3,15,16,17). The coefficients $\alpha, \beta, \gamma, \theta, \mu$ are determined by the computer program for the best fitting of the whole experimental data of the present work. The following output was obtained from the fitting program:

$\alpha = 0.00929$ $\quad\quad \beta = 1.1292$

$\gamma = -0.65518$ $\quad\quad \theta = 0.22489$ $\quad\quad \mu = 0.15694$

The regression coefficient R which reflects the accuracy of this fitting was found to be 88%. Moreover, the experimental data expressed as $(H.T.U)_{exp}$ (calculated from each experimental run using equations no (2) and no. (3)) were also compared with the (H.T.U.) predicted from the expressed suggested model by the fitting program as $(H.T.U)_{pre}$ for the whole runs of the present work and Fig.(9) was plotted between $H.T.U_{exp}$ and $H.T.U_{pre}$ and it was found that most of the data (about 90%) lies within + 30 % of the suggested model prediction which reflects the strength and the benefit of the present design correlation for stripping column design in the field of ammonia removal.

It is worth mentioning that, up to our knowledge, the present general correlation appears in literature to the first time, hence it cannot be compared with any other design correlation in the field of ammonia stripping.

The present correlation reveals also the stronger effect of the gas and liquid flow rates compared to the relatively weaker effect of both the packing type and the influent ammonia concentration.

Conclusions

The present work led to the following conclusions:

1- High removal efficiency (> 95%) are achieved by the present technique for ammonia stripping especially at high gas rates and low liquid rates.

2- Air stripping technique is an efficient and easy tool for ammonia stripping even at high ammonia concentration levels (up to 1000 ppm) which simulates some industrial waste effluents.

3- Ammonia removal efficiency is sensitive to the flow rate ratio (G/L) and it exceeds the 95% efficiency for (G/L) > 4000.

4- Higher values of $H.T.U_g$ are obtained at lower values of liquid rates and higher values of gas rates due to inefficient mixing between phases inside the stripper.

5- A general correlation for the whole experimental data has been achieved by a computer program using regresion analysis which led to the following design equation:

$$(H.T.U)_{og} = \alpha\, G^{\beta}\, L^{\gamma}\, C^{\theta}\, D^{\mu}$$

where

$\alpha = 0.00929$

$\beta = 1.1292$

$\gamma = -0.65518$

$\theta = 0.22489$

$\mu = 0.15694$

6- the performance of the stripping technique is more sensitive to gas and liquid flow rates rather than ammonia concentration or packing type.

Recommendations

1- The present work needs to be extended towards its applications in the removal of other organic volatile contaminants from actual industrial effluents especially in petroleum refineries and petrochemical plants for the control of its waste streams.

2- Temperature effect has to be investigated especially the hot conditions prevailing in the Arabian Gulf which supports strongly the validity of this stripping techniques which is accelerated at hot environments.

3- The present general correlation needs to be applied in the recent computer programs available for actual design of stripping towers for ammonia removal.

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Nomenclautre

A	Cross sectional area of packed column, m^2.
D_p	Equivalent diameter of the packing, m.
HTU_{og}	Overall height of transfer unit based on the gas phase, m.
HTU_{oL}	Overall height of transfer units based on the liquid phase, m.
K_ga	Overall mass transfer coefficient for the gas phase, kg/(m^3 s Pa).
K_la	Overall mass transfer coefficient for the liquid pahse, s^{-1}
L	Liquid flow rate, kg/s.
m	Henery's constant
NTU_g	Number of transfer units based on the gas phase.
P	Gas pressure, kPa.
S	Stripping factor, $S = mL/G$.
G	Gas flow rate, kg/s.
X_i	Ammonia mass fraction in influent liquid.
X_o^*	Ammonia mass fraction in effluent liquid.
Y_i	Ammonia mass fraction in influent gas, in equilibrium with ammonia mass fraction in effluent liquid
Y_o	Ammonia mass fraction in effluent gas
Z	Packing height, m.
ρ	Liquid density, kg/m^3.
η	Removal efficiency of ammonia

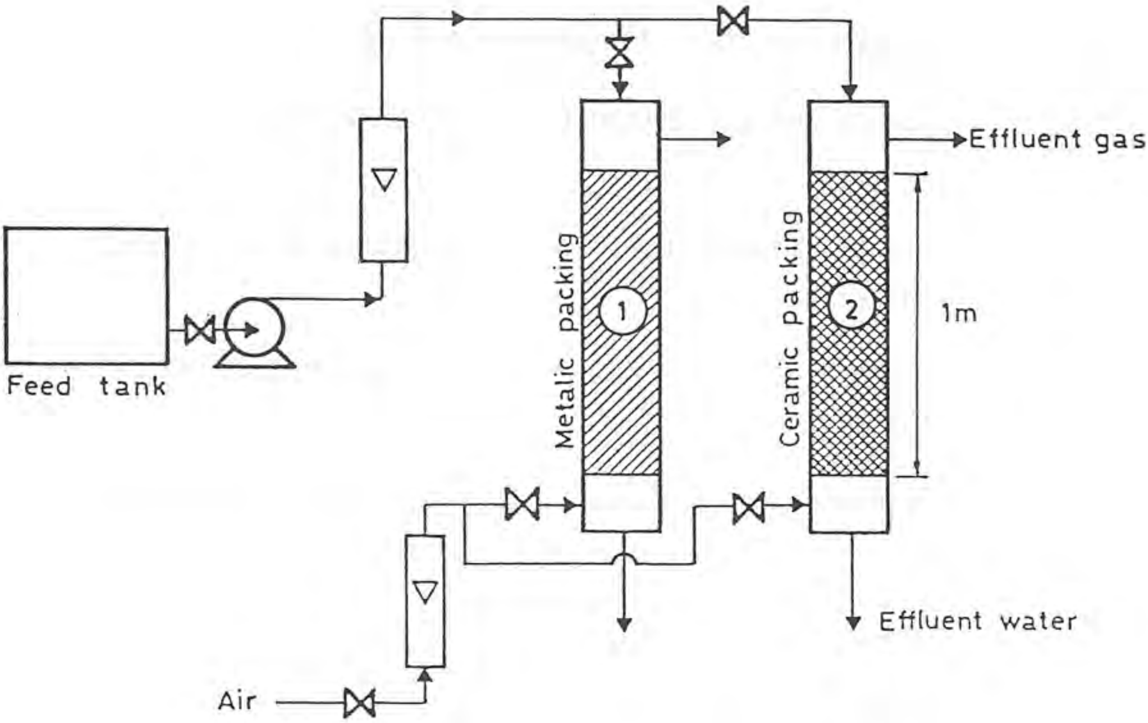

Fig.1: Schematic diagram of stripping system

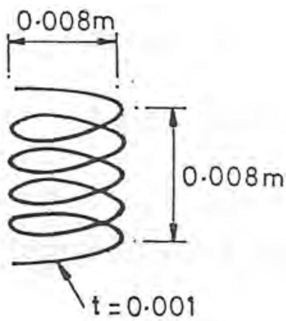

Fig1.a Metalic packing

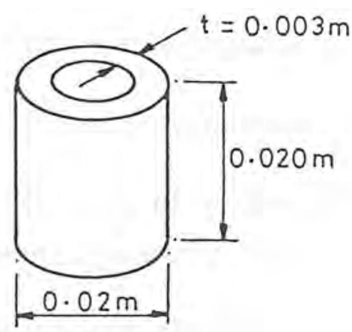

Fig.1.b Ceramic packing

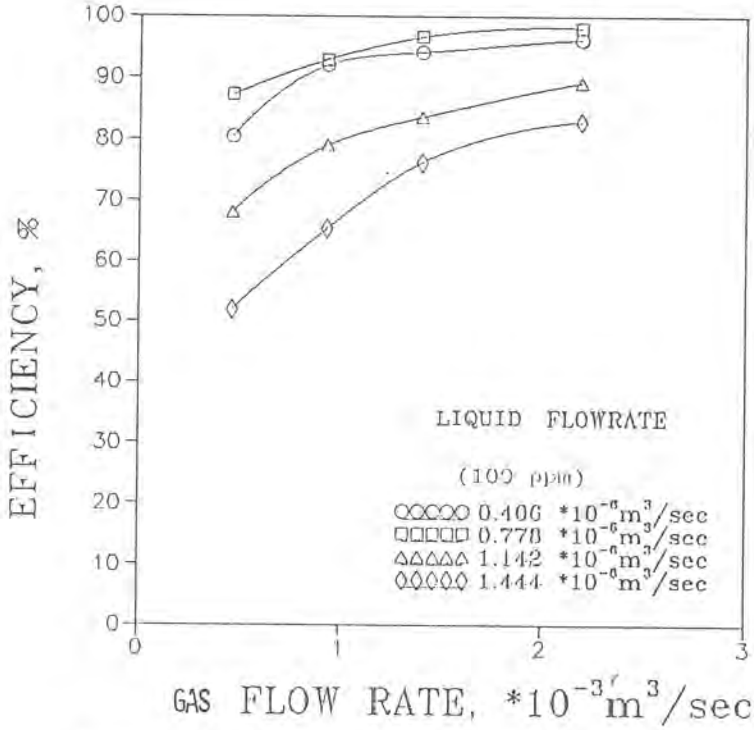

Fig.(2): Dependance of removal efficiency on gas flow rates at 100 ppm concentration NH_3 (Metal packing)

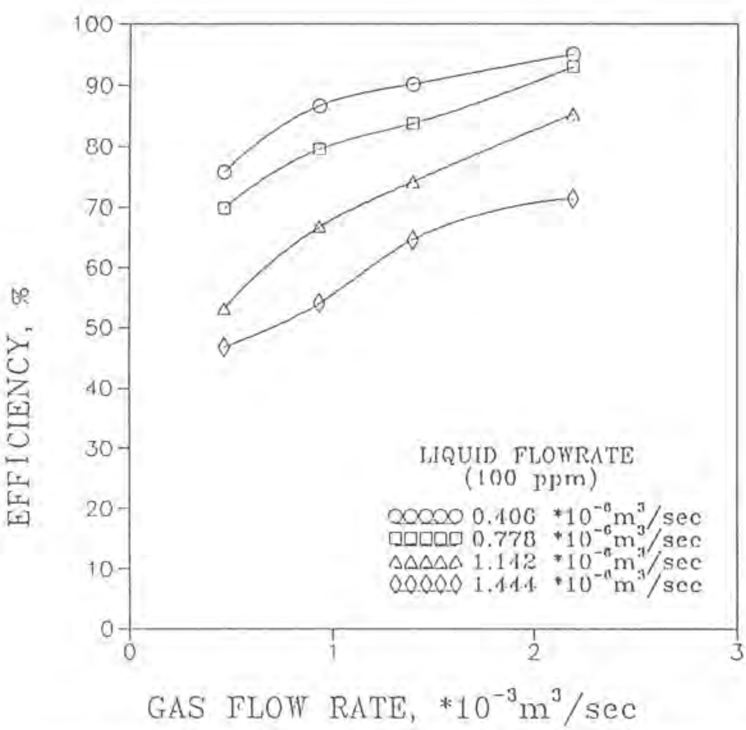

Fig.(3) : Dependance ofRemoval Efficiency on Gas Flow Rates at 100 ppm NH_3 Concentration (ceramic packing)

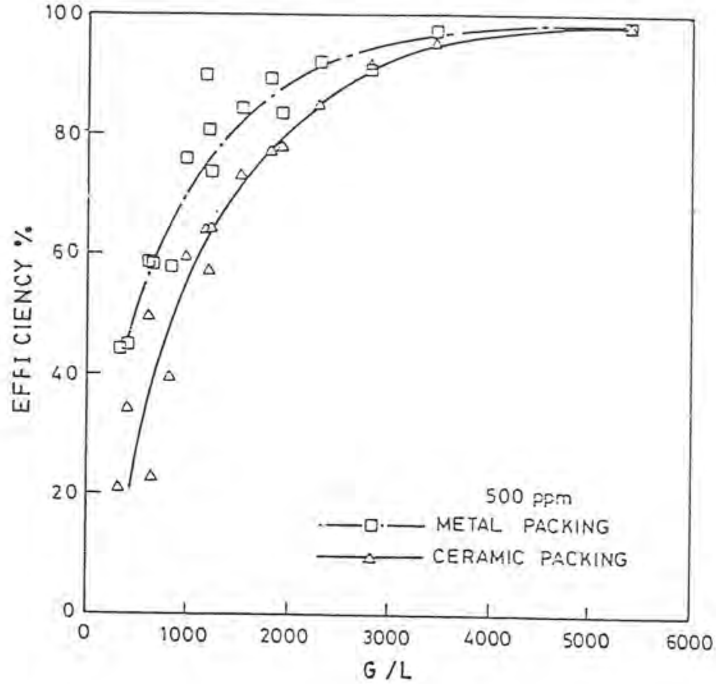

Fig.(4): Comparative Results of the Two Types of Investigated Packings at Various (G/L) ratios and 500 ppm

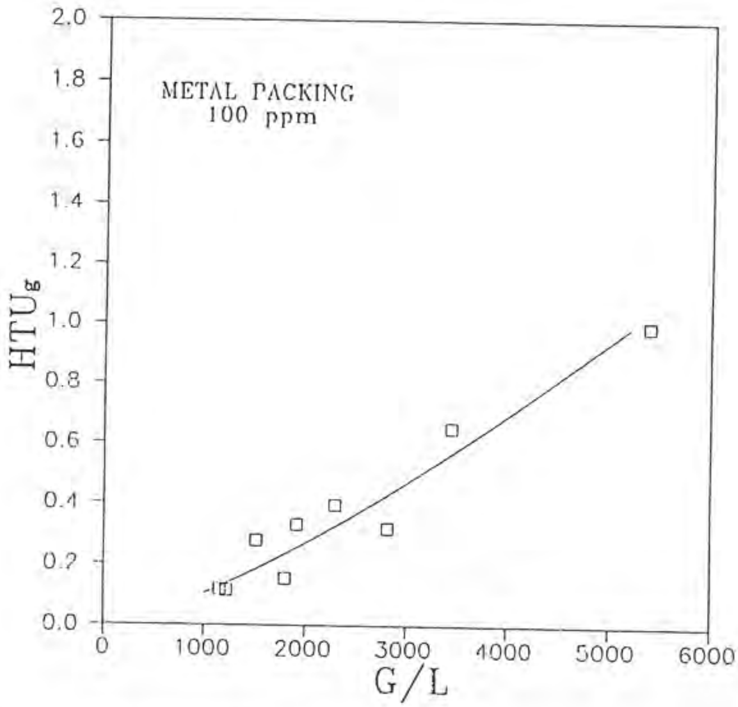

Fig.(5): Dependence of HTU_{og} on (G/L) Ratio for Metal Packing at 100 ppm Influent Concentration

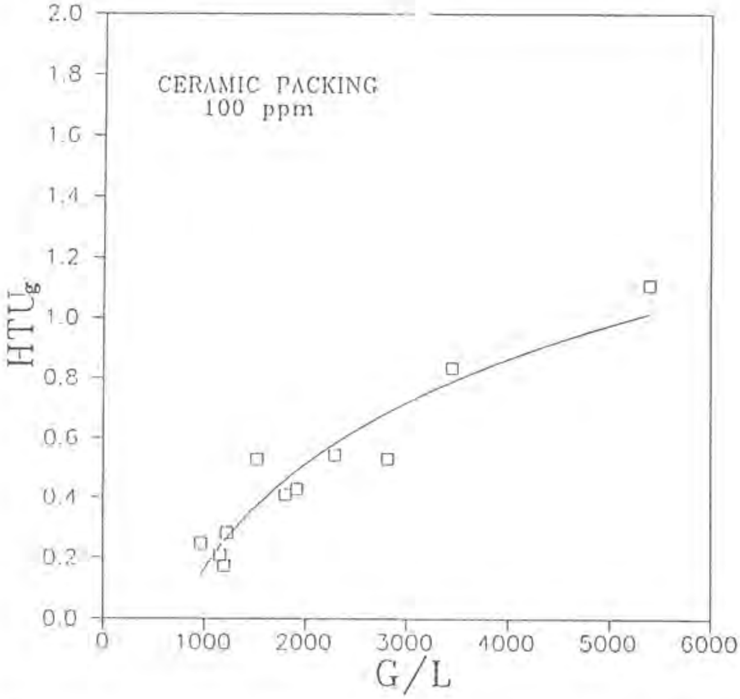

Fig.(6): Dependence of $(HTU)_g$ Ratio for Ceramic Packing at 100 ppm Concentration

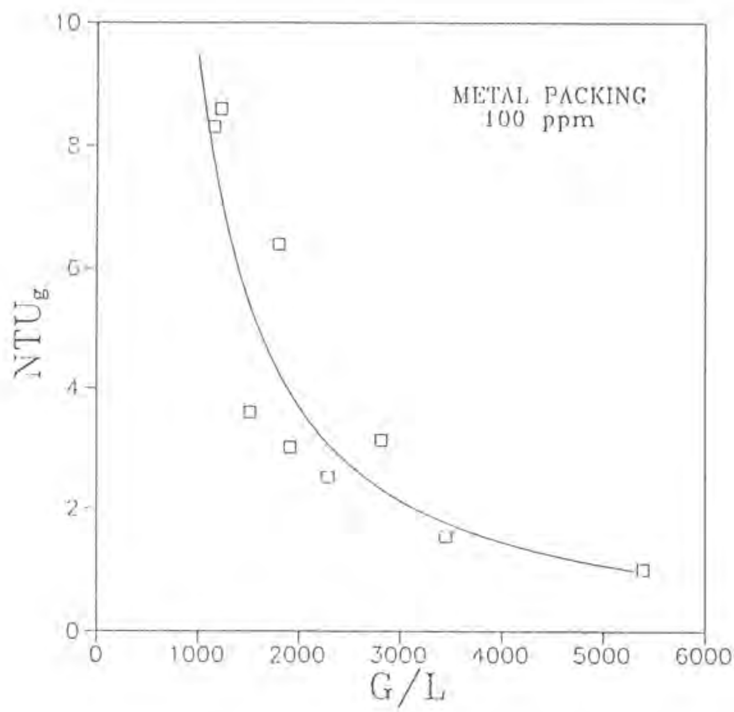

Fig.(7): Dependence of NTU_g on G/L Ratio for Metalic Packing at 100 ppm Influent Concentration

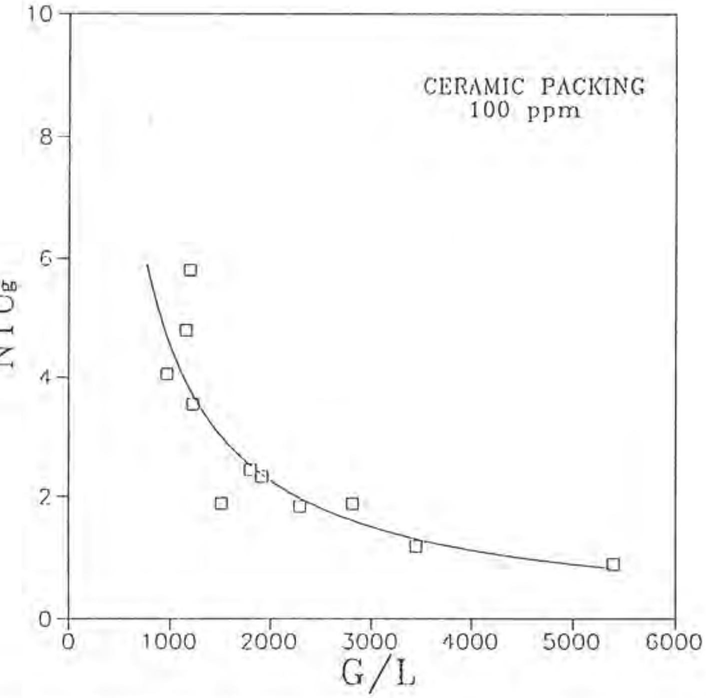

Fig.(8): Dependence of NTU_g on (G/L) Ratio for the Ceramic Packing at 100 ppm Influent Concentration

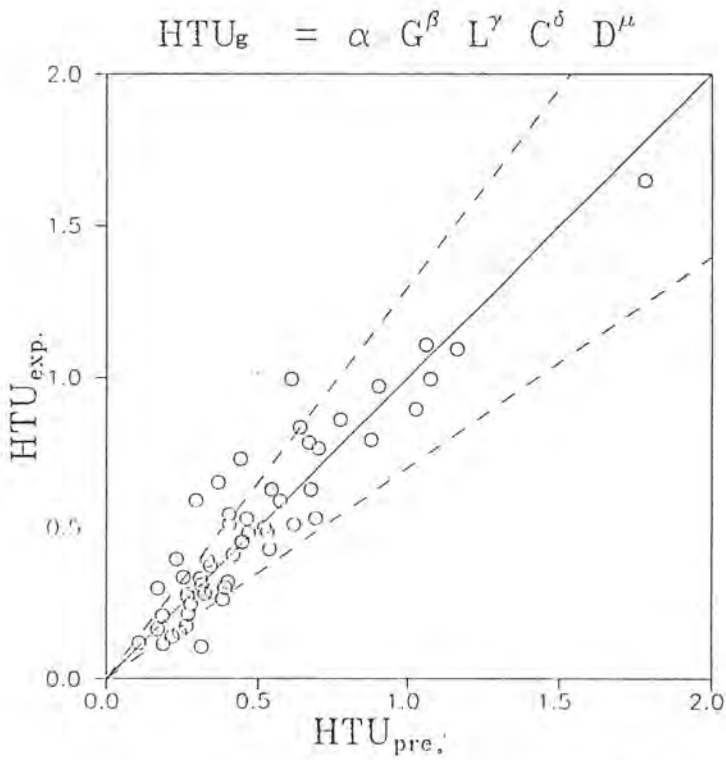

Fig.(9): Comparison of Experimental HTU and Predicted Ones According to the Recommended Design Correlation of the Present Wrok

Application of Ultraviolet Light In Disinfection Of Municipal Wastewater Effluents

Dr. Shaukat Farooq

APPLICATION OF ULTRAVIOLET LIGHT IN DISINFECTION OF MUNICIPAL WASTEWATER EFFLUENTS

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ABSTRACT

The use of ultraviolet radiation for the disinfection of wastewaters, relative to the established technologies of the chlorination and ozonation, is an emerging process application. It is the only physical process whereas all the disinfectants are chemical processes. Several studies have shown that use of UV radiation as "potentially" advantageous for the disinfection of relatively high quality treated wastewater. The use of UV radiation as an alternate disinfectant received the boost from the fact that chlorination result in formation of the potentially hazardous halogenated organics in drinking water. A comparative study was undertaken to investigate the effects of UV light alone, ozone plus UV light, and ozone alone on the disinfection of *Mycobacterium fortuitum* so as to determine whether the inactivation by ozone is augmented in the presence of UV light. The results of the experiments have shown that comparable inactivation occurred in all three system. However, UV light slightly improved the degree of inactivation when it was used with ozone, as compared to ozone alone. Furthermore, it was shown that UV light alone was a strong disinfecting agent.

Keywords: UV light, disinfection, ozone, wastewater

INTRODUCTION

Description of the UV Process

Disinfection by ultraviolet radiation is a physical process relying on the transferance of electromagnetic energy from a source (lamp) to an organism's cellular material (specifically, the cell's genetic material). The lethal effects of this energy result primarily from the cell's inability to replicate. The effectiveness of the radiation is a direct function of the quantity of energy, or dose, which was absorbed by the organism. This dose is described by the product of the rate at which the energy is delivered, or intensity, and the time to which the organism is exposed to this intensity (1,2,3).

The ideal UV disinfection model follows first order kinetics, whereby:

$$N = N_o e^{-kIt}$$

where:

N	=	bacterial density remaining after exposure to UV
N_o	=	initial bacterial density
k	=	rate constant
I	=	intensity of UV radiation
t	=	time of exposure

The product It is the UV dose. Thus the response, noted by the log of the survival ratio, N/N_o, can be plotted against dose; the slope is the rate coefficient, k. This is shown in Figure 1(a). Deviation from this model is generally manifested by "shoulders," whereby minimal response is noted below a "threshold" dose; and by tailing effects, often attributed to occlusion (shadowing) of bacteria by particulate matter.

The primary artificial source of UV energy, at present, is the low pressure mercury arc lamp. It is almost universally accepted as the most efficient and effective source for disinfection systems application. The primary reason for its acceptance is that approximately 85 percent of its energy output is nearly monochromatic at the wavelength of 253.7 nanometers (nm), which is within the optimum wavelength range of 250 to 270 nm for germicidal effects. The lamps are long (standard lengths are typically 0.75 and 1.5 m (2.5 and 4.9 ft) arc lengths) thin tubes (typically 1.5 to 2 cm (0.6 to 0.8 in) in diameter). The radiation is generated by striking an electric arc through mercury vapor; discharge of the energy generated by excitation of the mercury results in the emission of the UV light. These lamps can be suspended outside the liquid to be treated or submerged in the liquid; the intent is to get the energy into the liquid as efficiently as possible. Typically, if the lamp is to be submerged into the liquid, it is inserted into a quartz sleeve to minimize the cooling effects of the water. Figure 1(b) is presented to schematically represent the principal concerns when considering UV disinfection. In this example, the lamp is placed in the liquid, with the lamp perpendicular to the direction of flow. Other configurations may have the lamp parallel to flow, or the lamp may be suspended above the flowing liquid. Referring to Figure 1(c), as the lamp emits radiation, the intensity will attenuate as the distance from the lamp increases; this is due simply to the dissipation or dilution of the energy as the volume it occupies increases. A second attenuation mechanism involves the actual absorption of the energy by chemical constituents contained in the wastewater. This, analogous to the chlorine demand, is the "UV demand" of the wastewater.

The UV demand of a wastewater is quantified by a spectrophotometric measurement at the key wavelength of 253.7 nm; this expresses the absorption (or transmittance) of energy per unit depth. The outputs is absorbance units/cm, or a.u./cm. The percent transmittance can be determined from this unit by the expression:

$$\% \text{ Transmittance} = 100 \times 10^{-(\text{a.u./cm})}$$

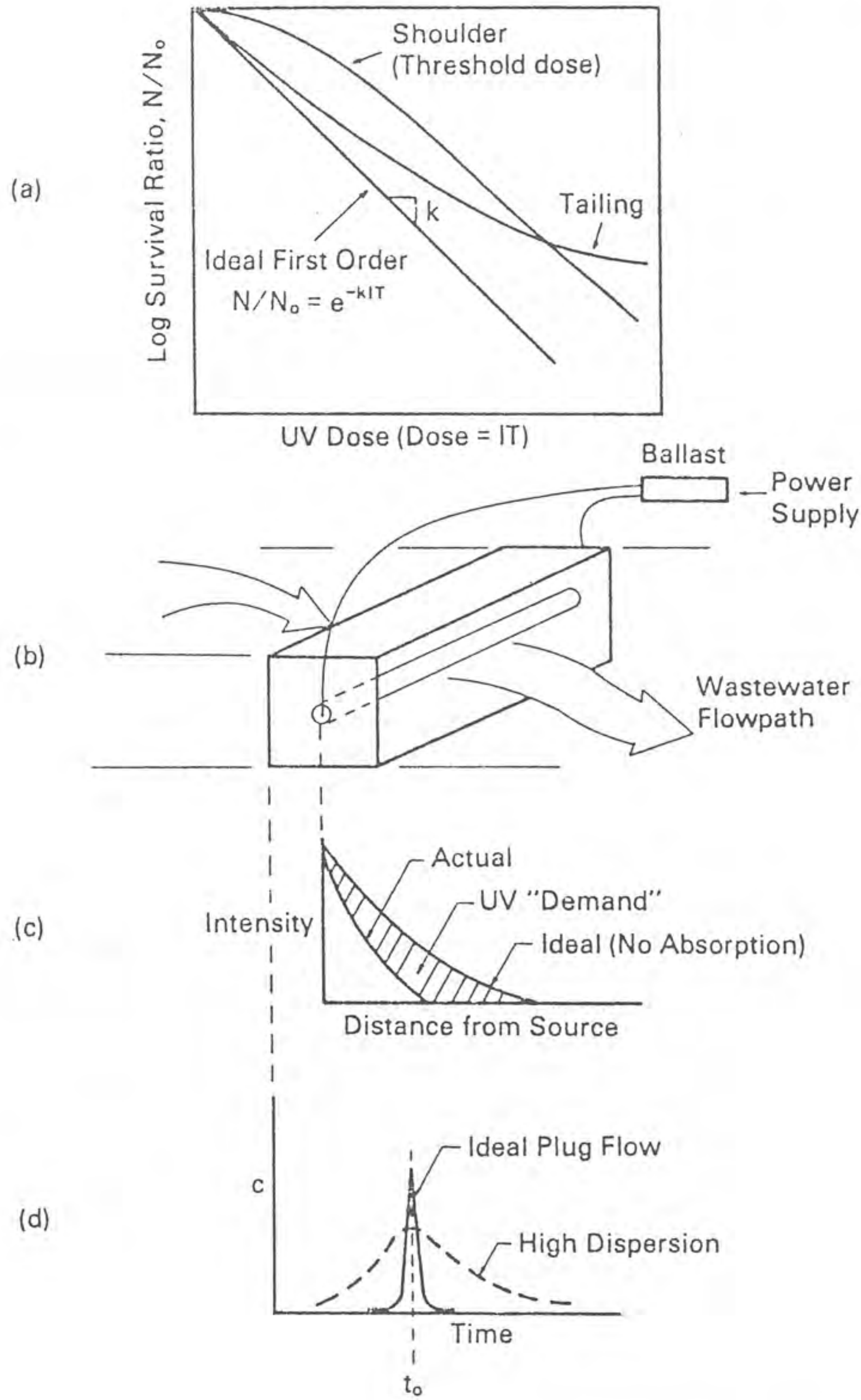

Figure 1. General description of UV design.

The term most often used for design purposes is the UV absorbance coefficient, α, expressed in base e:

$$\text{UV absorbance coefficient, } \alpha = 2.3 \text{ (a.u./cm)}$$

The unit for α is cm^{-1}

Although wastewater characteristics are different site to site, ranges of the UV demand can be described for different levels of treatment:

	UV Absorbance Coefficient $\alpha(cm^{-1})$	Percent Transmittance	Absorbance (a.u./cm)
Primary Treatment	0.4 to 0.8	67 to 45	0.174 to 0.35
Secondary Treatment	0.3 to 0.5	74 to 60	0.13 to 0.22
Tertiary Treatment	0.2 to 0.4	82 to 67	0.087 to 0.174

A second major is the provision of adequate exposure time to the microorganisms in order to meet the dose requirement at a given intensity. The key is to have plug flow through the system (see Figure 1(d)) such that each flow element resides in the reactor for the same amount of time. Perfect plug flow is not going to be achieved, of course; some dispersion will exist, such that there will be a distribution of exposure times about the ideal, theoretical exposure time. A design objective will be to minimize this distribution.

Evaluation of UV Disinfection

A major research and demonstration project was completed under joint sponsorship by the USEPA and the New York City Department of Environmental Protection [4]. Conducted at the Port Richmond Water Pollution Control Plant, Staten Island, New York, it investigated the performance of three large scale UV systems in the disinfection of secondary effluent and high rate settled raw wastewater. The systems included two 100-lamp quartz systems which differed only in the spacing between quartz surfaces—1.25 cm and 5.0 cm (0.5 in and 2.0 in). The third unit used Teflon tubes to carry the wastewater, with the lamps suspended outside the tubes. The effluent characteristics were highly variable; the suspended solids ranged between 5 and 50 mg/l, with a UV absorbance coefficient (base e) between 0.25 cm^{-1} and 0.5 cm^{-1}. Primary effluent was also treated to determine the application of UV to combined sewer overflow wastewaters. These were characterized by high suspended solids levels and UV absorbance coefficients between 0.5 cm^{-1} and 1.0 cm^{-1}.

The UV process was found to be very effective in the disinfection of secondary effluent. Log survival ratios between −3 and −4 could be achieved under practical loading conditions. Similarly, it was shown that a log survival ratio up to −3 could be accomplished with primary effluent.

There are significant number of plants now installed throughout the United States for the disinfection of treated municipal wastewater. A list of UV installations which are operational is presented in Table 1.

Table 1: Summary List of Facilities in the U.S.A. or Canada Utilizing Ultraviolet Light (UV) Which are in Operation

Facility Name	Size, mgd		Other Treatment Processes	Equipment	Comment on Performance
	Design	Current			
Albert Lea (Minnesota)	12.53	3.91	2-Stage activated sludge, tertiary filters	Pure Water Systems, Inc.	Good performance. Meeting coliform kill requirements. Coliform count is less than 1 MPN/100 ml. Some have been corrected. Operating since June 1983.
Ozark (MO)	0.72	0.20	Oxidation ditch	U.V.Purification Systems, Inc.	In operation 1 year. Initial problem with ultrasonic cleaning system achieving desired bacterial kill.
Tillonsburg (Ontario, Canada)	2.4	1.3	Extended Aeration	Trojan Industries	In operation for 2 years. Achieving 1/2 100 MPN coliform/100 ml.
Eden (Wisconsin)	0.16	0.10	1° clarifier, roughing filter extended activated sludge (for nitrification)	UV Technology	Good Performance
Red Top Meadows (Ketchum) (Idaho)	0.180 (dry 0.06)	0.180	Extended aeration oxidation ditch	UV Technologies (Now ENERCO)	Good performance. Fecal coliform count = 0 for 100% fecal coliform kill. Bulbs replaced annually. Operating since 10/82.

Advantages of UV Disinfection

Other than the simplicity of the process, UV also offers the advantages of system flexibility and a capability of responding quickly to changes in demand. There is relatively little complexity to the hardware, and maintenance generally requires low skill levels. The hazards of the process are low, principally related to the high electrical loads and the personal exposure to the UV radiation; these are conditions which are easily safeguarded. A major advantage of the process is the absence of a residual in the wastewater and any subsequent impact on the receiving water. A corollary to this is the ability to 'overdose' with UV and still not affect the receiving water. This allows for a less rigorous control requirement than associated with the use of chlorine. The absence of a residual can also be viewed as disadvantage when considering the operational control of the process. There is no immediate monitor of performance analogous to the chlorine residual. Since the energy levels are not high enough to affect chemical reactions, there are no significant intermediates formed by the process, even at overdose levels. This is clearly an advantage over the chemical addition processes.

MATERIALS AND METHODS

Cell culture

Mycobacterium fortuitum and acid-fast bacteria, was used as the primary organism in this study. A pure culture of *M. fortuitum* was obtained from the study of Engelbrecht *et al.* (1974), who proposed the acid-fast bacteria as a possible new microbial indicator of wastewater disinfection efficiency. In this study, *M. fortuitum* was used as the primary organism due to its greater resistance to ozone and ease of obtaining kinetic data [5].

Medium and cultivation method

In preparation of inactivation experiments, *M. fortuitum* was grown in a broth medium consisting of Middlebrook & Chon 7H9 mineral base, 4.7 g; malachite green, 1.0 mg; and sodium propionate, 1.0 gl^{-1} deionized water. The culture was incubated for 72 h at 37 C in a water shaker bath before harvesting. Cells were harvested by means of a Sorval GLC-2 (DuPont Instruments, Newton, CT) general laboratory centrifuge at 1800 revolutions per for 15 min. They were then washed twice with a total of at least 150 ml of 0.0025 M phosphate buffer. Cells prepared in this manner were resuspended in phosphate buffer and were kept at 4 C until needed.

M. fortuitum was enumerated using the 7H9 Middlebrook & Cohn medium indicated above but with 1.5° Bacto agar (Difco Lab, Detroit, MI) to solidify the medium. A spread plate technique was employed in this study to enumerate the surviving organisms. Samples were diluted with phosphate buffer solution such that a 0.1-ml aliquot of diluted sample would give at least 50 and no more than 200 colonies per culture dish. Incubation was accomplished at 37 C for 72 to 96 hours.

Analytical technique

The UV spectrophotometric method of Schechter [6] was employed to monitor the aqueous concentrations of ozone, whereas the concentration of ozone in the gas phase was determined as described in *Standard Methods* [7].

EXPERIMENTAL TECHNIQUE AND RESULTS

The survival kinetics of *M. fortuitum* at various temperatures was determined in a continuous flow type reactor consisting of a 55 mm dia. and 270 mm long pyrex glass column, having a volume of 500 ml (Fig. 2). A gaseous mixture of ozone and air was supplied at the bottom of the reactor through a medium porosity, horizontal fritted glass diffuser. Feed water, consisting of deionized-buffered water (clean system) and inoculated

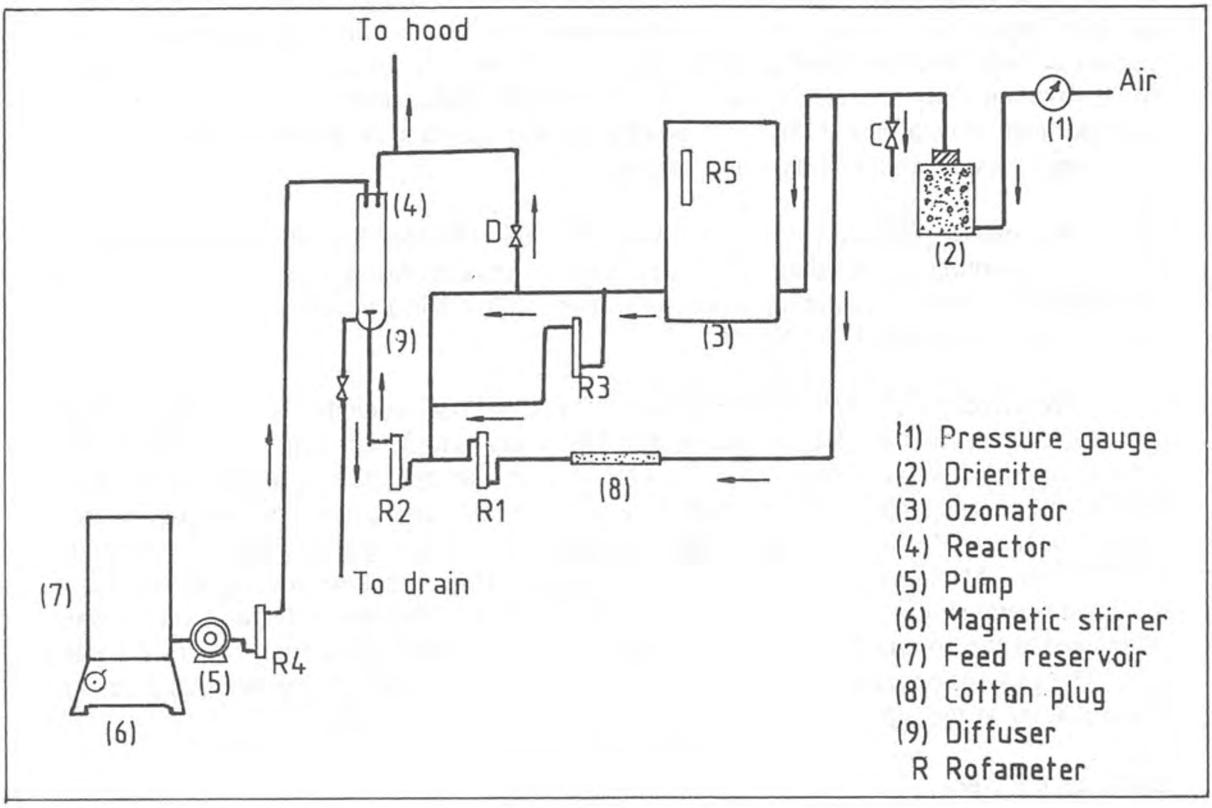

Figure: 2. Flow diagram of the continuous flow reactor.

with test organisms, was fed to the reactor at the top and was removed from the bottom under the force of gravity. A methylene blue tracer study showed that the reactor provided complete mixing. Further, during all experiments, steady state was attained with respect to achieving a constant ozone residual at each detention time prior to collecting samples for enumeration of organisms. An inactivation study was also performed in the reactor to determine the reproducibility of the experimental data in that the same experiment was repeated four times under identical conditions; good reproducibility was observed. Further, during all experiments. steady state was attained with respect to achieving a constant ozone residual at each detention time prior to collecting samples for enumeration of organisms. An inactivation study was also performed in the reactor to determine the reproducibility of the experimental data in that the same experiment was repeated four times under identical conditions; good reproducibility was observed.

The ozone generator, Welsback Model T-408 (Philadelphia, PA) was fed either oxygen or breathing air (dried to $-70°F$ dew point by passing through Drierite, i.e. $CaCl_2$). The generator was operated at a pressure of 8 psig (55.2 kN/m^2) and the feed pressure was maintained at 15 psig (103.5 kN/m^2).

The effect of UV light on disinfection with ozone was studied in a fermentor (New Brunswick Scientific Co., NJ) consisting of a 15 cm dia. and 30 cm long Plexiglas reactor and having an effective volume of 4 ℓ. This reactor was operated as a continuous flow system and was equipped with a 15 W low pressure mercury germicidal lamp (General Electric Schenectady, NY), with the major UV light energy being emitted at a wavelength of 253.7 nm. Mixing was provided by three stainless steel impellers equally spaced on a shaft and operated at a speed of 380 revolutions per min. The ozone air gas mixture was introduced at the bottom of the reactor through a stainless steel sparger at a rate of 1 ℓ per min. The gaseous ozone concentration was 16.8 mg/ℓ. Inoculated feed water was pumped to the reactor at the top and was removed from the bottom by another pump.

Effect of UV light

A study was undertaken to investigate the effects of UV light alone, ozone plus UV light, and ozone alone on the disinfection of *M. fortuitum* so as to determine whether the inactivation by ozone is augmented in the presence of UV light.

The minimum detention time studied in these experiments was 72 sec. The results of these experiments are shown in Fig. 3. It may be noted that comparable inactivation of *M. fortuitum* occurred with the three different conditions, i.e. UV light, ozone plus UV light, and ozone. When UV light was employed alone, the ozone-air mixture was replaced by an air flow rate of 1 ℓ/min in order to maintain identical mixing conditions. From Fig. 3, it would appear that UV light slightly improves the degree of inactivation when it is used with ozone, as compared to ozone alone. However, the importance of ozone residual becomes less significant with respect to inactivation in the presence of UV light as it increases the degree of inactivation of *M. fortuitum* even though ozone residual is decreased. It can also be seen in Fig. 3 that UV light alone is a strong disinfecting agent; thus, the increased inactivation in the case of ozone plus UV light may be attributed to the application of UV light rather than to the catalytic effect of UV light with ozone.

Experiments were also performed using an activated sludge effluent obtained from the local treatment plant. The effluent was brought to the laboratory, equilibrated at 24°C,

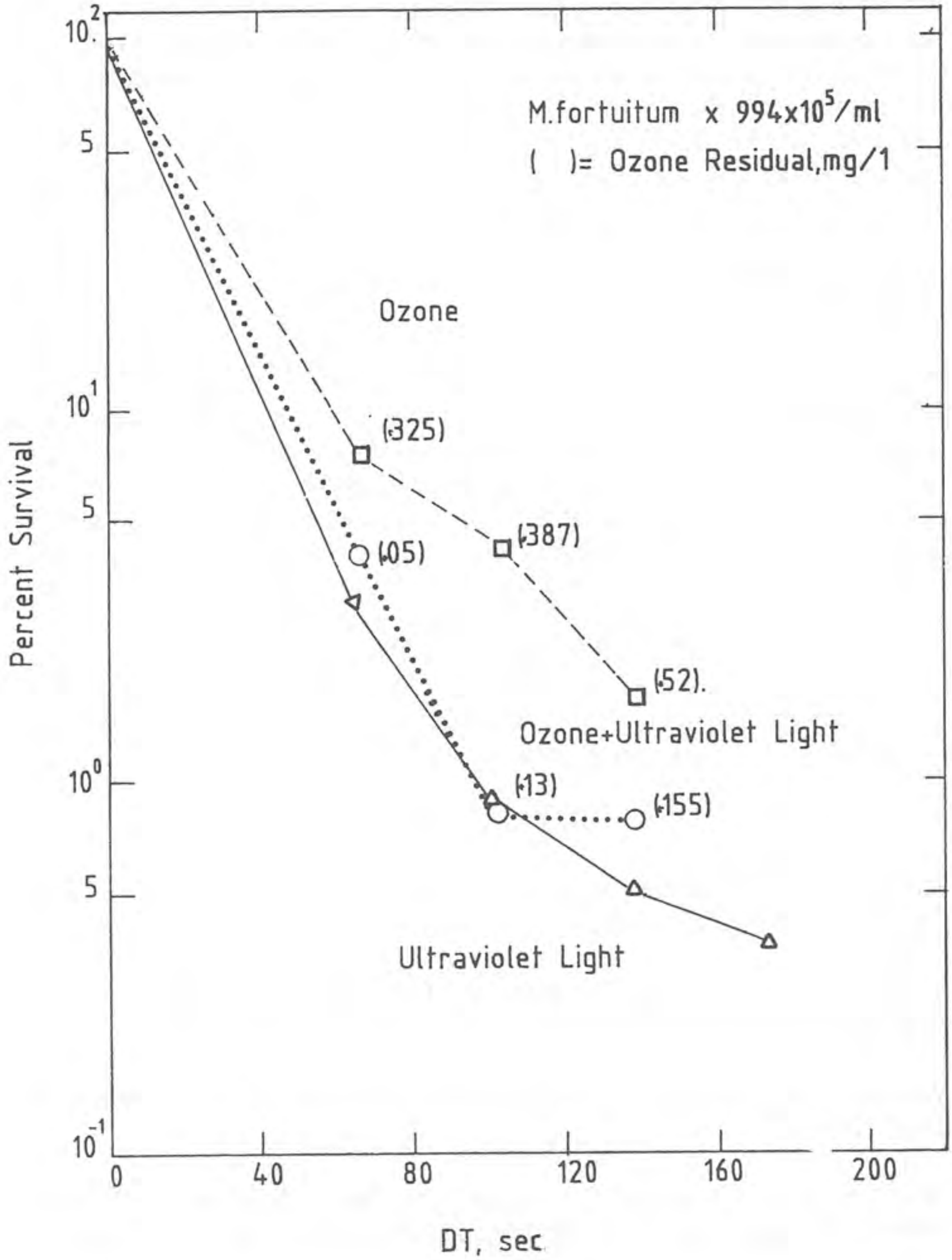

Figure: 3. Effect of u.v. light on the survival of M. Fortuitum at a constant rate of applied ozone, inoculated with *M. fortuitum*, and tested at a detention time of 72 sec. Two different conditions were evaluated, i.e. ozone plus UV light, and ozone alone. The percentage survival of *M. fortuitum* observed was 6.2 and 6.7 with ozone plus UV light and ozone alone, respectively. When UV light was employed in combination with ozone, the ozone residual decreased to 0.3 mg/ℓ as compared to 0.78 mg/ℓ in the case of ozone alone, when the partial pressure of ozone in the gas phase was kept constant in both cases. In these experiments, oxygen was used to generate the ozone as a higher concentration of ozone was required in order to meet the high ozone demand of the wastewater. However, although ozone was applied at a flow rate of 1 ℓ/min, the concentration of gaseous ozone was increased to 26 mg/ℓ due to the use of oxygen instead of air.

Hewes *et al.* [8] combined ozone with UV radiation to determine the rates of oxidation of difficult to oxidize, 'refractory', chemical species which were found ubiquitously in secondary effluent and were not removed by conventional secondary treatment plus carbon adsorption. Five 'model compounds', i.e. glycine, ethanol, acetic acid, glycerol and palmitic acid, were chosen for their investigation. Their results indicated that the organic compounds were not oxidized by ozone when the reaction was 'activated' by UV radiation. However, their data cannot be compared with the findings of this study, because chemical oxidation of these ozone refractory compounds generally takes hours as compared to seconds for the inactivation of *M. fortuitum* with ozone in both a clean system and secondary wastewater effluent.

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Treatment of Phenolic Wastewater Using Modified Rotating Biological Contractor

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TREATMENT OF PHENOLIC WASTE WATER USING MODIFIED ROTATING BIOLOGICAL CONTACTOR

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ABSTRACT

A modified four stage Rotating Biological Contactor (RBC) was used for the treatability studies of synthetic phenolic waste water. The operating parameters used in this study were hydraulic loadings, organic loadings and rotational speeds. Organic loads were used in the range of 3.04 to 78.6 g phenol.m^{-2}.day^{-1}. The RBC was operated at speeds of 12, 18 and 24 rpm. In most cases the percentage of Phenol removed was more than 90% and in some cases as high as 99%. Biokinetic coefficients based on Kornegay and Hudson models and a mathematical model were obtained using linear regression. Higher organic loading rates were possible in this unit than the conventional types of RBC of similar diameter for a given percentage removal of COD. The unit was also tested for shock loads to verify its regaining capacity and effect of reversal flow on performance of RBC.

KEYWORDS

Rotating biological contactor; phenol removal; organic loading; hydraulic loading; shock load.

1. INTRODUCTION

Pollution of water and environment due to industrial and other wastes is one of the major problems facing the developed as well as developing countries. One of the industrial wastes of serious consequence from the point of water pollution is the phenolic waste water from coal carbonization plants, refineries and petrochemical industries.

In the Rotating Biological Contactor (RBC), the biological film responsible for removal of pollutants is formed on the surface of disks. The RBC was first conceived in Germany by Weigand in 1900. His patent for the contactor describes a cylinder consisting of wooden slats. Pescod and Nair [1] in 1972 showed that RBC is a versatile aerobic process which will be efficient in tropical countries.

RBC process allows for a longer and more intense contact time, a continuous sloughing of biomass and high degree of treatment compared to trickling filters. RBC process has been found to have many advantages like easy adaptability for small to medium size installations, simplicity of construction, operation and maintenance, relatively low reactor volume due to large microbial population on the disc, ease of sludge seperation and lower power and maintenance requirement.

While considerable work has been done in India in the use of RBC process for treatment of domestic waste water (Khan and Siddiqui in 1972) [2] not much work was done on the treatment of different industrial waste waters using RBC.

In this communication the studies were carried out on the effect of hydraulic loading rates (HLR), organic loadings, rotational speeds, hydraulic and organic shock loads and effect of periodic reversal flow on organic removal. Biokinetic coefficients based on Kornegay [3] and Hudson [4] models are estimated and also a mathematical model based on experimental results using regression analysis is presented.

2. MATERIAL AND METHOD

The RBC in this study is a laboratory model and the discs were modified by attaching porous nechlon sheets to enhance biofilm area. That sheet has a light weight and occupies less space. The RBC trough is made of perspex glass and is divided into four equal volume chambers by staging baffles which have openings to hydraulically inter-connect the consecutive chambers.

33 discs of inert light weight material provides 4.68 m^2 of total surface area, constituting a specific area of 155.5 $m^{-2}.m^{-3}$ for microbial growth. They are rigidly mounted on stainless steel shaft and were submerged in the trough to about 40% of the disc diameter. The discs were rotated by a 0.5 HP gear reduced electric motor. The dimensions of RBC used in this study are tabulated in Table 1 and schematic diagram is shown in Fig.1.

TABLE 1 Dimensions of laboratory RBC

Number of discs	33	Spacing between discs, cm	2
Number of stages	4	% submergence of the disc	40
Diameter of discs, cm	30	Surface area per stage, m^2	1.17

2.1 Start up Operations

A synthetic phenolic waste was produced by thoroughly mixing phenol in tap water with the required nutrients for biomass growth. Acclimatization was carried out to develop biomass on the discs. In the beginning, the RBC was fed in batch mode using waste water from I.I.T oxidation pond. The synthetic waste was added in small quantities every day for acclimatization over a period of one month. Then, the experiments were conducted in continuous mode operations. The experimental parameters measured were Phenol concentration, COD, BOD and pH. Analysis was carried out by the method given in standard methods [5].

3. RESULTS AND DISCUSSION

3.1 Effect of Influent Substrate Concentrations

The organic substrate concentrations employed in this study were: 183, 313, 497, 700 and 1046 mg/l of phenol concentration with a rotational speed of 18. Hydraulic loads used were: 0.02, 0.04, 0.07 and 0.11

$m^3.m^{-2}.day^{-1}$. Fig.2 shows the results of organic removal experiments under different hydraulic loadings and with varying influent phenol loadings. For an influent phenol load of 170 mg/l, the percentage of removal at a flow rate of 0.02 $m^3.m^{-2}.day^{-1}$ was 100. As the flow rate was increased from 0.02 to 0.11 $m^3.m^{-2}.day^{-1}$, the phenol removal was found to decrease from 100% to 97.5%. Also as organic loadings increased from 3 g $phenol.m^{-2}.day^{-1}$ to 78.6 g $phenol.m^{-2}.day^{-1}$, the percentage of phenol removal decreased from 100 to 50.7, and the crossponding BOD and COD results were recorded.

Fig.3 shows the phenol removal with varying organic loadings. A correlation plot between the phenol removed and phenol applied was presented as shown in Fig.4. A linear relationship exists between the phenol applied and the phenol removed over a range of 3 to 78.6 g $phenol.m^{-2}.day^{-1}$ .It is inferred that the process possessed a first order behavior. At a load more than about 25 g $phenol.m^{-2}.day^{-1}$, the trend approached zero order behavior. Above this point an increase of load will not results in additional removal.

The phenol removal in different stages of RBC is shown in Fig.5. The influent phenol loads were 200, 363, 500, 713 and 1156 mg/l. At a hydraulic flow rate of 0.07 $m^3.m^{-2}.day^{-1}$, the effluent COD values were 0.0, 0.0, 55.2, 230 and 477.5 mg/l respectively. The crossponding BOD and COD results were estimated and recorded. It was observed that most of the reduction occurred in the first two stages. This may be explained by the fact that at higher organic loadings the mass transfer and availability of oxygen may be the limiting factors while at lower organic loadings the role of these is insignificant. It was also observed that biomass production and attachment was maximum in the first two stages and was relatively low in the third and fourth stages.

In order to maintain uniform growth of biomass at all stages, flow reversal was successfully tried by introducing the influent at the fourth stage and withdrawing the effluent from the first stage periodically.

3.2 Effect of Rotational Speed

The effect of rotational speed was studied at 12, 18 and 24 rpm. Figure 6 shows the effect of rpm on organic removal. Increased rotational speed has a pronounced effect in waste water treatment in several ways. It provides faster contact between the biological mass and waste water, aerates the waste water and provides the necessary mixing velocity in each stage.

However, better performance was found at 18 rpm compared to 12 and 24 rpm. This indicates that an increase of RPM beyond the optimum value will enhance sloughing off the biomass from the disc and gets carried away together with the effluent and hence the decrease in efficiency. Also at lower organic loading it was not necessary to increase RPM because higher organic removal was possible at lower RPM itself.

3.3 Effect of Flow Reversal

RBC is operated as a series of individual stages through which waste water flows in one direction. The present experimental results reveal that most of the reduction of organic matter always occur in the first two stages, i.e biomass growth was more on the first two stages and diminishes with each successive stage. After reaching steady state operation, flow reversal was tried by introducing the influent at the fourth stage and withdrawing the effluent from the first stage periodically.

Tropy [6] has suggested that the RBC biofilm grows more rapidly in response to an incremental organic concentration. Based on this concept, a flow reversal should be able to provide more substrate to the microorganisms in the last stages and thus enhance its growth. Process performance was evaluated in terms of phenol and COD. Figure 7 shows the results of the flow reversal experiments. From these results, it can be observed that flow for one day reversal is more efficient than two days reversal and normal flow. From the results it is clear that this method can be established to enhance the efficiency of RBC process without any extra cost. The advantages of the flow reversal are a higher utilization of the biomass carrier surface and as a consequence thereof a smaller reactor volume in order to reach a required effluent standard and a higher capacity to handle diurnal organic loading fluctuations because of an even distribution of the biofilm thickness produced by this effect in all the stages (as shown in Table 2).

TABLE 2 Approximate biofilm thickness (for 30 days) with and without flow reversal (in mm).

Stage No.	Biofilm thickness with periodic flow Reversal	Biofilm thickness without flow reversal
1	2.0-2.5	2.5-3.0
2	2.0-2.5	2.0-2.5
3	2.0-2.5	1.5-2.0
4	1.5-2.0	1.0-1.5

3.4 Process Stability

Hydraulic as well as organic shock load tests have been conducted for the present study. Fig.8 shows the performance of RBC under hydraulic shock load. The two shock times applied were of 1.5 and 3.0 hrs. At an influent phenol concentration of 745 mg/l an organic removal of 100% was observed at a HLR of 0.02 $m^3.m^{-2}.day^{-1}$. At this time the HLR was suddenly increased by a factor of five and after 1.5 hrs shock time, the organic removal was observed to be 82.7%. The system regained its original efficiency level within 2 hrs after the removal of shock load. Similarly, after 3.0 hrs of shock time the system regained its original efficiency level within 5 hrs after removal of shock load.

Performance of organic shock load is shown in Fig.9. The influent substrate was suddenly increased from 206 to 807 mg/l of phenol at a constant HLR of 0.10 $m^3.m^{-2}.day^{-1}$ for 1.5 hrs shock time, and the effluent phenol changed from 0.0 to 210.2 mg/l. After this period the system regained its original capacity within 5 hrs after the removal of shock load. Similarly, after 3.0 hrs of shock load the effluent of COD changed from 150 to 1250 mg/l. The system returned to its original capacity within 6 hrs after removal shock load.

3.5 Mathematical Model

Many mathematical models have been presented in the literature to describe the biokinetics of the RBC system. Based on Monod's kinetics, Kornegay in 1975 [3] and Hudson in 1976 [4] have developed mathematical models for RBC substrate removal.

3.5.1 Kornegay Model

The first attempt to formulate a relevant kinetic model for RBC process was made by Kornegay and Andrews who used their concept to develop a steady state model for the RBC system by the equation:

$$Q(S_o - S_1) = P\ A\ S_1/(K_s + S_1) + ((\mu_{max})_s / Y_s)\ X_s\ V\ S_1 / (K_s + S_1) \quad .....(1)$$

where $P = (\mu_{max})_A X_f d / Y_A$

and it assumes that under most operating conditions the suspended growth are washed out and the liquid volume has a negligible effect on RBC performance and the above equation becomes:

$$Q(S_o - S_1) = P\ A\ S_1/(K_s + S_1), \quad ....(2)$$

By rearranging the above equation we have,

$$A/Q(S_o - S_1) = K_s/(P\ S_1) + 1/P \quad ....(3)$$

The area capacity, P and saturation constant, K_s can be calculated by plotting a graph as shown in Fig. 10. The intercept gives 1/P and the slope of the graph gives K_s/P. The values of P and k_s obtained from the figure are shown in the Table 3.

3.5.2 Hudson's Model

Hudson et al. [4] have developed a mathematical model based on Monod's growth function and plug flow regime. The model can be stated as follows.

$$\theta/(S_o - S_1) = (K_s / \mu_v) \ln (S_o / S_1)/(S_o - S_1) + 1/\mu_v \quad ....(4)$$

Where $\mu_v = \mu_{max} X/Y$

The values of biokinetic coefficients K_s and μ_v can be estimated from the slope and intercept of the line as shown in Fig. 11. From figure the values of K_s and μ_v are shown in Table 3.

TABLE 3 Biokinetic constant values in different mode of expressions based on Kornegay and Hudson models.

Biokinetic constants	Mode of expressions		
	Phenol conc.	BOD	COD
Kornegay's Model			
K_s, mg/l	38	47	153
P, $mg.m^{-2}.day^{-1}$	32,032	99,816	87,478
Hudson's Model			
K_s, mg/l	95	727	788
μ_v, $mg.l^{-1}.hr^{-1}$	243	849	935

3.5.3 Model Based on Experimental Results

In the development of a mathematical model for the RBC system, the parameters considered are influent organic loading (L), flow rate (Q), and rotational speed (R), retention time (t), effective surface area (A), submerged disk depth (D) and liquid temperature (T). The percentage of substrate removal (F) can be expressed as:

$$F = A_o + A_1*L + A_2*Q + A_3*R + A_4*t + A_5*A + A_6*D + A_7 * T \quad ....(5)$$

Wu et. al [7] modified the equation by omitting the variables t, D, A, and T because they do not significantly contribute to the regression. In the present case, since A is constant and T is assumed constant, we can write:

$$F = A_o + A_1*L + A_2*Q + A_3*R \quad ....(6)$$

The partial regression coefficients in the equation is determined by conducting regression analysis of the experimental data and the following correlations were obtained:

1. F(Based on phenol) = 106.73 - 0.755*L + 0.018*Q - 0.129*R ....(7)
2. F(Based on COD) = 96.72 - 0.286*L + 0.014*Q - 0.334*R ....(8)
3. F(Based on BOD) = 94.35 - 0.169*L + 0.0042*Q .....(9)

3.6 Correlation between Phenol and both COD and BOD

Fig. 12 shows the interrelations between phenol concentrations and COD and the following equation was obtained:

Phenol conc.= 0.37*COD - 1.122 ....(10)

Similarly the interrelations between phenol concentrations and BOD gave the following equation:

Phenol conc. = 0.352*BOD + 43.766 ....(11)

4. CONCLUSION

1. The RBC system successfully treated phenolic waste water of phenol concentrations 183 to 1046 mg/l. The percentage removal achieved by the unit varied from 100 at 3 g phenol.m^{-2}.day^{-1} to 50 at 78.6 g phenol.m^{-2}.day^{-1}.

2. At lower HLR which gave the longer retention time the maximum efficiency was obtained. The efficiency of phenol removal was found to decrease with increase of HLR which gave shorter retention time.

3. The phenol removal was found to decrease with an increase in organic loading. The correlation between phenol applied and phenol removed reveals that the organic removal exhibits a first order behaviour up to an influent load of 25 g phenol.m^{-2}.day^{-1}, above which the trend changes gradually and approaches to zero order.

4. Increasing the rotational speed beyond the optimum value, treatment levels decreased. 18 rpm was found to be optimum rotational speed in the present study. At low organic loadings (up to 25 g phenol.m^{-2}.day^{-1}) it was not necessary to increase rotational speed since 100% removal of phenol was possible.

5. Most of COD removal occurred in first two stages due to lesser amount of biomass in the later stages since the availability of nutrients from the waste decreases along the stages.

6. The system was able to regain its full capacity within a short time after the shock loads.

7. The RBC performance was found to be improved by reversing the flow periodically. The enhancement of organic removal decreased with increase in reversal period.

8. The design biokinetic constants P and K_S defined in Kornegay's model based on mode expressions were estimated and shown in Table 3. Similarly, in the case of Hudson's model, the biokinetic constants μ_V and K_S were calculated and shown in Table 3.

9. Since the order of kinetic reaction is more than zero and the plug flow is expected to be more efficient than CSTR in series, Hudson's model can be taken as more efficient than Kornegay's model.

10. The interrelations between phenol and COD, and phenol and BOD are obtained.

NOMENCLATURE

A -Surface area of discs, m^2
Q -Hydraulic flow rate l/day
V -Volume of the reactor, l
HLR -Hydraulic loading rate, $m^3.m^{-2}.day^{-1}$
K_S -Saturation constant, mg/l
P -Area capacity, mg.m^{-2}.day^{-1}

S_o -Influent substrate concentration, mg/l

S_1 -Effluent substrate concentration, mg/l

X_f -Active biomass per unit volume of attached growth, mg/l

X_s -Active biomass per unit volume of suspended growth, mg/l

Y_A -Yield coefficient for attached growth, day^{-1}

Y_s -Yield coefficient for suspended growth, day^{-1}

d -Active depth of microbial film on any rotating disk, mm

θ -Mean hydraulic detention time, hr

$(\mu_{max})_A$ -Maximum specific growth rate attached growth, day^{-1}

$(\mu_{max})_s$ -Maximum specific growth rate suspended growth, day^{-1}

μ_{max} -Maximum specific growth rate, day^{-1}

μ_v -Specific growth rate, day^{-1}

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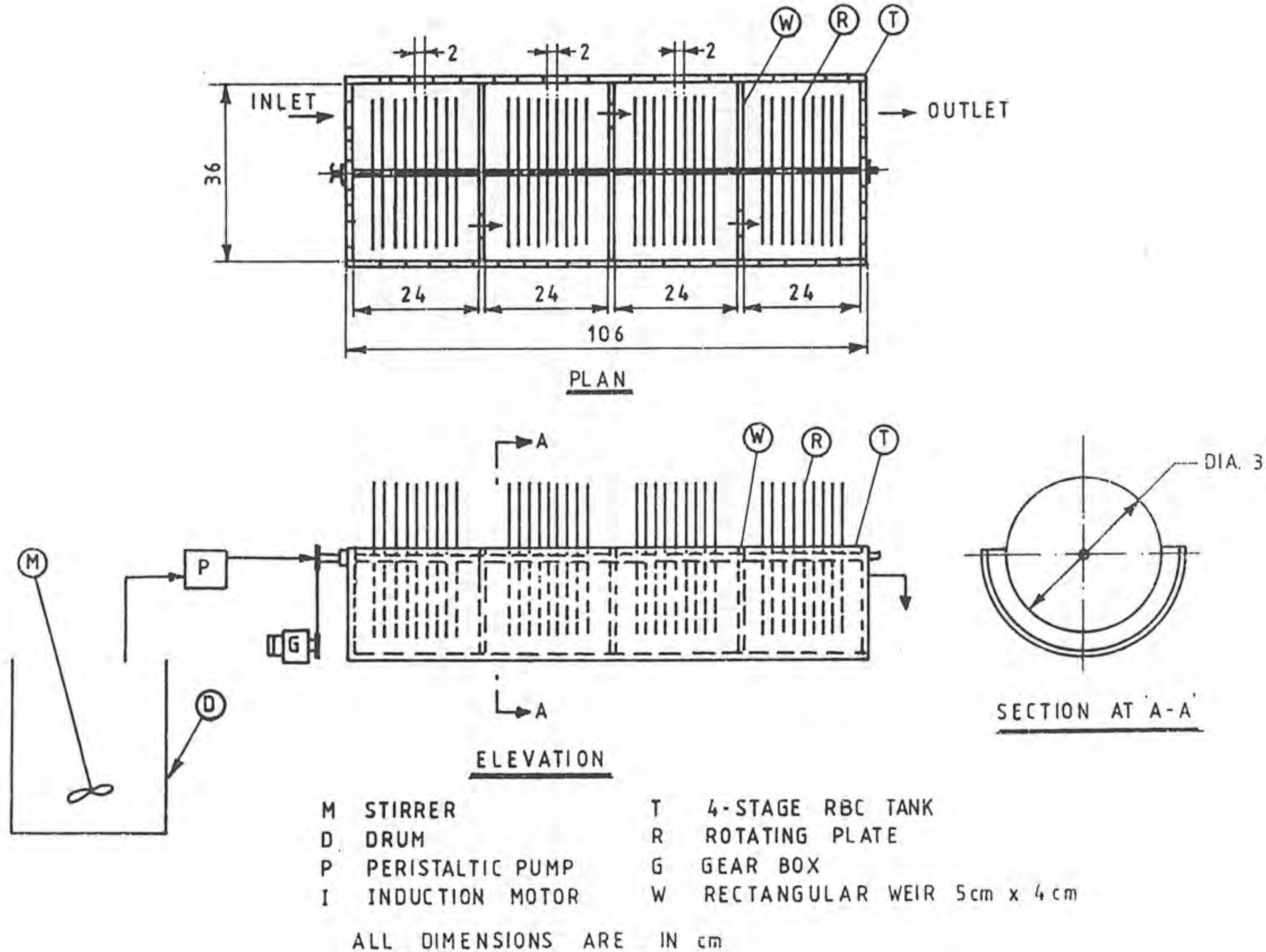

Fig.1. Schematic diagram of the Rotating Biological Contactor Unit.

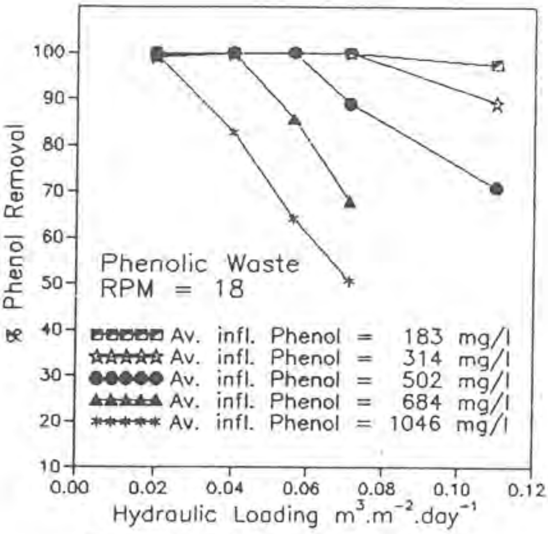

Fig.2 Effect of Hydraulic Loadings on Phenol Removal in RBC

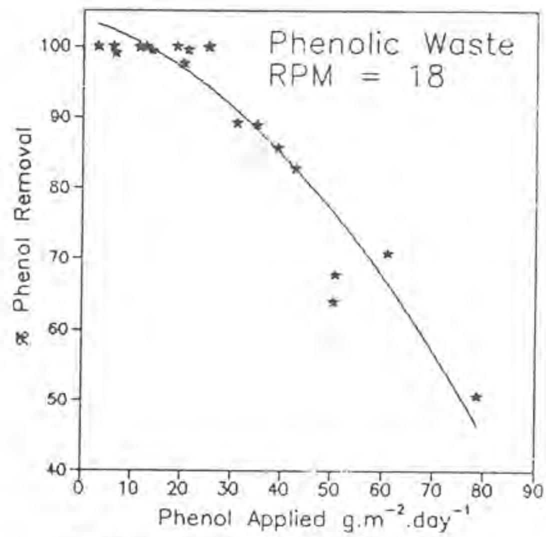

Fig.3 % Phenol Removal with Varying Organic Loadings

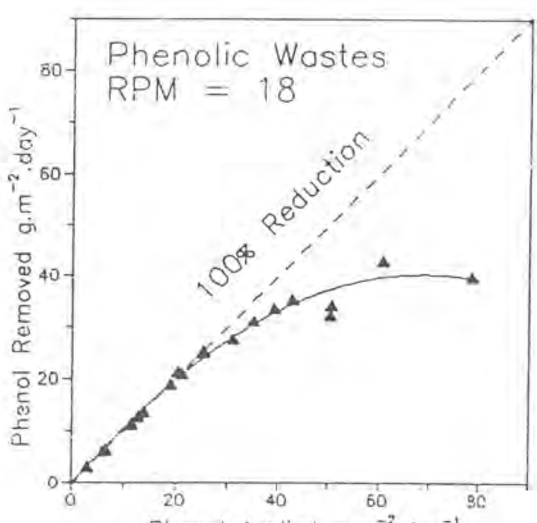

Fig.4 Correlation Plot for Phenol Removal

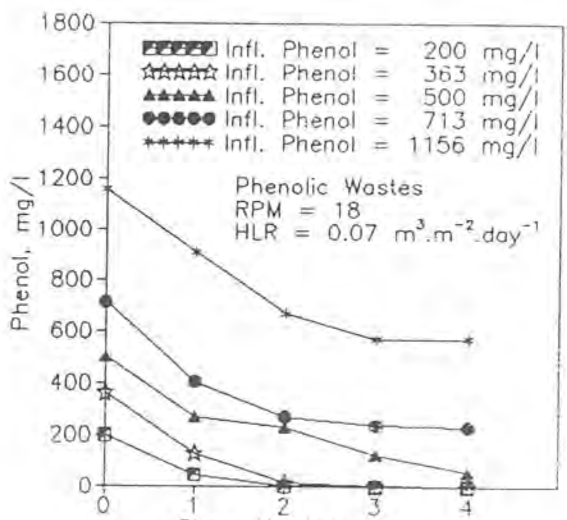

Fig.5 Phenol Removal in Four Stages of RBC

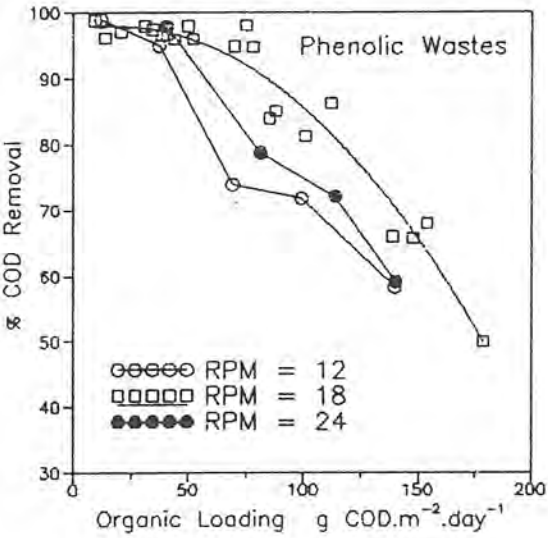

Fig.6 Effect of RPM on COD Removal in RBC

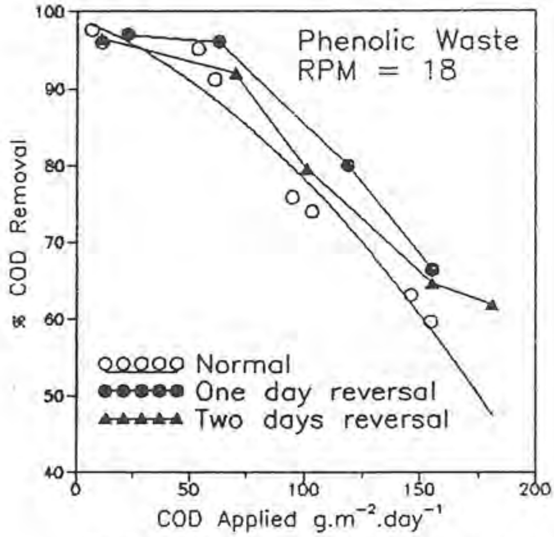

Fig.7 Effect of Flow Reversal on %COD Removal in RBC

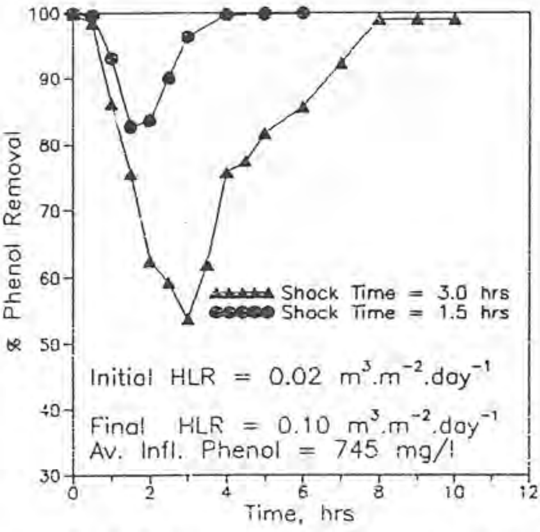

Fig.8 Performance of RBC under Hydraulic Shock Load (Based on Phenol)

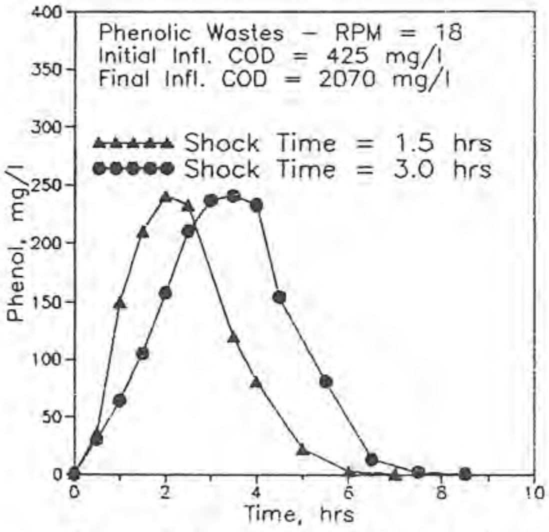

Fig.9 Performance of RBC under Organic Shock Load

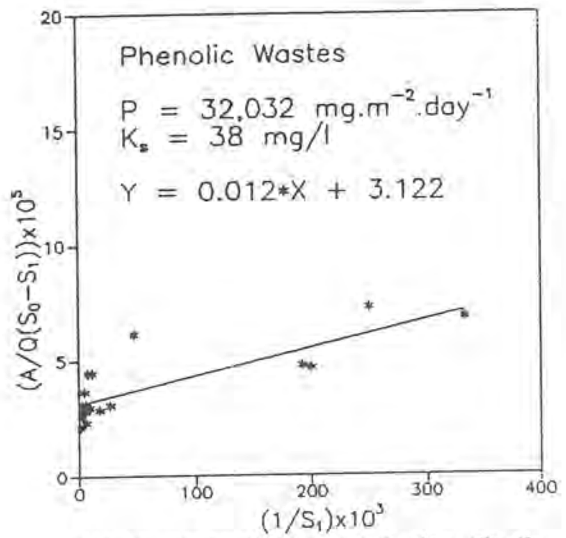

Fig.10 Determination of Design Kinetic Constants for phenol (Based on Kornegay's Model)

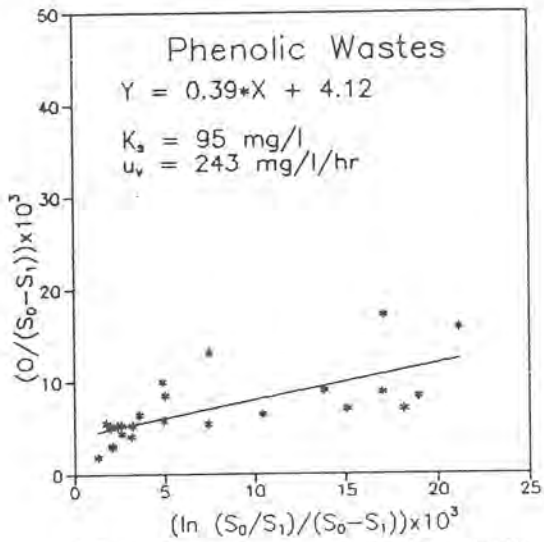

Fig.11 Determination of Design Kinetic Constants for Phenol (Based on Hudson's Model)

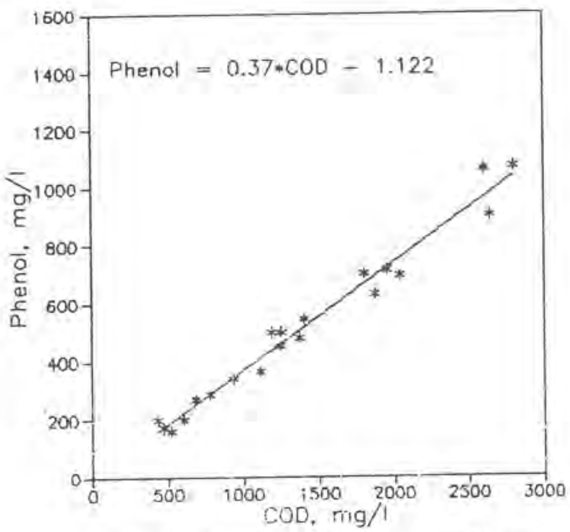

Fig.12 Correlation between Phenol and COD

Session - 10
Drinking Water

Recycling Filters Backwash Water in Drinking Water Treatment Plants

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Recycling Filters Backwash Water in Drinking Water Treatment Plants

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Abstract

Buraydah Drinking Water Treatment Plant (BDWTP) is one of the major water treatment plants in the Central Region of Saudi Arabia. It was designed to remove primarily iron and manganese from groundwater supplies and produce a maximum of 96000 m^3/d filtered and chlorinated water. A pilot study was conducted to assess the potential of using clarification without chemical addition to recycle portion of the filter backwash water which represents about three percent of this plant daily production. The results of the experimental work show that it is possible to reduce the iron content of the backwash water from 50 to 12.9 mg/L using a 4 hr detention time in a 3 m column height. This will also lead to reduction of the manganese content of the supernatant from 10 to 1.82 mg/L. Recycling the supernatant will increase the iron and manganese concentrations in the raw water to about 1.86 and 0.52 mg/L from the present levels of 1.5 and 0.5 mg/L respectively, and therefore has a minimum impact on the current performance of the plant.

Key words: groundwater treatment; iron and manganese removal; backwash water, recycling.

Introduction

Groundwater treatment plants are usually designed to remove iron, manganese, hardness, dissolved solids and trace inorganic substances present in the raw water. The treatment processes involved depend on the elements to be removed, but in all of them filtration is a common and an important process. The type and quantity of waste water produced in water treatment plants vary significantly from one raw water source to another and from one type of treatment to another. Waste water which is a byproduct of water treatment processes contains materials that are removed fron raw water and residues from chemical addition.

Backwashing of water treatment plant filters is considered the second major waste water producing process (1). About 30 to 40 per cent of the total solids produced in water treatment plants appears in the backwash water, and in plants practicing iron removal 50 to 90 per cent of all solids removal occur in these filters (2). Filter backwash water represents a large volume of liquid with relatively low solids content. Typically, two to three per cent of all filtered water may be used for filter washing (2, 3).

Backwash water can be returned to the head of the treatment facilities after some type of treatment. The supernatant resulting from the separation of solids of the plant waste water can be recycled and fed back to the plant water inlet for reuse. The solids remaining in the supernatant after settling would be low compared to the solids removal capability of the water treatment process. The advantages of recycling backwash water in plants using alum as an example, include improved settling of basin sludges, lower filter head losses, elimination of waste discharge, and recovery of much of the waste water (4).

In practice, backwash water is discharged to a detention reservoir. The capacity of such reservoir depends on the number and area of plant filters, the frequency of backwashing and the maximum and average day's output of the plant (3).

The removal of iron and manganese from groundwater supplies is usually accomplished by aeration followed by sand filtration. Since concentrations of iron and manganese in groundwaters are generally low, the volumes of solids produced are correspondingly small. The sludge produced is red and black in color which consists of ferric oxide, manganese oxide, and other iron and manganese compounds (5).

Groundwater treatment in the Central Region of Saudi Arabia is aimed at the removal of dissolved solids, hardness, and in some cases iron and manganese. All major plants in the region utilize reverse osmosis technology except Buraydah Drinking Water Treatment Plant (BDWTP) which has only aeration and filtration process. All of the plants except BDWTP have been designed to recycle the filters backwash water after clarification.

The objective of this paper is to present the results of an experimental study for the purpose of evaluating the feasibility of recycling backwash water after its clarification without any chemical addition at BDWTP to achieve a better utilization of the limited groundwater supplies in the region.

Study Location

The study was conducted in BDWTP which was designed for the removal of iron and manganese from groundwater supplies without utilization of filter backwash water.

The BDWTP plant with a maximum design capacity of 96000 m^3/d is located in the city of Buraydah, the second largest city in the Central Region of Saudi Arabia. The plant was constructed in 1985 with the aim of removing iron and manganese from groundwater supplies. It is supplied with water from 21 nearby deep wells. The raw water contains concentrations as high as 900, 300, 360, 160, 1.5, and 0.5, mg/L of total dissolved solids, hardness, chloride, sulphate, iron, and manganese, respectively.

Treatment of raw groundwater in the plant is accomplished through aeration, filtration and chlorination (Figure 1). Two aeration towers; 13 m high and 5 m in diameter utilize diffused air and are available for the oxidation of iron and manganese in

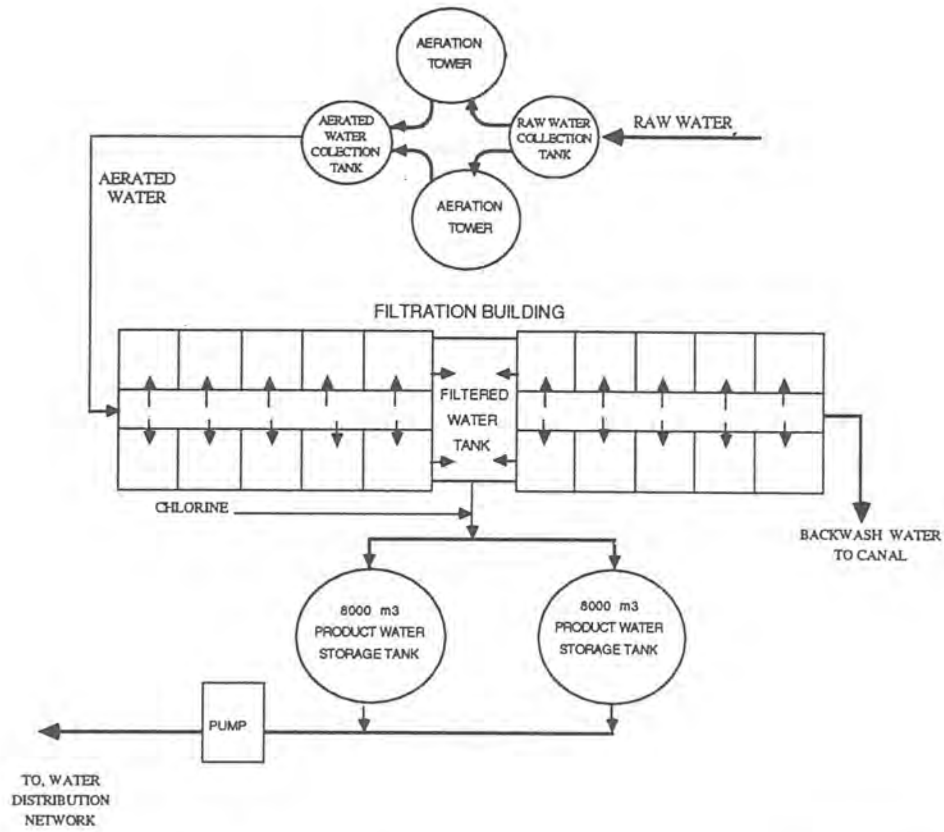

Figure 1: Schematics of water treatment processes at BDWTP

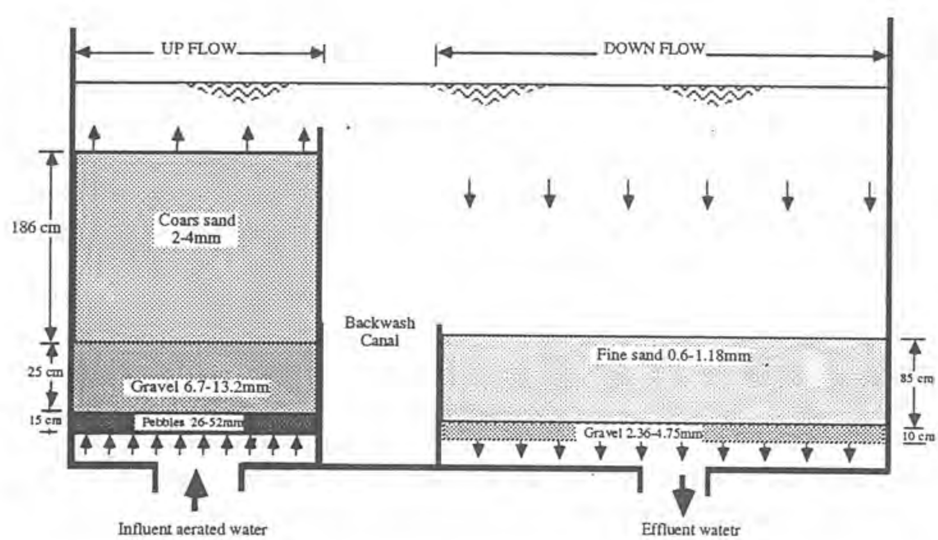

Figure 2 : Cross section of a typical sand filter in BDWTP

addition to the cooling of the raw water. The filtration process consists of 20 conventional gravity filters. Each filter is divided into an upflow (19.7 m^2) and a downflow (39.3 m^2) sections. A cross section of a typical filter is shown in Figure 2. Filters have capacities of 200 m^3/h and each is backwashed every 24-36 hrs using filtered water in the following manner:

1. Air is pumped at a rate of 444 L/sec. through the bottom of the upflow section for 7 min followed by a mixture of air and water for 10 min then water washing is followed for 7 min The water flow rate is 200 L/sec.

2. Air is pumped at a rate of 294 L/sec. through the bottom of the downflow section for 4 min, followed by 7 min of water washing only. The water flow rate is 350 L/sec.

The average water requirement for a filter backwash cycle is about 350 m^3. The backwash water is directly discharged outside through a canal to an open area about 1.5 Km south of the plant. The filtered water is chlorinated prior to storage in two 8000 m^3 product water reservoirs.

The average daily water production of the plant during the period 1410-1413 H along with the quantity of daily used backwash water are shown in Figure 3. On the average about 2018 m^3/d of product water is used in 1413 H for filters backwashing and this represents 3% of the plant average daily production.

Experimental Procedure

In order to characterize the quality of BDWTP backwash water in term of iron, manganese and total solids concentrations, a filter was selected and samples were collected every 3 min during the whole backwash process. The same procedure was repeated for a second filter and final results of both filters were averaged. Based on the results obtained, about 100 L of backwash water was collected during the backwashing of the upflow of a filter. The collected water represented the whole backwash water except the first 7 min, which represents the water within the filtering media that contains very high concentrations of iron and manganese. The same procedure was repeated for the collection of backwash water from the down flow section but in this case the collected water represented the whole backwashing period.

Backwash water from either sections was pumped to a collection tank. Collected water was then pumped continuously to the experimental column at predetermined flow rate. Recycling of the water to the collection tank was provided to insure homogeneous liquid and to prevent any settling. The experimental set up (Figure 4) is composed of a tank, a pump and a 4 m long by 7 cm ID plexiglass column. Sampling valves were installed at 50 cm intervals and water is allowed to overflow at 300 cm height (maximum height).

The flowrates were selected to give a maximum detention times of 2 and 4 hrs. First, the column was used for clarification of the upflow backwash water then for the

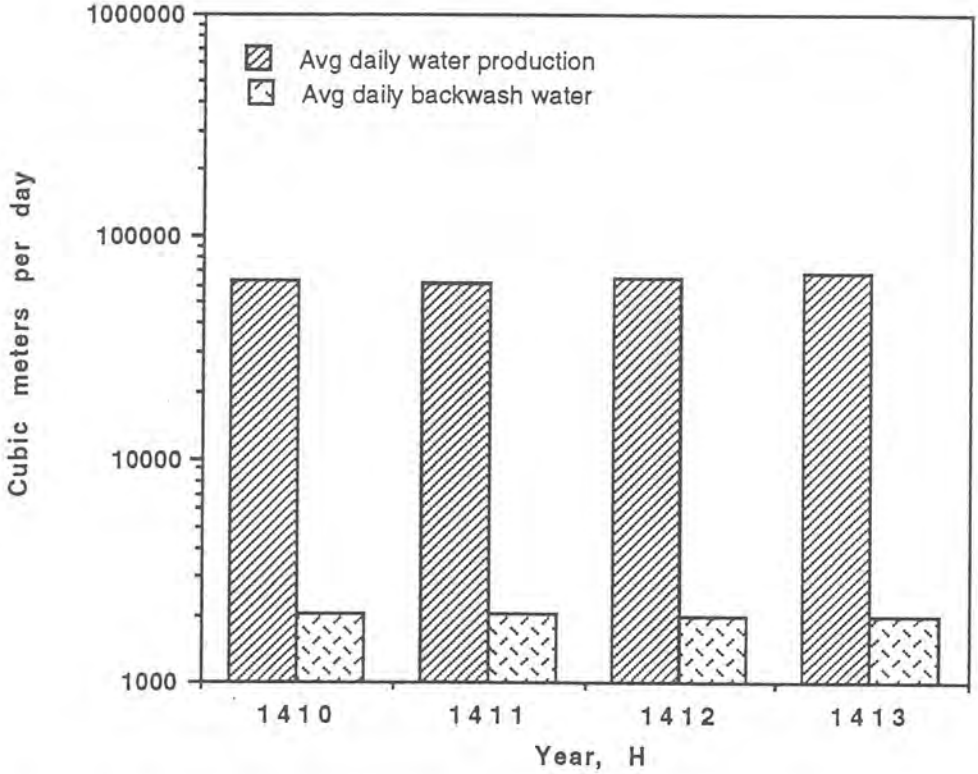

Figure 3: Average daily water production and backwash water requirement at BDWTP

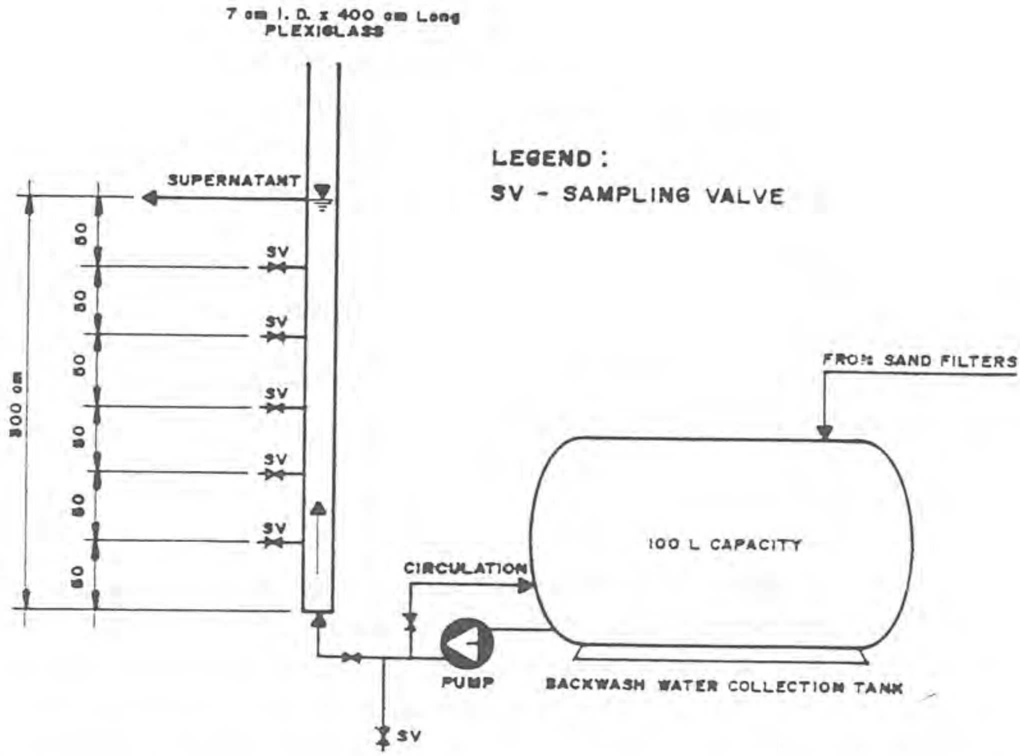

Figure 4 : Experimental column setup for backwash water clarification

down flow. For each case, water was pumped for 6 hrs at the low flow rate, then samples were collected twice at about 30 min interval from the six different heights as well as from the influent water. Water flow rate was then increased to give 2 hr detention time. Samples were collected for iron, manganese, and total solids analysis. For Fe and Mn analysis, samples were collected in plastic bottles with acidification to pH<2. For total solids analysis samples were collected in 300 ml glass bottles.

Analysis of samples for total iron was carried out using Perkin Elmer 1100B Atomic Absorption Spectrophotometer in addition to the use of HACH DR-3000 Spectrophotometer after acid digestion. Manganese was determined using the latter technique. For total solids, the procedure outlined in the Standard Methods (6) was followed.

Results and Discussion

Backwash water quality results

Results of the backwash water analysis for iron, manganese and total solids are shown in Figure 5. The maximum iron and manganese concentrations in the backwash water of the filters upflow section were found to be 354 and 11.3 mg/L, respectively. This has occurred at about 3 min from start of the backwash cycle. The minimum concentrations of the two parameters at the end of the upflow wash cycle were 0.45 and 0.1 mg/L, respectively. For the down flow section, the maximum concentrations of iron and manganese were 97 and 6.5 mg/L, respectively. At the end of backwashing, the concentrations of these two elements were 0.97 and 0.4 mg/L, respectively. It is obvious that the minimum values of iron and manganese are higher in the backwash water of the downflow section than that of the upflow. This is partially due to the shorter time used for backwashing the downflow section as compared to the upflow section (11 min Vs 24 min) as well as differences in the used air and water flow rates. Water flow rates at the up and down flow sections were 10.15 and 8.91 L/sec-m^2, respectively.

For total solids (Figure 5b), the maximum values obtained in the backwash water were 784 and 164 mg/L for the upflow and downflow sections, respectively. The corresponding minimum values were 52 and 0 mg/L. The values of total solids have reached their constant levels at about 10 and 5 min from the start of backwashing the upflow and downflow sections, respectively.

Column study results

Based on the results of the backwash water quality, water was collected from one filter during the backwash process of the upflow section excluding the first 7 min. The average iron, manganese, and total solids content of the collected wash water were 50, 10, and 96 mg/L, respectively. For the downflow section, the corresponding values were 31, 4.5, and 80 mg/L. The results of column up and downflow sections wash water are shown in Figure 6 and 8, and also summarized in Table 1.

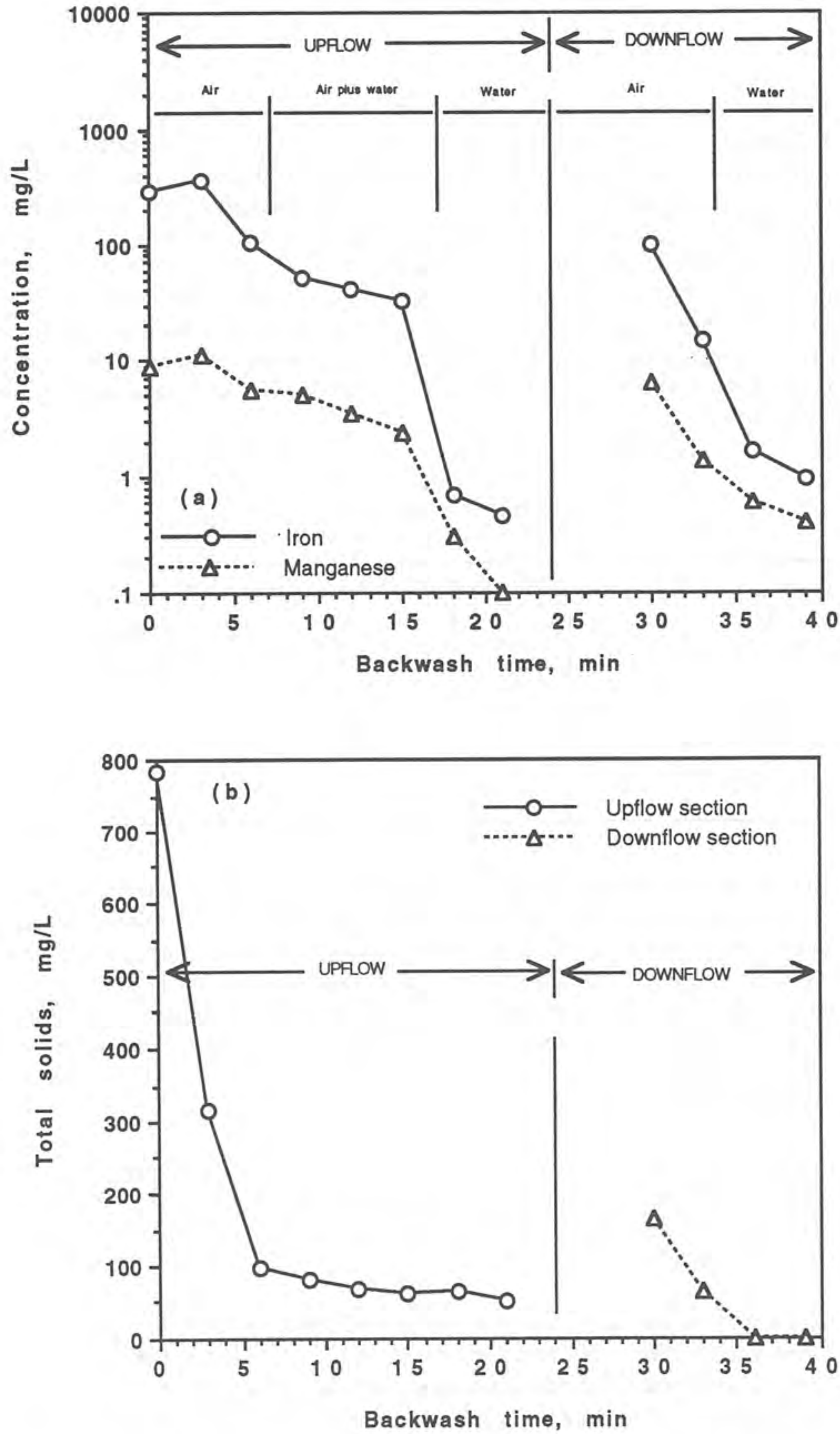

Figure 5: Quality of the backwash water of BDWTP
(a) iron and manganese, (b) total solids

For clarification of the upflow section backwash water, the supernatant water quality (Figure 6a) indicated that iron concentration at 3 m height was 33 mg/L for the 2 hr detention time as compared to 12.9 mg/L for the 4 hr detention time. No major reduction in iron concentration above 2.5 m of the column height for the 4 hr detention time. Manganese minimum values (Figure 6b) that were reached for the 2 and 4 hr detention times were 3.7 and 1.0 mg/L, respectively. The manganese levels did not vary that much above 1.5 m of the column height. As for total solids (Figure 6c), the minimum values were found to be 72 and 20 mg/L for the 2 and 4 hrs detention times, respectively. Total solids for the 4 hr detention time, remained constant above 2.5 m of the column height. This would indicate that the remaining iron and manganese were in the soluble form.

Table 1
Summary of minimum values obtained for the supernatant water of the column study

Wash water from section	Minimum values obtained (mg/L) for					
	Fe		Mn		Total solids	
	2 hr	4 hr	2 hr	4 hr	2 hr	4 hr
Upflow	33	12.9	3.7	1.0	72	20
Downflow	20	15	1.82	1.2	42	24

The clarification results of the downflow section backwash water (Figure 7) indicated less variation between the 2 and 4 hr detention times. Iron minimum values were found to be around 20 and 15 mg/L for the 2 and 4 hr detention times, respectively. There was no height effect above 1.5 m. The same observation is drawn on the manganese levels where the minimum values of 1.85 and 1.2 mg/L were reached for the 2 and 4 hr detention times, respectively. The minimum solids contents of the supernatant water (Figure 7c) were measured to be around 42 and 24 mg/L for the 2 and 4 hr detention times, respectively.

Comparison of the results of the up and downflow sections supernatant water clearly indicate the differences in the settling mechanisms of the flocs present in the wash water. Downflow wash water flocs were small, and therefore, less variation was observed for the two selected detention times.

Assuming that the daily backwash water in BDWTP is 2000 m^3, the required surface area of a clarifier with 4 hr detention time and 3.0 m height would be about 111 m^2. Of course a backwash water collection tank would also be needed. If the supernatant water is mixed with the plant incoming water then based on the study findings, the iron and manganese levels would be expected to increase by 24% and 4% respectively. Their concentrations would then be 1.86 and 0.52 mg/L instead of the assumed present levels of 1.5 and 0.5 mg/L, respectively. At these levels there would be no major impact on the available treatment in BDWTP.

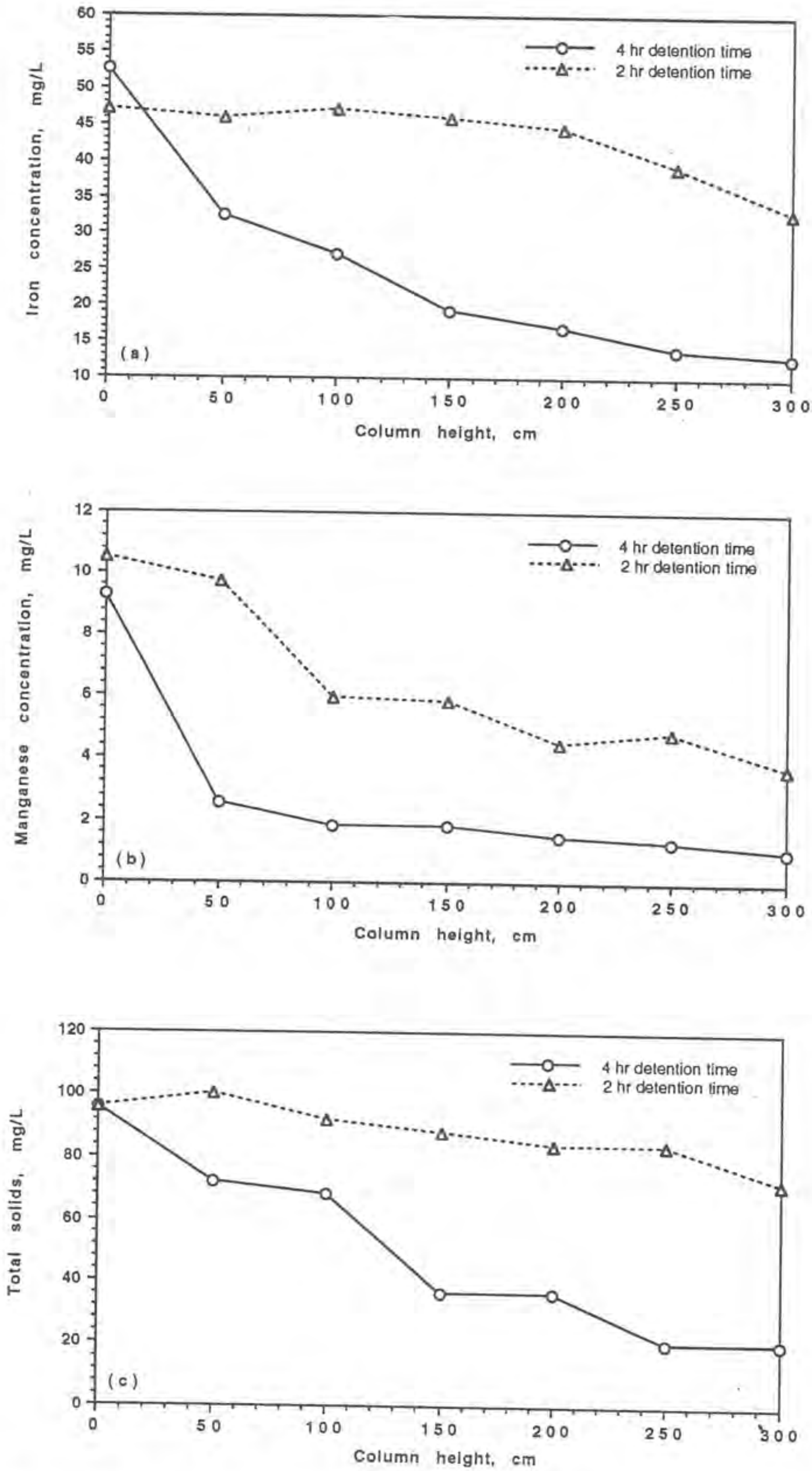

Figure 6: Quality of the clarified water of the upflow section (a) Iron, (b) manganese, (c) total solids.

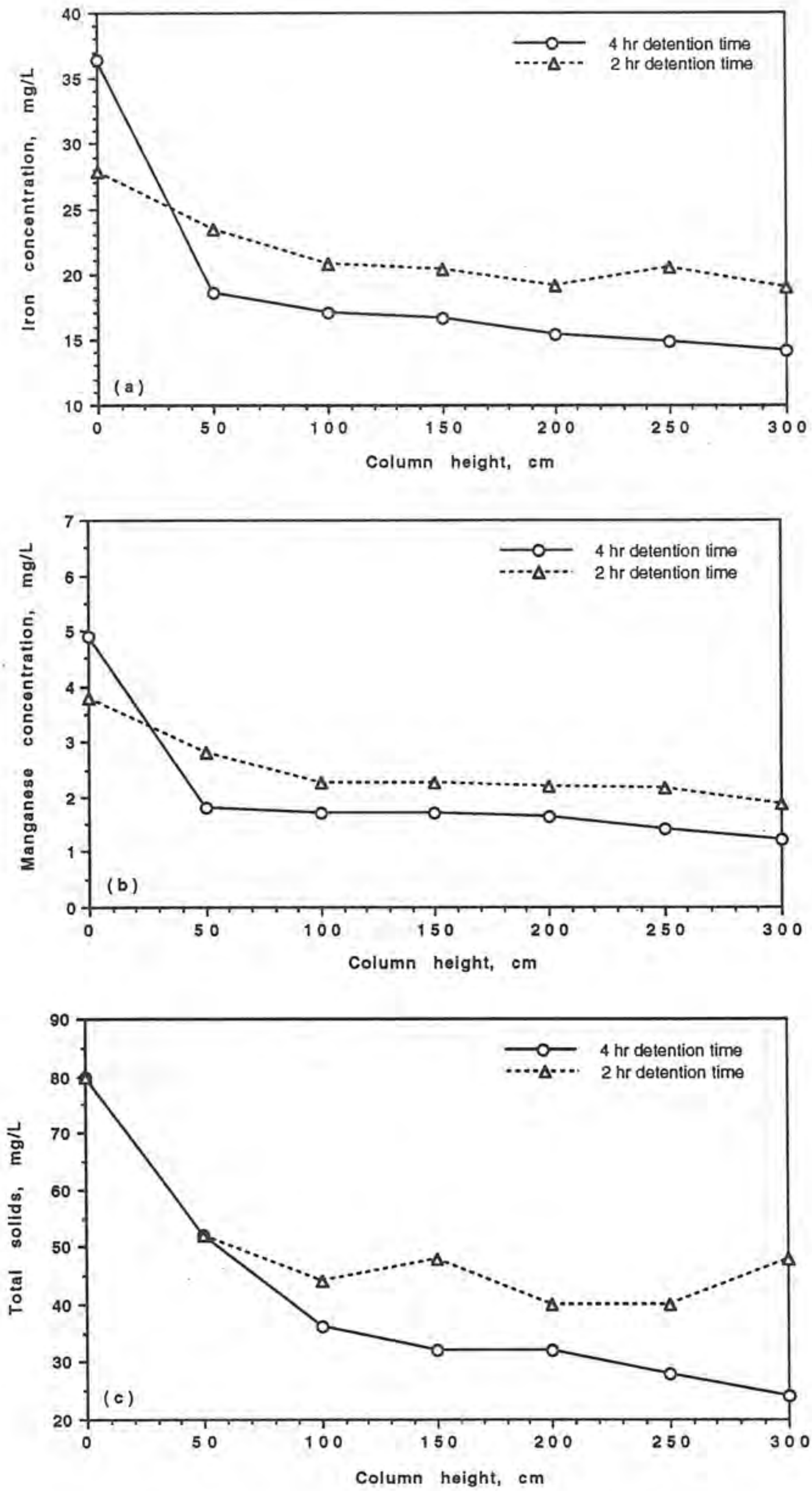

Figure 7: Quality of the clarified water of the downflow section
(a) iron, (b) manganese, (c) total solids

Summary and Conclusion

The paper has presented information on backwash water quantity and quality of an iron and manganese removal ground water treatment plant in the Central Region of Saudi Arabia. Such information have indicated that as much as 3% of the product water was used for backwash purposes. The iron and manganese content of the backwash water reached values of 354, and 11.3 mg/L, respectively. The total solids reached a value of 784 mg/L.

Results of pilot plant column clarification studies showed that it is possible to reduce the iron content of backwash water from 50 to 12.9 mg/L with a 4 hr detention time and 3 m column height. The manganese content of the backwash water was reduced from 10 to 1.82 mg/L for the same conditions. The impact of mixing the supernatant water with the raw water of BDWTP would increase the iron and manganese concentrations by about 24 and 4%, respectively.

It is recommended that more studies be carried out with different setup configuration and with the use of aeration for the wash water, prior to clarification.

Acknowledgement

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Iranian Experience in Water Supply & Distribution Systems in Rural Areas

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Iranian Experience in Water Supply & Distrubution Systems in Rural Areas

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ABSTRACT: In the Islamic Republic of Iran about 77.5% of rural dewellers gained by hygienic drinking water networks and about 84% of them have access to it (by 1993). Due to dispersion of rural settlements, unsuitability of chemical & bacteriological characteristic of water from one hand, and economical aspects on the other hand, so necessity of designing and constructing drinking water supply projects for villages as a summation instead of seperated ones will be so obvious. In this field about sixty grand and another three hundred potable water summation networks has been done, one hundred distribution systems are under way and fiftysix are under different phases of survay (by 1993). In this article all points of water resources in I.R.Iran, the experiences in water supply and distribution systems in rural area will be discussed.

KEYWORDS: Rural area, Water supply, Distribution system, Summation Networks.

Introduction: The I.R.Iran with an area about 1,648,195 sq.km, average annual precipitation 248 mm, total population of 58.11 millions, growth percentage for urban & rural areas 4.3 & 2.02 respectively, has 25 provinces, 450 cities and 65,146 villages. Urban and rural population possesing 33.138 and 24.972 millions in turn (Population informations in 1991). The responsibility of providing drinking water in rural areas belongs to Ministry of Jehad-e-Sazandegi, and in urban areas to Ministry of Energy.

The provision of safe water and management of waste water has had a central role in reducing the incidence of many water borne or water related communicable diseases[1,2]. Before the victory of Islamic revolution, the diseases associated with insufficient, poor quality, and contaminated water remained among the serious public health problems for much of the rural population in Iran. Water shortages, widely scattered rural population, inadequate financial and human resources, poor management, impose serious constraints on the provision of adequate potable water supply for large number of rural dwellers.

THE SCOPE OF PROBLEM: Lack of potable ground water and appropriate facilities, inadequate access to safe and convenient water are the main problems of Iranian rural population. The other problems are as follows.

1) Single well inside or outside the village for the vast majority of rural dwellers, travelling towards water sources, waiting in queues, hauling water is still time consuming and heavy work.

2) Multiple neighborhood stand pipes cause a number of difficulties in sampling, quality control and specially testing for residual chlorine. In areas with a communal diesel-pumped system, the villagers are unwilling to use the system or pay for the fuel to operate the pumps. At times in some villages many of the hand pumps would not function due to lack of proper maintenence and repair. In some areas rural dwellers receive water only 3 to 5 days per week due to shortage of water. When the pumps break down, the villagers return to their traditional and unhealthy water resources.

SOLUTION TO PROBLEMS: In 1975, an estimated 70% of rural population in Iran were without access to a safe and convenient source of water [3]. In an effort to improve the situation, in the first national development of five year plan (1989-1993) the Iranian Goverment has invested 2E11 Rls (200 Billion Rials) pluss $ 58E06 (58 Million dollars) for providing water supply and sanitation projects. In 1993, 28480 rural areas with a total population of 18E06 have benefited from safe drinking water. The percentage of the rural population enjoying safe potable water has reached from 30 in 1975 to 84 in 1993. Our decision makers reached to this idea that main economic and social benefit of financing, building, and maintenance of water supplies and sanitation facilities can improve health of rural population as mentioned in litrature [4]. Investing in water supply and sanitation is not always economical due to the fact that some rural households do not have enough income to afford the costs. However, where the rural dwellers are able and willing to pay for the consumed water should be asked to do so [5].

The rural potable water summation networks information which has been completed and under-use in 1993, is shown in table 1. Moreover, in this year about 10 summation networks with 35 villages and 3200 households which already have been built, reconstructed and developed. Also due to the problems that mentioned before, as shown in table 1, some villages has been covered by potable water service in the singular form. In this year about 144 singular projects are reconstructed and developed.

Community decision makers believe that placing a charge on incremental use sufficient to cover the financial costs may cause people to return to traditional sources as took place in other parts of the world. Savings in health care costs due to improved water quality would be more than make up for the cost of providing potable water facilities [6,7,8].

To overcome the above mentioned problems and to lay the foundation for accelerated progress in the future, Jehad-e-Sazandegi needed to throughly reassess their policies and investment strategies. Jehad took into account many factors and reached the conclusion to construct rural potable water supply complex [6,9,10,11].

The most obvious benefits of water supply complexes are as follows.

1- Time saving
2- Better access
3- Better quality control and monitoring
4- Increased water consumption
5- Possible improved health effects
6- Increase in household's income
7- Better quality for water users
8- Better planning, implementation, monitoring and evaluation
9- Better maintenance and repair of water supply systems, reducing the leakage rate in the pipes
10- Helping and providing work for the low-income groups in rural areas
11- Management of quantity by revising the allocation of water to different rural areas.
12- Safeguarding quality by clear and enforceable pollution controls
13- Encouraging the development of techniques and technology for economical use of fresh water
14- Better management of water collection, treatment and distribution
15- Involving all sectors and responsible groups in the community and the decision making process.

WHAT HAS BEEN ACHIEVED SINCE START OF REVOLUTION

A breif overview of progress achieved in the 1975 can be seen from Fig. 1. Between 1975 and 1993, the coverage in urban and rural water supply increase from 76% to nearly 100% and 30% to 84% respectively i.e. 54% increase in rural coverage. In addition to the above mentioned statistics, the Jehad is currently supplying potable water for 1820 villages and nomadic units, with a total population of half a million people by means of 350 mobile water tankers each year. Recently three mobile water treatment and a packaging unit with capacity of 550 packets (250ml to 1 litre) per hour for each unit is mounted and operated. In 1975, 1 in 5 rural residents had access to safe water. By 1993, the portion is 4 in 5. The WHO target of urban and rural water supply coverage by the end of International Drinking Water Supply and Sanitation Decade (IDWSSD, 1981-1990) was reaching from 71 to 93 and 32 to 86 percent respectively. In the CEHA technical meetings in Amman, Jordan 1991 declared that because of large population growths, the service levels in rural areas have stagnated [12]. But as Fig.1 shows, in our country the service levels in rural areas approximately have caught up with the growth rates and in urban areas exceed the guide line value. Our country acheived the above target, inspite of 8 years destructive war.

SISTAN RURAL WATER SUPPLY MASTER PLAN COMPLEX

Considering the lack of potable ground water in Sistan area and problems associated with the development of surface water treatment for each individual settlement, in 1978 central goverment adopted a master plan of water supply for entire 730 rural communities and newly developed town in the Sistan area, with the following characteristics:

-Population covered by the project is 273,000 currently and 477,000 by the year 2023.

-Area covered by the distribution system is 300,000 hectare.

-Water resources is Chah-nimeh lake.

-Method of treatment, coagulation sedimentation rapid sand filtration & disinfection.

-Maximum capacity of treatment plants 69000 cu.m/day.

-Total lengths of main supply lines 400 km.

-Ductile cast iron & Asbestos cement pipes.

Benefit-cost Analysis used in Sistan rural water supply master plan complex.

The benefits are measured for the most part in terms of the saving of the time and trouble of water pay. In this program, water borne diseases, infant mortality and morbidity and all aspects of health benefits are considered only qualitatively. Sistan rural water supply components have several salient features, including:

-Users contribution requirements

-Users expected for payment of water charge

-Community participation.

In general the cost of rural water supply depends on 7 variables as:

1- Usual cost of capital

2- Usual cost of labor

3- The cost of source (well,river & so on)

4- Size of village (The number of inhabitants)

5- The density of population (person per hectare)

6- The value of time of those hauling water

7- Operation and maintenence costs.

We should pay into consideration that rural dwellers are paying for water in another way, i.e. the time and effort it takes them to get water from waterholes or wells to where they want to use, it is a cost to them. This cost can be attributed to usage of extra time in income-producing activities. In practical applications, the following formula can be used:

(value of one hour of time saved) = (K) x (hourly wage of carriers)
+ (1-K) x (hourly earnings of
women in "marginal" time) [7]

where (K) is the fraction of the total village water that is brought to households by hired carriers and (1-K) is the fraction brought by household members themselves.

Note that where no hired carriers are used, the value of one hour saved reduces in this formula to the hourly earnings of women in "marginal time," where the word "marginal" is included simply to underline that these earnings might be lower than what women could earn for their primary income-producing formula can be used to estimate the earnings from "marginal" time:

(hourly earnings of women
in "marginal" time) = (a) x (hourly earnings of women in
petty tradings) + (1-a) x (hourly
earnings of women in other work
e.g., growing more food) [7]

where (a) is the fraction of the freed-up time that the women of village would spend in petty trading and (1-a) is the fraction they would spend in other work. Obviously, one can make only a rough guess of (a) and (1-a) in practice, but that is better than nothing.

The Iranian women's average daily wage in rural areas in agricultural work is 4480 and 2000 Rials in petty trading activities. In general 60 percent of income accrues to woman labour in rural dweller is due to agricultural and the remaining 40 percent would be in petty trading activities in marginal time.

(hourly earnings of women
in "marginal time") = (0.4)(2000 Rials)
+ (0.6)(4480 Rials)
= 800+2688 = 3488 Rials/day

If, in addition, 20 percent of the water used in this district is carried by hired carriers, and if hired carriers earn 560 Rials per hour in this work, then by the first formula above:

(value of one hour of time saved)=0.2x560+0.8x3488=2902 Rials/hour

In Sistan rural water supply complex, the investment for designing and financing is 8.2253E10 Rials. Considering the living time period about 28 years for this project, daily investment will be 86 Rials per household. The value of the time saved in water supply project is about 2902 Rials per hour. These figures can serve as a basis for estimating the economical benefits of project.

Table 1-Rural drinking water supply projects which was completed and under-use in 1993.

Sl. No.	Province	Summation Networks			Singular Networks		
		No. of Proj.	No. of Villages	No. of Households	No. of Proj.	No. of Households	Remarks
1	Tehran	3	11	300	41	13954	
2	Bushehr	2	12	1583	10	1864	
3	Yazd	12	12	1920	12	1425	
4	Semnan	4	22	3371	20	5061	
5	kohkilooyeh Buirahmadi	–	–	–	25	3029	
6	Charmahal Bakhtiari	3	16	665	8	3049	
7	Zanjan	–	–	–	37	6826	
8	Ilam	3	13	1250	5	520	
9	Luristan	8	65	3543	11	1280	
10	Hamadan	*					*No info.
11	Kurdistan	3	6	848	41	4278	
12	Sistan Baluchestan	8	199	8379	66	5739	
13	Hurmozgan	9	75	5900	60	6021	
14	Isfahan	6	14	3231	16	3884	
15	Khorasan	15	71	7176	32	6367	
16	Kerman	3	9	3535	5	2109	
17	Fars	2	9	220	38	7991	
18	Khuzestan	10	50	3083	6	1544	
19	Kermanshah	8	100	4252	3	535	
20	W.Azarbaijan	7	27	1367	37	4631	
21	E.Azarbaijan	2	10	514	70	13673	
22	Mazandaran	13	37	3964	31	7122	
23	Gilan	5	13	1306	68	6055	
24	Markazi	–	–	–	13	950	
Grand Total		118	771	56407	655	107907	

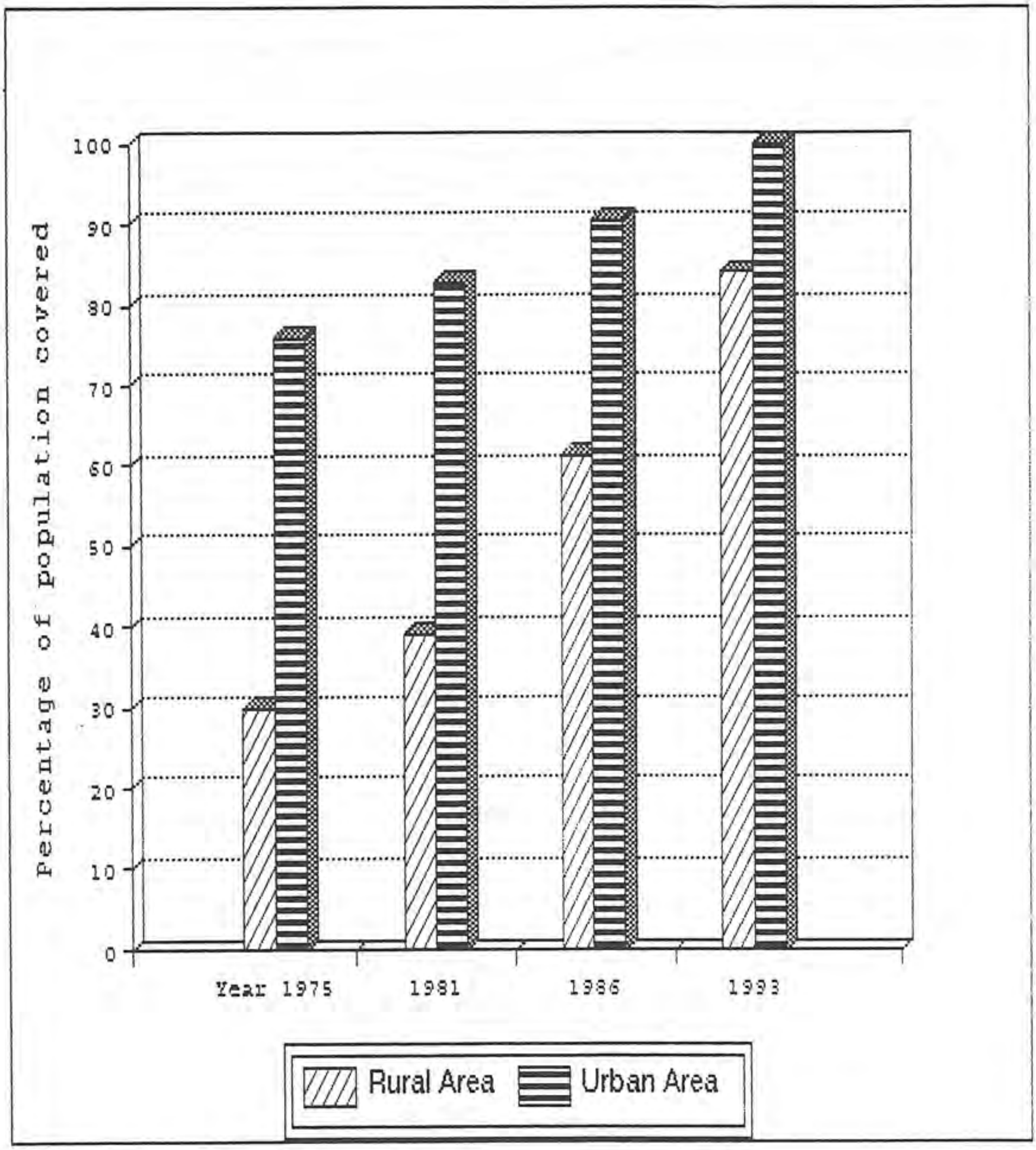

Fig. 1- Islamic Republic of Iran's rural & urban water supply service coverage in 1975-1993.

CONCLUSION:

This discussion covers improtant issues in problems of water supply in rural areas as related to strategy adopted in the water and sanitation Decade 1981-1990. [3]

This discussion is very much needed to enable the attainment of a high level services and facilities to obtain clean water for a large population. Generally speaking the commitment of the Goverment to provide safe water to rural as well as urban population of country has been fullfilled to great extent.

Research has been taken place in I.R. Iran approves the opinion that in countries with arid to semi-arid climates and scarcity of suitable water resources for drinking purposes and accessing is gathered with troubleshooting, designing and projecting the rural potable water plans should be done as summation.

In this way, the number of villages which can be gathered in one complex project and also the number of such projects depends on several objects such as local distribution of water resources, the quality and quantities, regional topography, distribution of villages, culture and economical abilities. From the health point of view purification, quality control and maintainence in summation projects can be done easily and better. So with all these reasons doing projects as summation has much more advantages.

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The Doha Water Loss Control Project

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THE DOHA WATER LOSS CONTROL PROJECT

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ABSTRACT

The project was set up as a direct result of increased revenue & capital costs, along with contents of desalinated water in a high rising water table. The major objectives of the project include the establishment of a leakage monitoring and control system and the reduction of leakage to an acceptable level. Activities in the project include leakage control, zoning and network analysis. Other activities include record verification and training of local staff. The Water Networks Department have adopted an active leakage control poiicy resulting in 50 leakage control districts being established for about 50,000 connected properties with each district being metered. Zoning of the distribution network in this way involves the operation of many valves. Daily flow readings can quickly indicate abnormal consumption. The paper concludes that it is evident that ways must be found to minimise waste, misuse, excessive use and leakage.

keywords : zoning, leakage control, network analysis

GENERAL.

This report is a general presentation of the leakage policy and practice used in Qatar.

Contents of the report are given as a basic outline of methods used to combat leakage, misuse and waste within the capital city of Doha.

No technical details are included within this report due to the general overall basic context of background information.

INTRODUCTION

Over the years potable water supplies have been moving away from natural sources in the Arabian Peninsula, particularly along the Gulf coast. Owing to the arid nature of the region, and escalating demand levels, the traditional ground water sources have become increasingly unable to satisfy the requirements for wholesome water. Ground water supplies have deteriorated in quantity with reductions in yields, and in quality as dissolved and suspended solids have increased. The abstraction of large volumes of ground water from well fields has increased the scale of sea water intrusion.

The State of Qatar is developing rapidly, and has already achieved a great deal in both urban and rural expansion. This development commenced in the 1940's when oil was first produced at Dukhan (fig. 1). Initially, production was comparatively small and it was not until the 1970's that oil revenues reached a level which enabled the development of the State to proceed at a faster rate. This expansion continually requires a reliable system of water supply and distribution for domestic, commercial and industrial purposes. Owing to the limitations of the existing ground water sources, it has become necessary to produce potable water by desalinating sea water, even though this is a costly process.

Having due regard to the high revenue and capital costs associated with the ever increasing demands for potable water, and also recognising the contribution of desalinated water to the problem of a rising water table, the State of Qatar Ministry of Electricity and Water decided to initiate the Doha Water Loss Control Project. Local consultant ASCO (Qatar) has been employed to plan and implement the project. Its major objectives are to establish systems for monitoring the flows within the water distribution system and for controlling leakage. The Study Area is shown in figs.1 and 2.

EXTERNAL SOURCES CONTRIBUTING TO RISING GROUND WATER TABLE

The Doha water cycle is illustrated in fig.3. There are two external sources of water which contribute towards the rising ground water table.

The first source is, of course, rainfall. The annual average rainfall measured at Doha International Airport is approximately 77.5mm. and in global terms this is equivalent to the Study Area receiving a daily rainfall volume of 60 Megalitres per day. The contribution to the Doha water table is significantly less than this amount because of the effects of evaporation and storm water run off to the sea.

The second source arises from sea water abstraction. Potable water production from the main desalination plants in Qatar has risen to levels approaching an annual average equivalent to 220 Megalitres per day (fig.4). Most of the distilled sea water originating from the two major production plants at Ras Abu Aboud and Ras Abu Fontas is discharged, after use and in a variety of ways, to the ground in the area covered by the Doha Water Loss Control Project. There are smaller outlying districts which receive treated water from the latter plant but which are not within the Study Area, and these include Umm Said and Wakrah.

The above figures show that, on average, a volume of desalinated water more than three times greater than rainfall volume reaches the ground water table each year.

The contributions of treated water to the aquifer can be accounted for as follows:

a) Domestic and industrial supplies - after use of water in the home, or by industry, the resulting effluent is discharged to the sewage treatment works, either via an individual cess pit with subsequent road tankering, or through the local sewerage system. The effluent reaching the sewage treatment works receives full treatment, including disinfection, and then enters the ground when used for irrigation within the Doha municipal treated effluent reticulation system. A minor proportion is pumped to outlying farms and desert lagoons.

b) Irrigation of public parks, landscaped areas, and of private gardens - potable water is used for irrigation purposes in public areas where treated effluent is unavailable or inappropriate, and also on private gardens. A preliminary estimate of the scale of potable water usage for irrigation purposes is 20% of total supply. It should be noted that not all the potable water used for irrigation purposes reaches the ground water reservoir; a significant proportion (possibly as much as 50% during certain seasons) is lost to the atmosphere through evaporation.

c) Leakage of treated water into the ground from water mains, services and fittings - leakage from trunk mains, distribution pipes and service connections is inevitable, as the experience of water authorities world-wide proves. Bursts and breaks occur constantly for a variety of reasons. By careful and diligent investigation, and by prompt remedial action, the amount of this loss can be controlled and kept down to an acceptable limit: beyond this limit no water authority would be prepared to commit its resources.

In Doha the existing consumption is estimated 436 liters per capita per day covering an approximate 540,000 supplied consumers. This figure constitutes the necessity for a public awareness compaign and strict controls in operational management. The latter has now been started since the establishment of zones from April 1994.

Along with demand problems the leakage in Doha is caused by various reasons. External corrosion on older pipe work which had not been sleeve wrapped, has created many weaknesses. These corroded pipes are also prone to bursting within higher pressure areas. Zones are thus being controlled to ensure the correct balance of pressure is present within the network.

Additional leakages in Doha are from general age of joints, pipes and fittings where maintenance and repairs are on going.

A number of techniques, both traditional and modern, are being employed to achieve success, at an acceptable and economical level.

WATER DISTRIBUTION NETWORK MODELS

Network analysis by computer of water supply and distribution systems is a means of investigating the complex relationships within a specified network between demands, pressures and flows. Network simulation is an extension of network analysis which aims to describe the operation of a network over a simulated period of time. At a lower level, the variations in demands of individual pipes can be modelled over a given period (fig.5).

As an aid to establishing the leakage monitoring and control systems for Doha, a number of computer models are being constructed to simulate the water distribution network. Doha receives its water supplies from two desalination plants which, between them, pump to five reservoir sites at West Bay, Gharaffa, Airport, New Salwa Road and Old Salwa Road. From these reservoirs, supplies are pumped to a number of water towers and elevated tanks located around the City. Local distribution zones are supplied by gravity from elevated storage.

The construction of a network model needs accurate and reliable records for the water distribution system; Project strategy has been to divide the Study Area into a number of zones, with one of the first tasks in each zone to verify the 1:1000 record plans held by the Water Networks Department. Whilst accuracy of mapping is important, the main objectives are to record and indicate on site locations of water pipelines and apparatus so that they may be found easily in the future, also to prove that valved pipe interconnections operate in the way shown in the records. Buried pipes and valve surface boxes are found using electronic underground pipe location equipment. It is necessary to operate all the sluice valves fitted in the water distribution system to prove that they turn satisfactorily, confirming their current status (i.e. open or closed), and direction of closing. Most valves in Doha close in the clockwise direction. Faulty valves which will be essential to successful isolation of a sub area for leakage control purposes must be repaired or replaced.

A preliminary network model is thus developed using all the data of pipe configurations, lengths and diameters collected during the records verification phase. Additional information required includes ground elevations, top water levels and dimensions of service reservoirs water towers, and details of the operation of pumps and specialised valves.

Assessments of the probable demands for water in each part of the model, (balanced by the known volumes of water supplied from each source) are used to complete the model prior to field calibration. In Europe the standard practice is to assign to each section of pipeline within the model the number of properties connected to that pipeline, and to use an average water demand figure for each property in order to generate a value for the likely flow within that pipe. In Doha the range of property types and water demands are very large. In order to determine average water demands for various types of consumer, a programme of recording and analysing of individual consumer demands have been completed by the Project team using a number of special meters. These are each fitted to selected consumers' service pipes, with an electronic data logger to record the flow patterns. Using this method of metering needs a large amount of data to be collected before any reliable estimates of typical consumption patterns can be derived. An alternative procedure, is to install one large meter in the main pipe serving a defined area of supply with known boundaries. The flows recorded can be related to the total numbers and types of properties lying within that area.

The Project team has also adopted the second solution by utilising a number of *insertion probe flow meters* (fig 6) which are easy to transport from site to site, and are inexpensive to maintain. An electronic data logger is attached to record both flows and pressures in the pipeline over the chosen period of time. These temporary meters are also used as an aid to the design of permanent meter installations. Permanent meters will be fitted in the main pipelines in due course as part of the leakage control and monitoring system, but they are expensive to buy and install, and are not economic to employ in all circumstances.

The network model is finally calibrated by comparing the results for flows and pressures with measured field data, and at this stage its validity is established. The major use of the completed network model is to help in designing supply zones and leakage control areas. Other applications include predicting the effects of future system modifications, and determining valving off arrangements for leakage control purposes and in the event of a major burst.

LEAKAGE CONTROL AND WASTE DETECTION

Current leakage policies vary worldwide from high level inspection to visual chekcks deriving from consumer and other reports. Consumers in Qatar on mains supplies are required to have water storage equivalent to 24 hours installed at ground level, from which the water is pumped. As a result, the water undertaking is often not aware of poor mains pressures owing to a burst, until consumer complaints are received, once ground tanks are empty. Thus leaks causing a substantial loss of pressure may run to waste unnoticed for more than 24 hours.

Leakage control varies from place to place & is dependant on costs of water, leakage levels & pressures.

An early task for the Doha Water Loss Control Project team has been to determine the most appropriate method of leakage control for the City, and to assess the operational resources needed to implement it effectively.

The two major aspects influencing the decision to adopt an active leakage control policy in Doha are: a) the high costs of producing potable water by desalination, and b) high per capita consumption. These and other factors such as the costs of finding and repairing leaks, the costs of installing permanent meters, and the frequency of bursts, have been used to determine the optimum leakage control policy. As a result a *combined metering* system has been elected as the preferred option.

There are about 50 000 connected properties in the Doha Study Area; 50 *districts* (leakage control districts) are being established, each containing around 1000 properties. Each district has defined boundaries and is supplied through one or more permanent meter to record flows. These meters are utilised in two modes, hence the term *combined metering*: routine daily usage is to monitor and record normal flow patterns. However, when these patterns deviate from a previously determined setting the district is checked for leakage, usually at night when it is quieter and *sounding* (listening) for leaks is more effective, when flows are at a minimum.

When the leakage control team is searching for leakage at night the permanent meter is used in a different way. Branches of the distribution network are systematically closed, and the reductions which each valve closure causes to the flows registered at the meter are compared with the expected values. The expected values are derived from a knowledge of the numbers and types of consumers being supplied from each branch, and from previous test results. In this manner, any abnormal changes in the flows to a particular section of the system can be identified. The efforts of the leakage control teams can then be concentrated in that section.

To find leaks, the leakage control team is able to call upon a variety of techniques, from the traditional *Sounding Bar* (stethoscope) to the modern *leak noise correlator*. The correlator consists of a portable computer which receives radio signals from two microphone sensors placed on the pipeline either side of the suspected leak. The computer is fed with information on pipe diameter and material, and the distance between the sensors; it then calculates (from the difference in time of the leak noise reaching each sensor) the location of the leak.

Finally, for each district a *nightline* (minimum rate of flow) is established as a benchmark to compare with previous tests (thus helping to identify trends in the levels of leakage). The *nightline* is usually expressed as litres per property per hour. Consequently, the nightlines for districts of differing sizes can be directly compared.

SUMMARY OF ACHIEVEMENTS.

Up to July 1994, noticeable achievements have been recognised for future strategy.

> The establishment of Zoning is an effective success of the project, which enables careful monitoring of the network distribution. Supplies can now be diverted accordingly during such major problems as Trunk-Mains breaks, production shortfalls or major maintenance programmes.

- Creation of 50 wastes districts have sub divided the zones into manageable smaller areas where leakage control can be programmed and monitored on a regular basis.

- Up to date drawing of district meter installations, zones, waste districts (Open Run and Step Test) together with various scale schematic drawings are now available for use in the Water Networks Department.

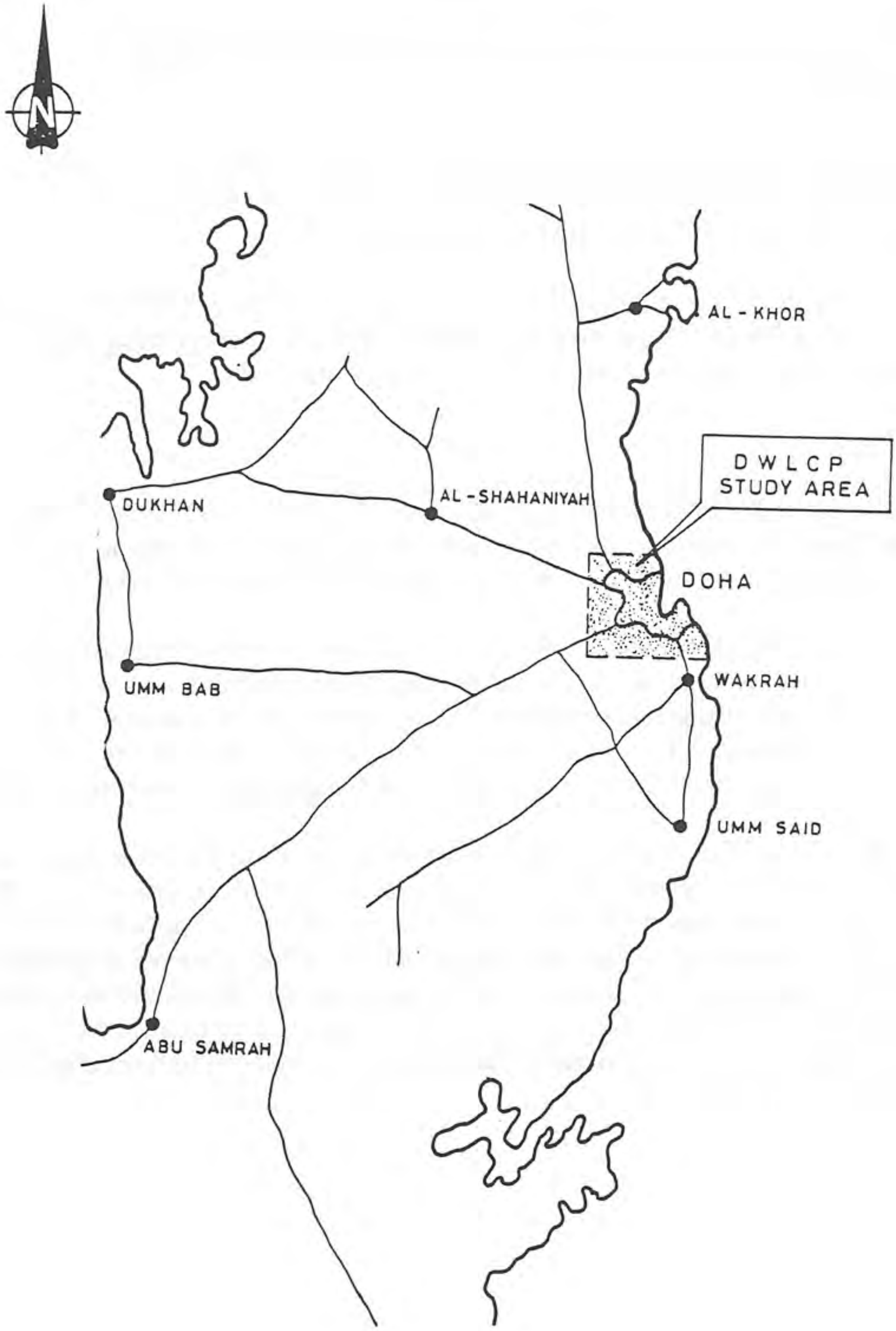
- Handover of methodology to the Water Networks Department will enable continuity for a newly established **"Distribution Control Unit"** as from September 1994 when the Doha Water Loss Control Project will be completed.

CONCLUSIONS

It is evident from the high cost of producing potable water in Qatar, & increasing demands, that ways must be found to minimise its waste, misuse, and leakage. The problems posed by the rising water table reinforce the need to continue and take the appropriate actions now.

The activities being carried out under the Doha Water Loss Control Project are designed to combat the problem in several ways. The steps being taken to monitor and control not only leakage from the water distribution system but also the daily mode of operation of the system itself, have been described. Other Project activities include a review of the current water bylaws, and participation in the Ministry of Electricity and Water's public awareness campaign.

The training of local staff is also a matter of prime importance to the future success of the leakage control system. At present, there are six local technical staff working closely with the Project team, five of them on a full-time basis. Much of their work has been of a practical nature, and this is essential to a full understanding of the subject. Network modelling, flow measurement and management of leakage control operations will fall more readily into place once basic principles have been mastered. When the consultants have finished their work it is hoped that the Water Networks Department leakage control section will be trained and already established.



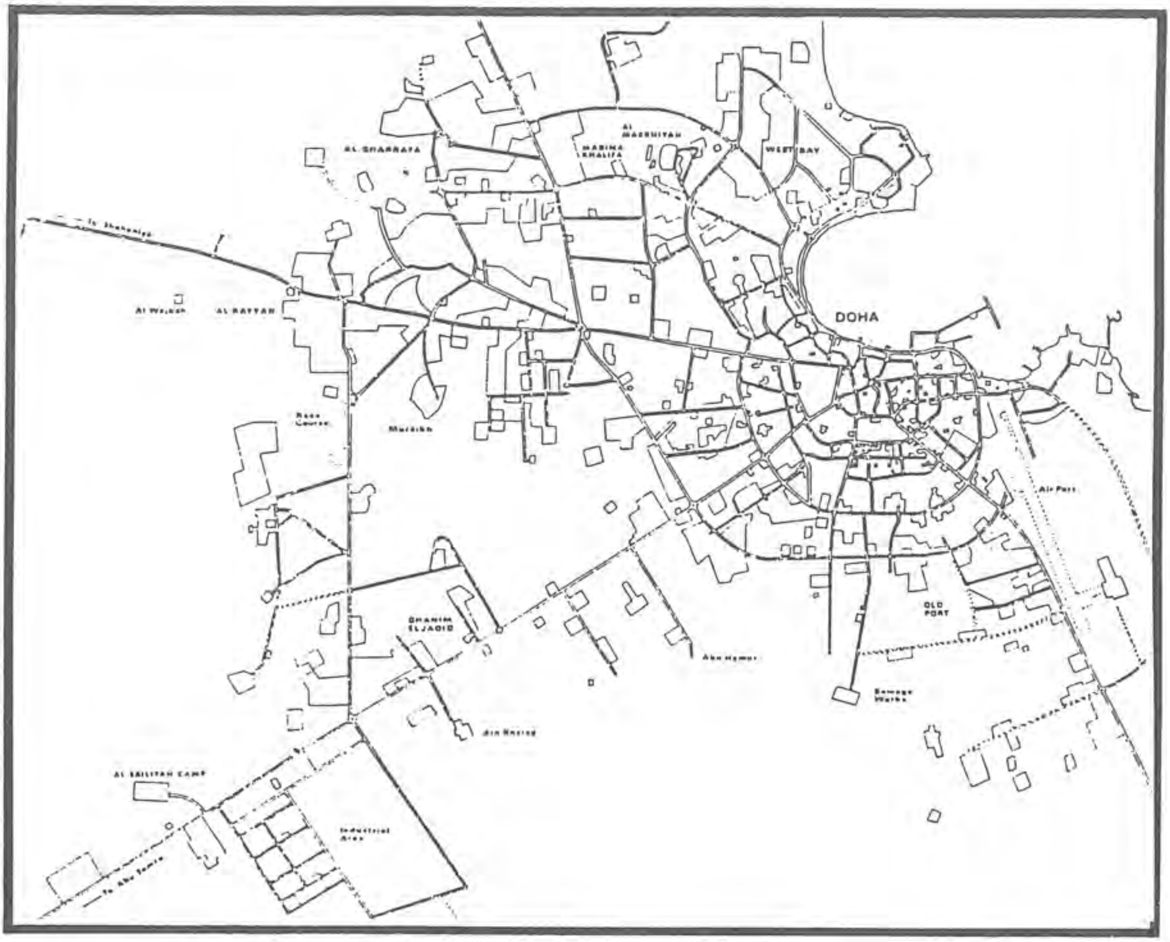
GENERAL PLAN OF QATAR

Fig. 1

ASCO

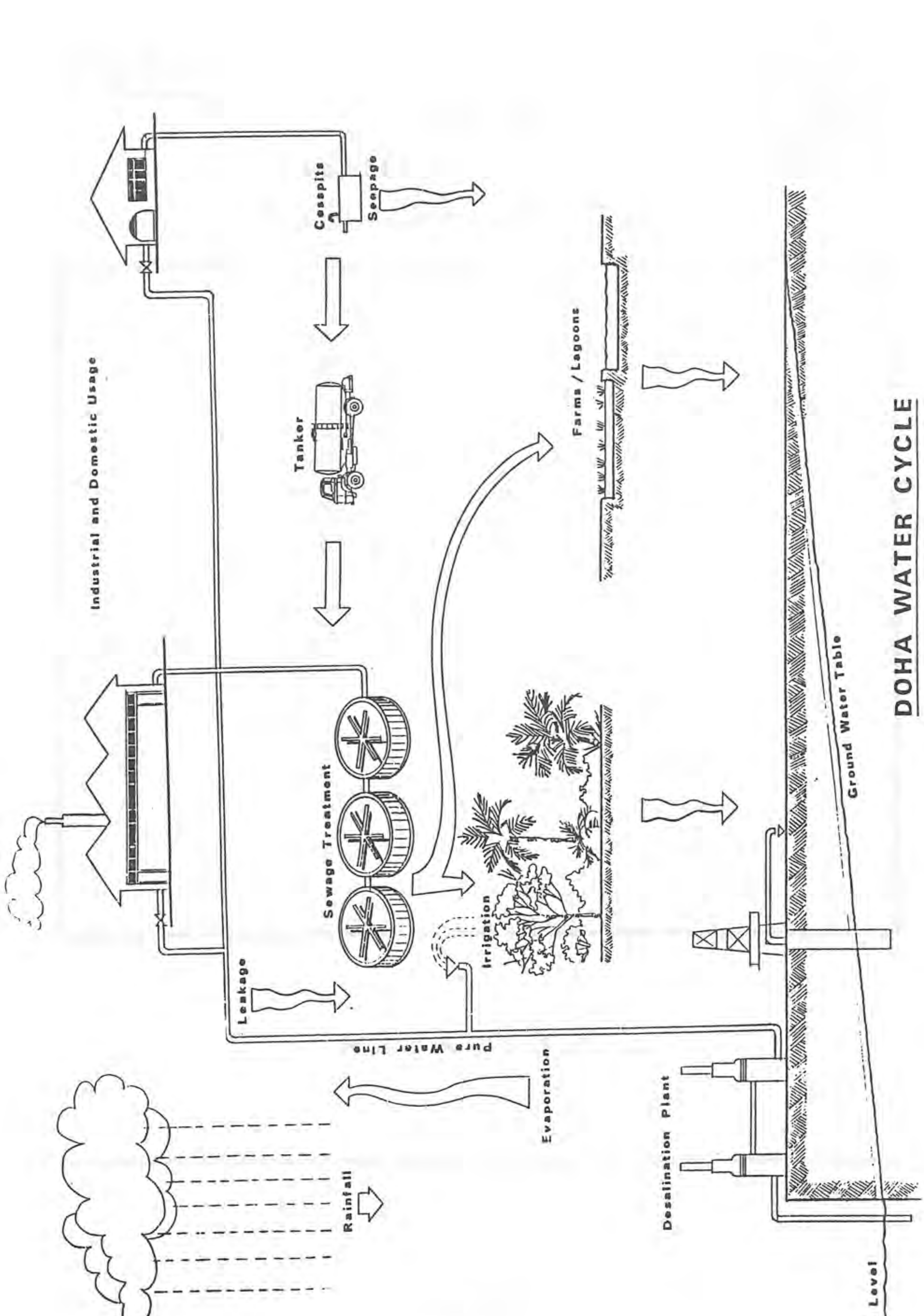
ASCO

DOHA WATER LOSS CONTROL PROJECT

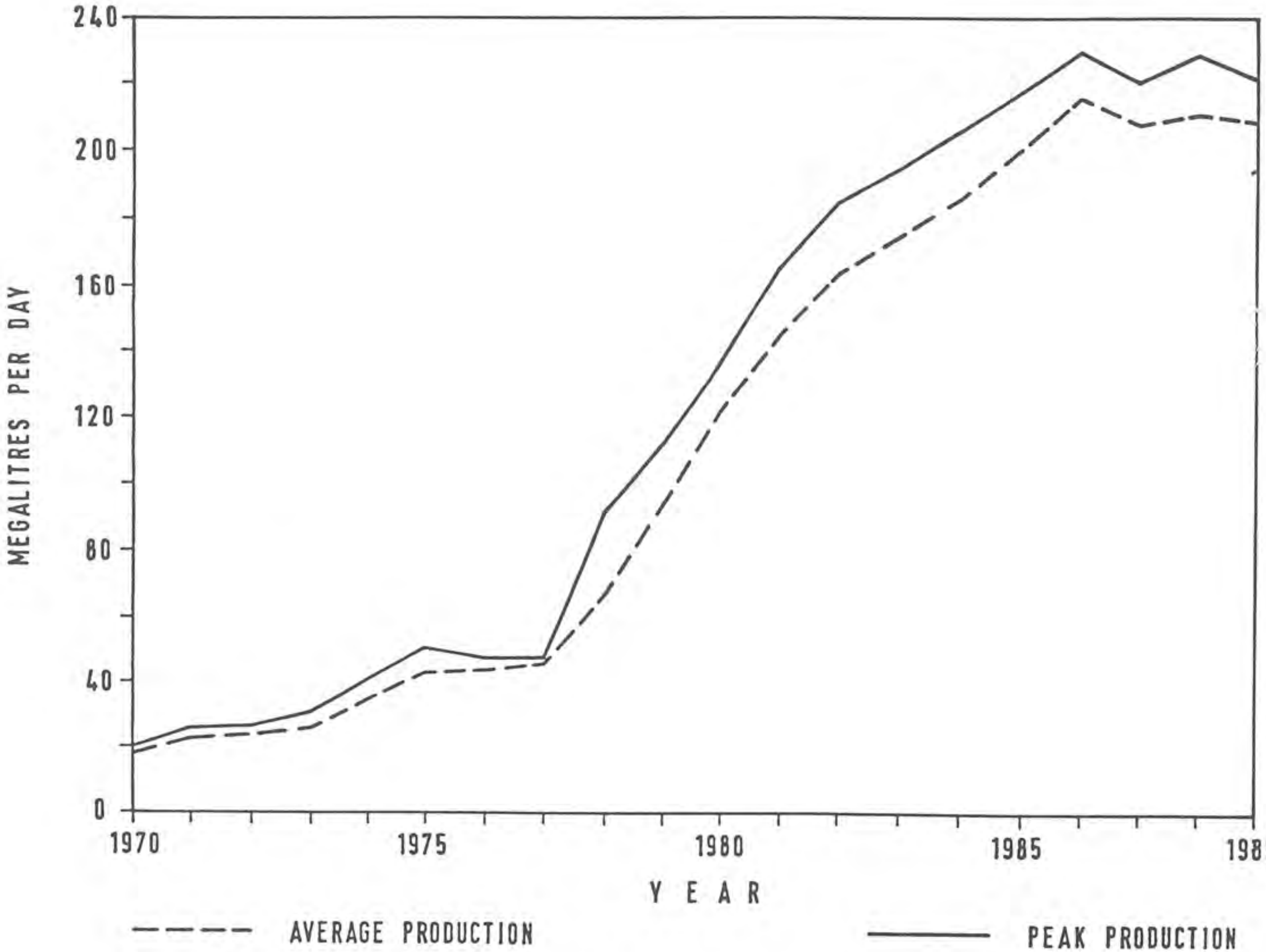


STUDY AREA

Fig. 2

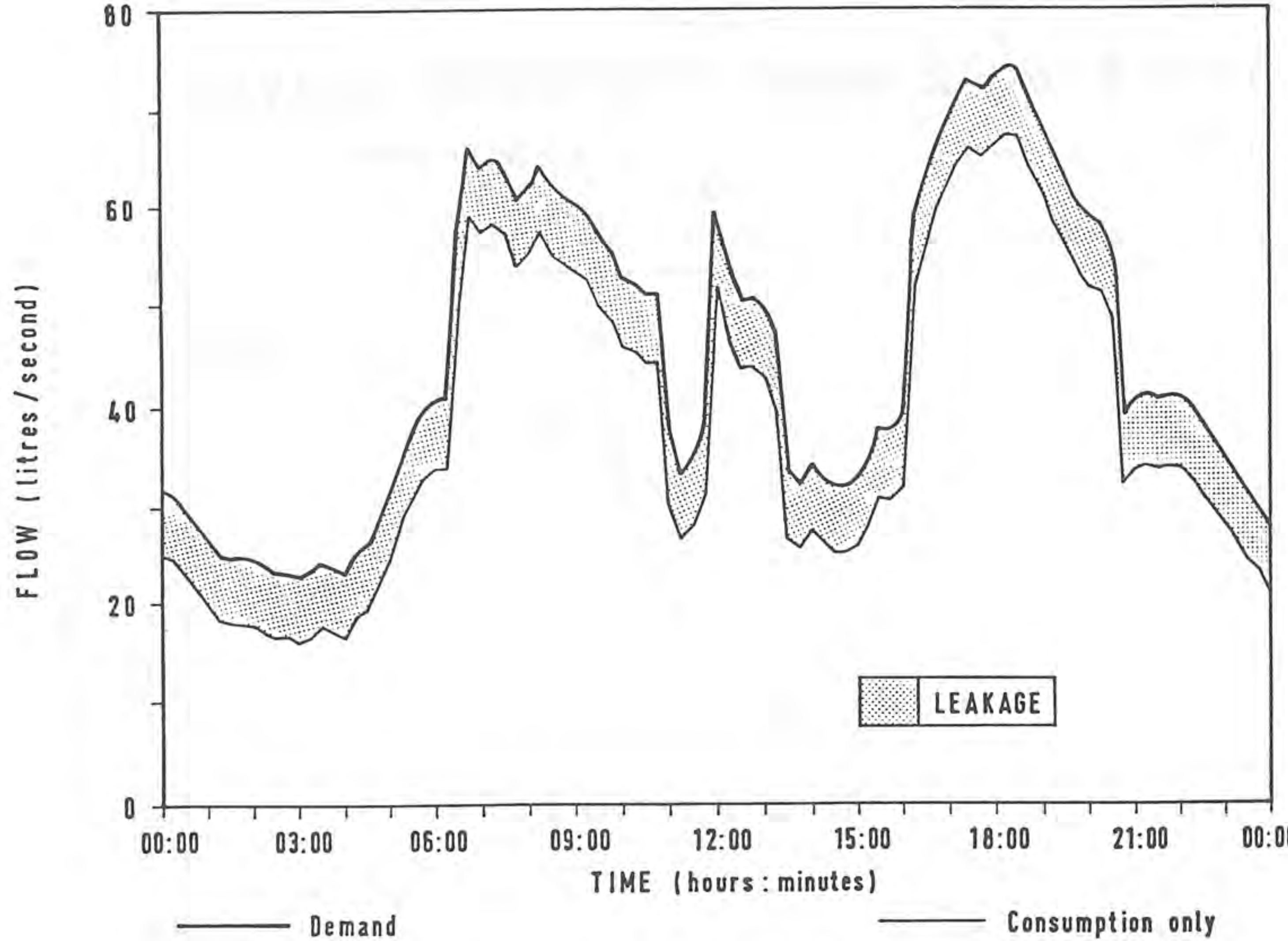


DOHA WATER CYCLE



WATER PRODUCTION FROM R.A.A. & R.A.F.

Fig. 4



Typical variation in demand during the day

Fig. 5

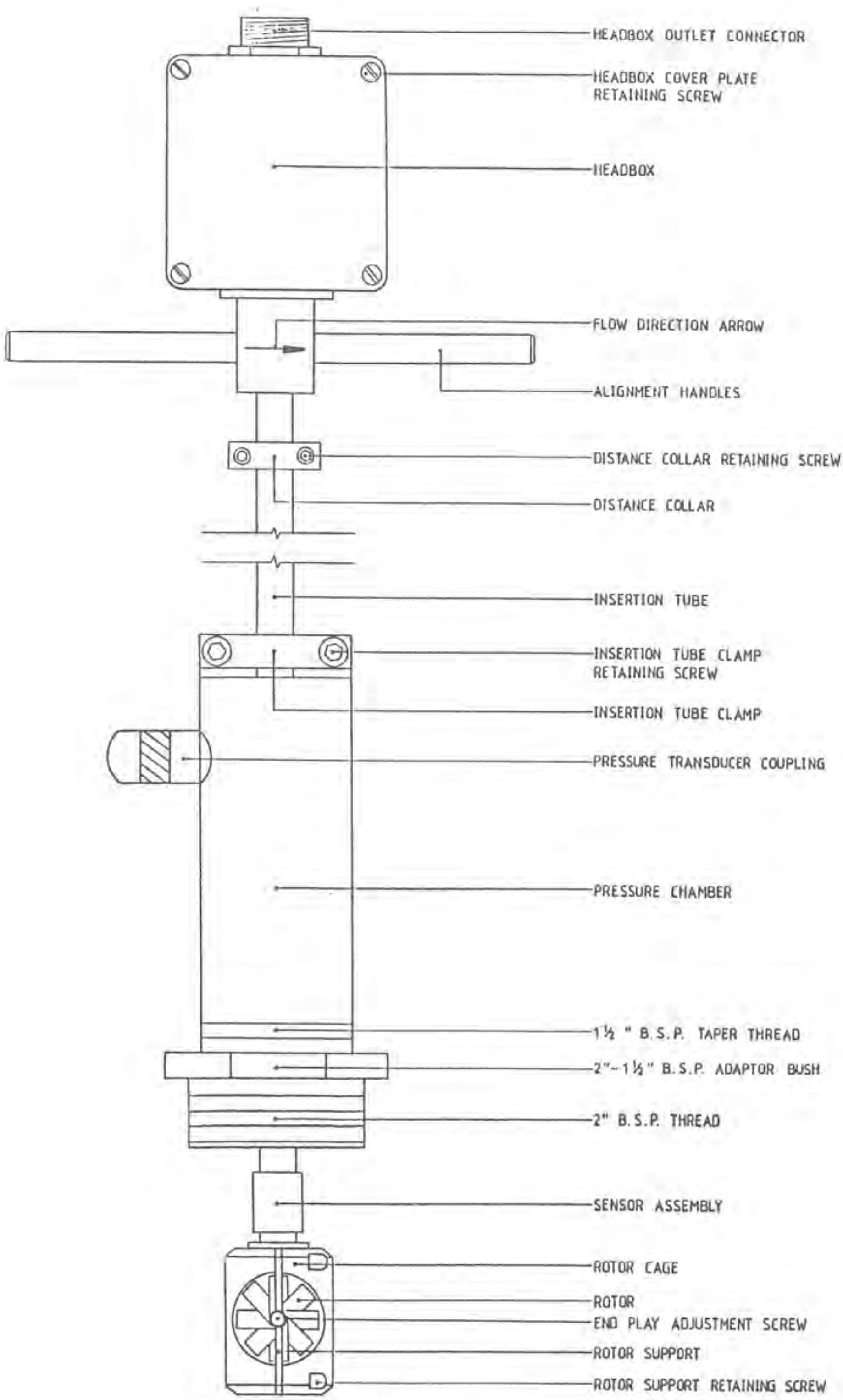


Fig. 6 – INSERTION PROBE FLOWMETER
(ROTOR TYPE)

N.B. ROTOR CAGE IS SHOWN DISPLACED FROM HEADBOX AND ALIGNMENT HANDLES BY 90° FOR CLARITY.

Session - 11
Mathematical Modeling

Mathematical Optimization of Scarce Water Resources Systems

Dr. Falah Aqeel Al Motairi and Mubarak Saad Badi

Mathematical Optimization of Scarce Water Resources Systems

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Abstract: The state of Kuwait is suffering a shortage of water supply since its existence and the problem seems to continue in the future unless better means of utilization and regulations are made. The objective of this study is to optimize a mixture of water resources to meet present as well as future water demands, while satisfying physical, economical, technical and environmental constrains. To attain this, system analysis and mathematical modeling are applied for the establishment, evaluation, selection and optimization of the designed systems. Accordingly, an efficient water system alternative is selected and represented by a mathematical model for optimization.

Introduction:

The state of Kuwait is producing large quantities of fresh water mainly by desalination, but at very high costs and selling it to the public at subsidized prices.

Therefore the development of an efficient optimum water resources system to supply the increasing demand is an essential factor for Kuwait national security. In more details, we can identify the present as well as the future water resources problems in Kuwait as follow:

A- Heavy dependence on costly desalination plants limits the future expansion needs.

B- Increasing water demand with population growth.

C- Unavailability of long-term strategical water resources development planning .

Proper planning must consider all the problems involved in water resources development, indicating all variables, parameters and constraints , that will call for the use of systems approach.

Systems approach for Water Resources Development :

Systems approach is defined as the scientific method for solving certain complex problems from a systems perspective that is then applied within a broad context. This approach is comprehensive and objective oriented which depends on systematic ways for defining of variables, parameters, and constraints for systems evaluation.

Drobny (1971) defined systems analysis as "a strategy for problem solving that relies heavily on mathematical modeling to assess the technical and economical optimum alternative system for performing various functions and meeting different needs with limited resources."The activities of the interactive systems analysis are modeling of existing and alternative systems, optimization, evaluation and selection.

Mathematical Modeling of Water Resources System

A model is defined as a representation of a real situation to give more information and general insight about the case it represents. Many different models are used for different purposes with different disciplines, and these classified as icon, analog and mathematical models. Mathematical models are the most used for water resources development, since they can represent complex water resources systems by means of mathematical relations and equations that include all variables.

In systems analysis, mathematical models are classified according to their function, such as optimization and simulation or according to the technique used for solution such as linear , non-linear , or dynamic programming.

The components of all mathematical models are defined as parameters, variables, and constraints that must be explained clearly as possible, and these in general defined as follows:

Parameters: Define known or assumed important system characteristics, such as river flow, reservoir elevation-volume relationship, production cost for a given source.

Decision variables: Define system operations , whose values are determined by solving the mathematical model.

Constrains: Limit the range of variations of the decision variables and specify the limitations of available resources.

The objective of the study is to develop an optimal water resources system that is capable of satisfying the present as well as the future water demands for municipal, agricultural and industrial purposes. This system must be developed while meeting health standards at all times and under various conditions, and fulfilling physical, economical, technical, environmental and socio-political constraints.

In order to obtain the objective in an effective and efficient manner, systems analysis and mathematical programming are extensively applied to study and analyze the complex water problems. However, there are many types of water resources problems, such as inadequate quantity to meet demands at various times and different places, undesirable water quality, and inability to distribute the available water due to shortage in the distribution system capacity. The problem of inadequate water supply has a stronger influence on the communities than the system inability to distribute the available water, which Kuwait is considered a typical type of such problem.

System Alternatives Design and Evaluation:

The possible water resources systems are established, and then subjected to evaluation criteria and comparison for the selection of the best configurations of sources, treatment facilities and development. The Evaluation criteria to be used in this research include development cost, water quantity from each source, water quality, the environmental impact of source development and socio-political impacts.

The availability of water resources for development and expansion is measured by the existence of the source itself and the technical ability to have an access into it , while the maximum quantity of water is controlled by the previaling environmental, economical and technical factors.

The environmental considerations include natural degradation resulted from water source development and the effect on the source itself (e.g, salt water intrusion of underground aquifer, high depression of water table).

The economical condition is a predominant factor in water resources development, and mostly it controls the decision-making process , even though the development of the water sources is technically proven feasible.

Components Of The Optimized Water Resources System Alternative:

1. Desalination:

The conversion of sea water to fresh water by desalination process has been the main potable water sources in Kuwait since 1953. The capacity of the desalination plants has been increased from one million gallons per day (mgd) in 1953 to 250 mgd in 1990 (Ministry of Electricity and Water (ME&W), 1991).

Although there are substantial advances in desalination technology that have improved the process efficiency, production cost of desalted water is still economically infeasible as compared with other conventional water sources.

Therefore the cost of desalination is the main constraint in expanding the use of this process in Kuwait for satisfying the ever increasing water demands. To meet this cost constraint, this system alternative will be based on the assumption that no additional desalination plants will be installed within the time frame of the plan, but a full and efficient utilization of the already constructed desalination plants will be assumed.

In spite of the high costs of desalination plants, selection consideration is based on the following facts:

A-The high construction capital cost of the plants will be wasted if these plants are not included.

B-These existing desalination plants are dual purpose-based plants producing both water and electricity.

C-The need for different independent water sources for the national water supply system.

D-The fact that other possible water resources will not be satisfying future water demands even they are developed up to their maximum capacities.

2. Local Underground Water:

The exclusive study and survey of groundwater in Kuwait proved the existence of large groundwater quantities in the sedimentary rocks of Kuwait geological group and the formation of Dammam limestone.

In general, underground water salinity increases from being brackish water with a total dissolved solids (TDS) of less than 3000 parts per million (ppm) in the southwestern and western parts of the country to salty and brine water in the Northeast and east. Water salinity is also increasing with depth.

The utilization of groundwater sources in Kuwait as part of the selected system alternative will depend on the following factors:

A-The most important natural condition of groundwater to be employed for the system benefit is the natural lateral flow in a southwest-northeast and east direction towards the discharging areas, that will lose to the sea if not intercepted.

B-Groundwater salinity increases with the Lateral flow direction so proper location of wells will produce water quantity of an acceptable quality.

C-The treatment cost of groundwater is a function of the natural water salinity and the permissible salinity after treatment.

D-Along with the treatment cost there is a transmission cost, that is related to the distance the water will be transmitted from the production field to the serving centers.

According the natural conditions of groundwater and the Iso-salinity map of Kuwait the groundwater is divided into six different zones as follows:

Zone I: This zone represents large quantities of brackish water with the lowest salt concentration, about 3000 ppm and it is located 150 Km from population centers.

Zone II: This zone is located at a distance of 110 Km from the population centers , but it has more salt and needs more treatment(The TDS is 4500 ppm).

Zone III: This zone is located about 80 Km. from population centers and the water salinity there is about 6000 ppm.

Zone IV: This zone is located at a distance of 40 Km and has water with total dissolved solids of 7500 ppm.

Zone V: This zone has a large quantity of water with high salinity up to 10,000 ppm and it is located at the areas of population centers near the coast.

Zone VI: Large quantity of water is available, but with high TDS and it is located at high depths.

3. Reuse of Municipal treated Waste Water:

The reuse of water is an important alternative for meeting the ever-increasing water demands, especially for non-potable purposes. The inclusions of water reuse as an integral component of the water supply system is a necessary task in water shortage regions, since most municipal withdrawals are returned as waste water, that can be treated and reused.

Municipal wastewater is the most widely available and least variable source for reuse purposes due to the stable physical, chemical and biological constituents that can be removed easily by one of the common treatment methods. The available technology could treat wastewater to produce good water quality that can meet drinking water standards, but high costs are expected and the acceptance of potential users is questionable.

However water reuse becomes economically feasible only when it becomes less expensive than the marginal cost of developing additional conventional water sources while satisfying all health standards. People will not find reused water acceptable for direct use even if it meets the drinking water health standards, since these standards don't necessarily include all of the chemicals and pollutants that may contained in waste water.

For these reasons water reuse will be indirectly used for underground water recharge, pasture irrigation and agricultural purposes in the case of Kuwait.

The treatment for reuse includes pollution control, weather the water is to be used for water supply or not. The true direct net cost of this treatment for reuse purposes is expressed as follows:

$$\text{Net cost} = CA - CP - CW \pm (CC)$$

CA = Advanced treatment cost

CP = Cost of pollution control treatment

CW = Cost of water treatment of an alternative source

CC = Difference in conveyance costs between the reusable supply and its alternative.

Mathematical Model For The Optimization Of The Selected Water System :

Mathematical representation of the selected water sources is developed as a programming model that includes all available water sources in the country. The aim of this model is to develop an optimum water resources system capable of meeting the designed objective function while satisfying the system constraints.

The mathematical statement of the problem is to optimize the objective function for minimizing the total production costs while producing adequate water quantity from limited water sources:

$$\text{Minimize} \qquad F(x) = \sum_{i=1}^{m} Ci\ Xi$$

Subject to the following constraints :

$$\sum_{i=1}^{m} Aij\ Xi \ >= \ Bj$$

$$\text{where} \quad Xi >= 0.0 \qquad i=1,2,...m,$$

$$Bj >= 0.0 \qquad j=1,2,...n$$

Xi = Decision variables which represent water quantity from different sources.

Bj = Constants that represent the system's constraints.

The decision variables include total water quantity to be produced by the available sources, such as desalination plants, local ground water and treated waste water. The associated constraints of the developed water system are of two types one is the exogenous constraints that represent the minimum water demands the system must satisfy, and the other is the upper bound constraints that represent the maximum production capacity of each water source.

Exogenous constraints of the system:

The system objective is to satisfy two different qualities of water demands; namely the high quality municipal demand and the low quality agricultural demand . The use of the terms high and low water qualities to identify the two types of water demands basically depends on the acceptable total dissolved solids(TDS), but in any conditions the water should be free of all toxic ,hazardous and radioactive substances.

The future municipal water demand is estimated to reach 300 million gallons per day (mgd) by the year 2010 according to the ministry of electricity and water (1989).

The minimum water requirements for agricultural and outdoor uses are based on the actual cultivable lands in the country. The total area of unused cultivable land is 132 square kilometers (sq.km) due to the lack of water.

However large areas in Kuwait are suitable for agricultural purposes especially for trees and pasture utilization if we can supply a minimum water quantity of about 350 mgd from the low quality water sources.

Upper Bound Constraints Of The System:

The system maximum production capacity is controlled by the maximum yield of each water source and the costs according to the following constraints:

A- Sea water conversion processes (desalination):

There is no upper limit on the quantity of water produced by desalination as long as the costs are affordable. Kuwait has a long ,calm ,shallow, and unpolluted coast line that makes the process is one of the most appropriate methods for fresh water production. Accordingly, the maximum desalination production capacity will be controlled by the cost while the minimum production capacity is controlled by the already 200 mgd constructed desalination plants .

B- Local ground water maximum yields:

The maximum ground water yield is based on the total available water quantity in the main aquifers , Kuwait group and Dammam formation. These aquifers are hydraulically connected as the main directions of flow in them resemble each other under the conditions of dynamic equilibrium.

The saturated thickness of the Kuwait group is gradually increasing with flow direction from 200 feet at west and southwest to 500 feet at central and east. Also the underlying Dammam formation saturation thickness increases from 400 feet at west and southwest to 1000 feet at east and northeast. (Sara, Ahmed, and Yidiri,1984).

As a result of the natural conditions of these aquifers the water quantity to be produced will increase as the aquifer's depths increase reaching a maximum total water quantity of 350 mgd distributed among six different designed salinity zones as follow :

<u>Zone I :</u> The wells in this zone are assumed to produce a maximum daily water quantity of 20 mgd.

<u>Zone II :</u> In this zone 40 mgd is assumed as maximum.

<u>Zone III :</u> The maximum production of this zone is 60 mgd.

<u>Zone IV :</u> The maximum production of this zone is 70 mgd.

<u>Zone V :</u> The maximum production of this zone is 80 mgd.

<u>Zone VI :</u> Large water quantity is available in this zone, but it has high concentration of dissolved solids, hence we will limit the maximum production capacity to 90 mgd.

C- Reuse of treated municipal waste water:

The maximum amount of waste water that can be reused as a water source depends on the municipal water withdrawals ,the use to which the water is put, and the treatment costs.

In fact the amount of water available for reuse is a function of the municipal water demand that makes the maximum reusable water quantity to be 50 % to 70 % of total demands a good estimation.

MODEL:

1]min =1000*$(C_1*X_1 +C_2*X_2 +C_3*X_3 +C_4*X_4 +C_5*X_5 +C_6*X_6$
3] $+C_7*X_7+C_8*X_8+C_9*X_9+C_{10}*X_{10}+C_{11}*X_{11}+C_{12}*X_{12}$
4] $+C_{13}*X_{13}+C_{14}*X_{14}+C_{15}*X_{15})$;
5]! subject to :
6];$X_1+X_2+X_3+X_5+X_7+X_9+X_{11}+X_{13}>300$;mgd,municipal demand.
7];$X_4+X_6+X_8+X_{10}+X_{12}+X_{14}+X_{15}>350$;mgd,Agricultural & outdoor demands.
8]$X_1 >200$; mgd, Desalination.
9]$X_2 <10$; mgd, Fresh groundwater.
10]$X_3 +X_4 <20$; mgd, Zone I groundwater.
11]$X_5 +X_6 <40$; mgd, Zone II groundwater.
12]$X_7+X_8 <60$; mgd, Zone III groundwater.
13]$X_9+X_{10} <70$; mgd, Zone IV groundwater.
14]$X_{11}+X_{12} <80$; mgd, Zone V groundwater.
15]$X_{13}+X_{14} <90$; mgd, Zone VI groundwater.'
16]$X_{15} <100$; mgd, Recycled municipal waste water.
END

$C_1 = 3.5$; $C_2= 2.0$; $C_3 =1.15$; $C_4 = 0.65$; $C_5 =1.25$;
$C_6 = 0.55$; $C_7= 1.3$; $C_8 =0.4$; $C_9 = 1.45$; $C_{10}=0.45$;
$C_{11} = 1.55$; $C_{12}=0.35$;$C_{13}=1.8$;$C_{14}= 0.85$; $C_{15}= 1.85$;

Model solution and conclusion:

A linear programming software package called LINDO is used to generate the solution for this model. For an optimized set of unit cost coefficients the minimum value of the objective function is $ 1 128 000.00

which gives the required demands according the following shares:

A. Municipal water demand;

$X_1 =$ 200 mgd - desalination
$X_2 =$ 10 mgd - fresh groundwater
$X_3 =$ 20 mgd - Zone I groundwater
$X_5 =$ 40 mgd - Zone II groundwater

B. Agricultural and outdoor demands;

$X_8 =$ 30 mgd - Zone III groundwater
$X_{10}=$ 70 mgd - Zone IV ground water
$X_{12}=$ 80 mgd - Zone V groundwater
$X_{14}=$ 90 mgd - Zone VI groundwater
$X_{15}=$ 80 mgd Recycled municipal wastewater

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An Overview of The Essence of Modeling Water Demands

Dr. Ahmed R. Khater

AN OVERVIEW OF THE ESSENCE OF MODELLING WATER DEMANDS

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Abstract

In areas short of water such the Gulf region , planning decisions with careful attention being paid to forecasting of future water demands are needed. The choice of an appropriate approach to water demand modelling plays a vital role in making such decisions. In view of this , the present paper disscusses the framework of alternative modelling approaches. The discussion comprises the methodological framework for three broad approaches , namely , historical extrapolation , statistical techniques and mathematical programming. Aiming at guiding the choice between modelling , the paper examines how these approaches can be applied with respect to the various water use activities.

Key Words : Water demand models , forecasting water demands , Demand projections

Introduction

With the fast growing population and the expansion of development activities exerting pressure on available water resources , management of water resources is becoming an increasingly serious concern throughout the world. Until early in the twentieth century, much attention was given to the supply side of water resources. Demand considerations and efficiency in use generally played a secondary role. The galloping rise in demand associated with the rapidly changing patterns of water use indicate that the availability of water can no longer be taken for granted and water use in the immediate future will be governed by increasing scarcity in various parts of the world.

Water management, specially in countries short of water, calls for planning with consideration being given to the use of non-conventional sources of supply. As a result , there will be a substantial increase in water costs , and the problem of water can no longer be seen simply as development of new sources. Accordingly , levels of demand should be affected , and anticipation of water problems and identification of control factors will depend on the forecasts of demand for water in the future.

In the economic sense , the term demand denotes willingness of consumers or users to purchase goods , services or inputs to production processes , in this case water or water-related products and services , since that willingness varies with the price of the commodity being purchased. The functional ralationship between various prices for water and the quantities that would be used at those prices essentially conforms to the economic notion of a negative relationship between price and quantity demanded. Obviously , water at certain levels of use is a requirement , and in that sense water uses within those levels are unresponsive to price. This implies that water is a unique commodity and should not be regarded as a perfect economic good. But the true requirement levels are usually only a small part of observed water use , and therefore questions relevant to demand are always the main concern. However , to act towards all existing and future uses with the assumption that demands are requirements means to deny possible demand regulations that can be achieved as the price of water increases.

Analysis of water demands in relation to planning decision in water resources management is a trans-disciplinary task that involves persons with diverse professional background , such as hydrologist , water resouurces planners , experts in the technology of water use, economists , and system analysts. On the other hand, the procedures that are used for forecasting water demands may vary from relatively simple historical extrapolation to analytical models at various levels of sophistication , this will depend upon the quality and quantity of data and the purpose of the forecast.

This paper addresses the growing interest in the examination of water demands and the need to pay careful attention to modelling and forecast of demand for water in the future. The paper discussesthe framework of the alternative approaches to water demand modelling and presents a review on how these approaches can be applied to the various water use activities , aiming at guiding the choice between modelling approaches.

Modelling Approaches

The essence of demand modelling is to decribe demand relations in a mathematical form. Therefore , demand models provide simplification , or abstraction , of complex physical reality and the processes in it and serve as tools in the solution of demand forecasting problem. Three broad approaches to water demand modelling are discussed in this section.

Historical Extrapolation

In the historical , or conventional , extrapolation future water demand for each category of water use is estimated over a specified time horizon based on an observed average consumption per capita and per unit production. The total demand is then calculated by multiplying the projection of population and production by the estimated average consumption rates.

This historical approach is relatively simple but it ignores the relationship between investments, costs, prices and the quantity of demand. Modelling demand according to this approach implicitly assumes that supply is not a rigid constraint , and that the technical , economic and behavioural characteristics of the water use categories are stable. A criticism of such mode ling procedure is that they yield a baseline forecasting of water use under the assumption that all demands are requirements and no conscious attempt will be made to affect that use , an assumption which is demonstratably incorrect.

However , it is possible to improve the outcome of historical extrapolation if the estimates of expected requirements are revised in the light of anticipated costs. This can be made by comparing in space and time the projected requirements with the available supplies defined as quantities related to costs (UN,1976).

Statistical Techniques

In the statistical modelling of water demand relations , a water activity is conceptualized as a black box with input and output variables and their associated costs or prices. In the black box representation , inputs and outputs are known as explanatory variables and dependent variables, respectively. Among the explanatory variables one should distinguish the so-called exogenous variables that have an effects on the dependent variables but which are not explained by the model. These include variables such as administered prices , enviornmental standards , governmental subsidies and amounts of precipitation (Kindler and Russell , 1984).

A statishical model of a water demand relation can generally be expressed as

$$D = f\ (X_1 , X_2 , ... , X_n) + E$$

where $f\ (\quad)$ denotes the function of explanatory variables $X_1 , X_2 ,.... , X_n$, and E is a random error variable describing the effect on D of all factors others than those explicitly considered in the form of explanatory variables.

In practical applications , the analytical form of the function f is commonly assumed to be either additive , multiplicative , or a combination of the two. These possibilities translate into linear , full logarithmic , or semilogarithmic forms :

$$D = a_o + a_1 X_1 + ... + a_n X_n + E$$

or

$$\ln D = b_o + b_1 \ln X_1 + ...+ b_n \ln X_n + E$$

or

$$D = c_o + c_1 \ln X_1 + ... + c_n \ln X_n + E$$

where D is the total or unit amount of water demanded ; $a_o , ... , a_n , b_o , ..., b_n$ and $c_o , ... , c_n$ are the structural parameters of the alternative models , $X_1 , ... , X_n$ are explanatory variables , and E is the random error term. These forms are convenient because they allow for easy estimation of model parameters by use of multiple regression analysis (Draper and Smith , 1966).

The identification and selection of appropriate explanatory variables is closely connected with determination of model structure for each dependent variable. Statistically , there are several properties of explanatory variables to be considered in the identification and selection process :

i) high corrlationship with the dependent variables .
ii) low corrlationship with other explanatory variables.
iii) high relative variation (ratio between standard deviation and mean) in the observed values of the variable.
iv) ease of reliable forecasting of the variables.

The random error term E is almost always assumed to have the following properties:
- the expected value of E is zero ;
- the variance of E is a constant ;
- the errors are uncorrelated across observations.

The ordinary least-squares method is the most commonly used technique for estimation of model parameters , provided that explanatory variables are determined independently of the dependent variable , and under the assumptions mentioned about the random error term. In the case of an interdepndent multi-equation model in which some of the dependent variables serves as explanatory variables for other dependent variables , the use of ordinary least-squares leads to biased and inconsistent parameter estimates. In this case , special methods of estimation, such as maximum likelihood or two stage least-squares , must be used (Goldberger , 1964 - Theil , 1978)

In the modelling process , after estimating the model parameters , the next step that should precede using the model is verification and validation of the model. Verification of a statistical model estimated by application of multiple regression analysis involves computation of various meassures of fit goodness (e.g., multiple correlation coefficient , coefficient of determination) , application of several tests of significance (e.g., F- test for the overall significance of the regression equation , t - test for the significance of the individual regression coefficients), and checking of the extent to which some basic assumptions of multiple regression are met (e.g., linearity , independence of residuals , constancy of residuals' variance , normality of residuals) (Johnnston , 1972). In addition , the model should be verified from the qualitative point of view by ascertaining whether the regression coefficients have correct signs in the sense of being in agreement with theory and common sense. The validation of a statistical model entails using a new set of data and testing the values of dependent variables derived by application of the model. However , in the statistical modelling of water demand , all the available data are usually too few for the model building phase , and there is no additional data to perform such testing.

It is evident from the above that using a statistical model to estimate any functional form and to test any hypothesis about the water demand relation requires quite large amounts of data. Regardless of the degree of complexity with which the projecting formula is composed , in the absence of data describing what is curently experienced , estimates of future water demand cannot be made. Such data could only come from repeated observation of the same water use activity over time or simultaneous observations of many activities of the same sort at the same time. On the other hand , the relative importance of data should also be considered , especially of price - quantity data. Extreme care must be exercised in the interpretation of statistical information , quantities and corresponding unit cost in particular , (if users are not charged according to the quantity of water they use , or average cost pricing is used , conclusions about price responsiveness of water use cannot be drawn).

Mathematical Programming

Mathematical programming techniques are concerned with establishing the best or optimal solutions to decision making problems. Thus, it involves the use of optimization methods such as linear , integer , dynamic and multiobjective programming. Modelling water demands using mathematical programming essentially requires knowledge of the engineering aspects of a water use activity and the processes in it. Such knowledge is needed to formulate alternative activity designs which in turn can be used to define water demand relations for the activity.

Following identification of the technological options available for changing water use and the estimation of costs of making such changes , the next step is to determine whether the saving in costs of water justify the increased costs associated with making the change. The goal in most cases is to find the optimal response of the activity to a change in the cost of water , a process which involves an exhuastive search among a wide range of possible combinations of alternative configurations. In this context , application of mathematical programming supplements an engineering design procedure with a systematic search method for determinig the optimal solution and the associated water demand (Hillier and Lieberman , 1974).

A mathematical programming model of water use activity is a combination of unit processes written in the form of a set of inequality and equality constraints on the system , where the decision variables are the levels of operation of the processe. The objective function for the model represents the criterion for choosing the optimum combination of unit processes , the measure that must be minimized or maximized. In water demand analysis it is most common that the objective function represents cost. In such cases tha problem is stated as follows :

Find **X** which minimizes $f(\mathbf{X})$

subject to the constraints

$$g_i(\mathbf{X}) \leq 0 \quad , i = 1, 2, \ldots\ldots, m$$

and

$$l_i(\mathbf{X}) = 0 \quad , i = 1, 2, \ldots\ldots, p$$

where **X** is an n-dimensional vector called the design vector which denotes the levels of unit processes , $f(\mathbf{X})$ is the objective function to be minimized and $g_i(\mathbf{X})$ and $l_i(\mathbf{X})$ are , respectively , the inquality and the equality constraints. The number of variables n and the number of constraints m and / or p not to be related in any way.

A linear programming model is applicable for the solution of problems in which the objective function and the constraints appear as linear function of the decision variables. Integer programming is used when the variables are restricted to nonnegative integer values. Dynamic programming technique is well suited for the optimization of multistage decision problems when decisions have to be made sequentially at different points in time and space and at different levels.

The choice of objective function expressed in terms of decision variables is governed by the nature of the problem , being the criterion with respect to which the solution to the problem is optimized. In some situations , there may be more than one criterion to be satisfied simultaneously. An optimization problem involving multiple objective functions is known as multiobjective programming problem. With multiple objectives there arises possibility of conflict and one simple way to handle the problem is to take the actual objective function as a linear combination of the multiple objective functions. Thus , if $f_1(\mathbf{X})$ and $f_2(\mathbf{X})$ are the two objective functions possiple , the new objective function to construct for optimization can be expressed as

$$f(\mathbf{X}) = a_1 f_1(\mathbf{X}) + a_2 f_2(\mathbf{X})$$

where a_1 and a_2 are some constants whose values indicate the importance of one function relative to the other.

There are no standarad procedure to be followed in the verification and validation of a mathematical programming model , and in fact , the distinction between verification and validation is no longer a sharp one as in the statistical modelling approach. However , this can be achieved by checking results of experimental model runs for their completeness and internal consistency. The model can also be applied to particular activities of the type modelled , and predicted water demands can be compared with the actual ones.

Choice of Modelling Approach

The choice between modelling approaches depends on such factors as data availability , skills and interests of the modelling team , and access to computational facilities. But this choice is also linked to the intended application of the model to be constructed. One way of summarizing the links between application and model type is to look at combinations of two principal characteristics of specific application : the level of analysis , and the problem to be addressed. However , the variety of situations under which demand analysis is required is so large that there is simply no way to provide a general recommendation of the best way to proceed (Kindler and Russell , 1984).

The following discussion on the application of demand models for the various water use activities provides guidelines for the choice of modelling approach.

Agricultural Water Demand

Models used in agricultural water demand analysis vary in complexity and methodology. In the conventional approach to estimation of agricultural water demands , the area to be developed for irrigation is specified and multiplied by a coefficient reflecting the volume of water required per unit area , thus givin an estimate of the water requirements. Such coefficients may be found from past experience. Though the water use coefficient approach has a certain value , it is often found inadequate to answer questions related to water resources planning. Statistical models are employed when analysing measured historical data. However , statistical methods have not been widely used for modelling water demand relationships in agriculture. For example , Bain *et al.* (1966) used regression analysis to relate water withdrawals for irrigation to prices charged in various irrigation districts of northern California. However , the statistical approach does not allow costs and prices to vary beyond their values recorded in the past.

Mathematical programming is a planning tool that is well suited for the analysis of water demand in the agricultural activities. The mathematical programming techniques are able to answer questions concerning economic demand for water , and to find the best solution in a set of feasible solutions. Linear programming is the most widely used technique for modelling agricultural activities (Miller,1966). Other mathematical programming methods that take into account some of the nonlinear , dynamic , stochastic aspects of irrigation systems , which are difficult to deal with in linear programming , have also been applied (Dudley *et al* , 1971).

Municipal Water Demands

A criticism of the historical forecasting procedures is that they are trying to solve a complex problem with simple solution procedure which ignores many factors that could affect the future demand. Statistical techniques are very helpful in identifying factors to account for variations in municipal water use. The most commonly used statistical technique has been regression analysis , in which cross sectional data are regressed on water use. Other techniques involve the use of time series data or pooled time series and cross sectional data.

Statistical models of aggregated municipal water demands have been most popular (Hanke , 1978). Residential water demands have also been the subject of considrable statistical modelling. In these modelling studies per capita water use has been correlated with income , household size , weather conditions and irrigable area (e.g., Morgan 1973 , Danielson 1979 , Linaweaver *et al.* 1966).

Mathematical programming techniques are not often used in modelling municipal water demand. Though by giving detailed consideation to costs of alternative combinations , mathematical programming models can yield useful indirect estimates of water demand. A specific application of this approach is the analysis of water conservation in residences , and assessment of behavioural responses (Howe , 1970). However , the statistical approach appears to be most promising for the residential water use category.

Industrial Water Demands

Because the purposes to which the water is used in industrial processes vary widely , and there are different uses of water within an industrial plant , the development of specific and accurate relationships explaining water use patterns is difficult. when the analysis is concerned with individual industrial plants , mathematical programming seems better suited. This is partly because the data problems of the statistical approach result in a rather crude avrage representation of the activity in question. When the problem at hand is of analysis above that of the individual activity , the model of choice is usually the statistical one.

Concluding Remarks

The growing rise in water demands , specially in areas short of water such as the Gulf region , calls for planning decisions with careful attention being paid to forecasting of future demands. The choice of an appropriate modelling approach plays a vital role in making such planning decisions.

The systematic review of projection methodologies reveals that the commonly applied approach of historical extrapolation deals with demands as requirements and ignores the relationship between the quantity of demand and the economic factors. Since water resources management can no longer afford the assumption that supply is not a rigid constraint , and water demand projections are essentially related to projections of the form and size of the economy , water should be viewed as an economic good. Therefore , the adequqcy of modelling water demands on basis of previous annual increment in water consumption has come into question.

The process of projecting water demands should be directed towards analytical modelling approaches such as statistical techniques and mathematical programming. The superiority of analytical models over extrapolation methods lies not in greater accuracy but in their capability of including economic factors and assessing the consequences of various policy options. One advantage of the mathematical programming approach over the statistical one is that the costs and prices may be allowed to vary beyond their values recorded in the past and the resulting predictions of the demand may be accepted with reasonable confidence. Another advantage of the mathematical programming is that new technologies can be introduced into the analysis explicity.

Because of the diversity of local conditions under which demand analysis is required , general rules or standard solutions can be of only limited value in working out details of modellng procedure. However , based on the nature of water use activities , mathematical programming as a planning tool seems well suited for the analysis of a wide range of demand forecasting problems in the agricultural and industrial activities. On the other hand , the statistical approach appears to be most promising for modelling municipal water demands .

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The Impacts of Present Groundwater Management On Its Future Conditions in Desertic Aquifers

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THE IMPACTS OF PRESENT GROUNDWATER MANAGEMENT ON ITS FUTURE CONDITIONS IN DESERTIC AQUIFERS

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ABSTRACT:

Little Attention has been given to the management issues of groundwater in desertic aquifers, especially the proper design of well fields in terms of spatial distribution between wells and well fields; and pumping policies. Large number of wells have been drilled in small areas; and groundwater pumping has been increased drastically from these areas during the last two decades in some parts of the Kingdom. This improper management of groundwater has resulted in unacceptable impacts on the water levels and quality in these areas. Numerical techniques have been used to assess the impacts of this practice in some local areas in the Eastern Province. If the existing water management policies and trend of increase in pumping continues, a considerable additional drawdown will occur in the aquifers in these local areas. Heavy pumping from large number of clustered wells in closely spaced well fields resulted in these detrimental impacts on groundwater. Therefore, a proper groundwater management and conservation schemes are required to be adopted in these areas to improve the groundwater conditions. These include optimization of well field design and well operation, and the implementation of effective irrigation scheduling techniques. The implementation of a comprehensive irrigation water management system in a large irrigation project has resulted in significant recovery of groundwater in the project area in the Eastern Province

INTRODUCTION

Most of Saudi Arabia is located in extremely arid regions, where the average annual rainfall ranges from 25 to 150mm. During the past two decades, the Kingdom has experienced extensive developments in agriculture, municipal and industrial sectors. Groundwater is the major water supply source for different uses in the Kingdom. Therefore, careful management of the groundwater resources has become essential for the country to maintain the long term quality and productivity of aquifer system in the country..

In the Eastern Province of Saudi Arabia, groundwater from the multi-aquifer system, which includes the Alat, the Khobar and Umm Er Radhuma aquifers is the main water source for irrigation, domestic, and industrial purposes. The groundwater from the Umm Er Radhuma (UER) aquifer is the main source of water for domestic and landscape irrigation purposes in residential areas such as Dhahran and Al-Khobar. Several agricultural areas in the Eastern Province utilize groundwater from UER aquifer. Khobar and Alat aquifers are being used for domestic and industrial purposes in major cities of the Eastern Province, such as, Al-Khobar, Dammam, townships along the coastal belt of the Eastern Province. The Neogene aquifer has been utilized mainly for irrigation purposes in Al-Hassa area. This aquifer is the main source of irrigation water supply for Al-Hassa irrigation and Drainage project.

This paper describes the impacts of the present groundwater management policies and well field design and operation on groundwater levels and quality in heavily populated and agricultural areas in the Eastern Province of Saudi Arabia (Figure 1); and suggests recommendations to improve the groundwater management.

HYDROGEOLOGICAL SETTING

The results of hydrogeological studies have indicated the existence of multi-aquifer system in Eastern Province. The main aquifer system in Eastern Saudi Arabia (Figure 2) consists of three main aquifers separated by semi-confining beds, in ascending order they are: the Umm Er Radhuma (UER) aquifer, the Rus aquitard with Midra and Saila Shales and Alveolina limestone members, the Khobar aquifer, the Alat Marl aquitard, Alat aquifer and the Neogene aquifer (Italconsult 1969, BRGM 1977, GDC 1980). Vertical flows between the aquifers occurs through the leaky intervening aquitards depending on the hydraulic head gradient in each aquifer (Rasheeduddin 1988, GDC, 1980). Analysis of groundwater budgeting of the multi-aquifer system in the area has indicated that the pumping of groundwater from any aquifer has a direct impact on the groundwater conditions of the other adjacent aquifers (Abderrahman and Rasheeduddin, 1994). The main aquifers are represented by limestones, dolomitic limestones, and dolomites. The aquitards consists of marls, shales, dolomitic, anhydrites chalky limestones etc.

The Um Er Radhuma aquifer crops out at Wadi al Batin in the Northwestern part of the Eastern Saudi Arabia. It constitutes the most important aquifer in the region. It is bounded by the evaporitic complex of the Rus Formation at the top and varicolored Aruma shales at the bottom. Lateral and vertical variations in lithology are evident, which influence the hydraulic characteristics of the aquifer. Field investigations by Italconsult (1969) indicate that the average depth below ground level is about 220 m in the Eastern Province. Previous studies have proved that the UER aquifer is highly productive and considered as one of the major aquifers in Eastern Province .The average transmissivity of UER aquifer is $6.4X\ 10^{-1}$ m^2/s at the Dammam Dome . Storativity is around $1.5X10^{-2}$ at Dammam dome indicating unconfined to semiconfined conditions , and around $5X10^{-4}$, at other parts indicating strictly confining behavior of the aquifer.

The Rus Formation along with the Midra and Saila Shales, and the Alveolina Limestone Member forms an aquitard separating UER Aquifer from Khobar Aquifer. It occurs under the whole study area. This aquitard consists mainly of chalky limestones, anhydrites, marls and shales. Thickness of this aquitard ranges from about 10 to 110m.

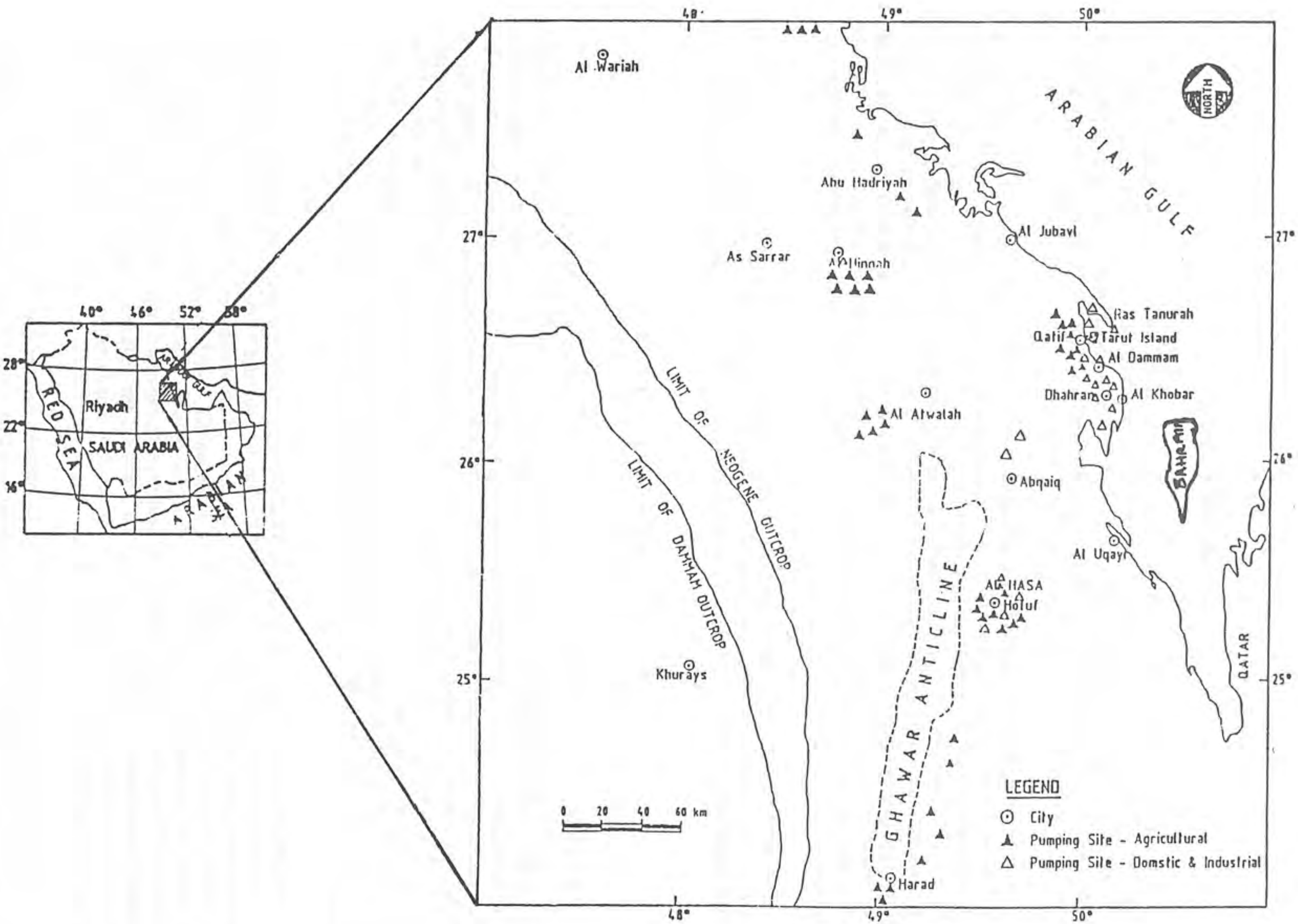

Figure 1. Location map of the study area

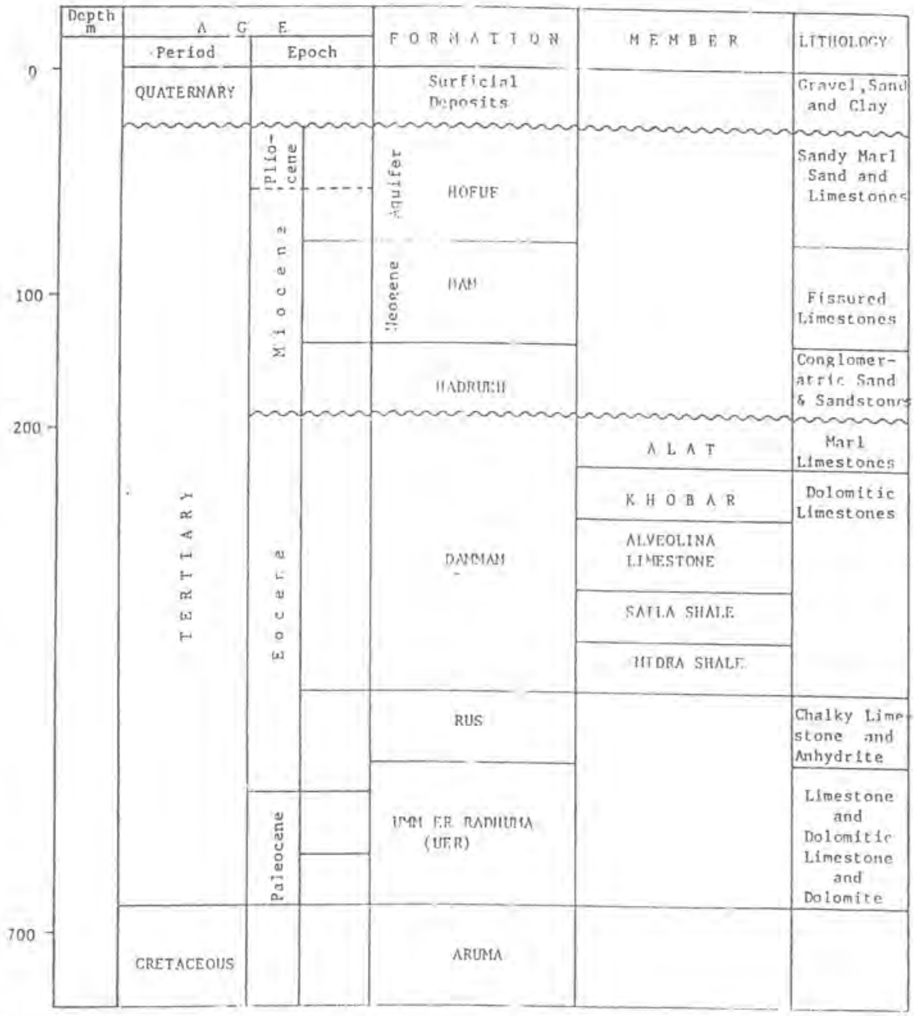

Figure 2. Generalized lithostratigraphic succession of the Eastern Province of Saudi Arabia (modified after BRGM, 1977)

The Khobar member of the Dammam Formation forms the Khobar aquifer. It is partially eroded in the study area especially at the Dammam dome area. The structural trends were similar to that of UER aquifer. It is mainly formed of skeletal-detrital limestones, dolomitic limestones and argillaceous limestones near the bottom. It is bounded at the bottom by Alveolina limestone and at the top by Orange Marl of Alat. The average transmissivity of Khobar aquifer is about $2.9X10^{-3}$ m^2/s. The average storativity is around $2.1X10^{-5}$ indicating the confining behavior of the aquifer.

Lower unit of the Alat member i.e., Alat Marl or Orange Marl forms an aquitard separating the Khobar and Alat aquifers. This unit exists in the study area, but it is not uniformly distributed throughout the study area due to erosion of the Dammam dome during pre-Neogene era. It is mainly formed of marls and shales.

The upper unit of Alat Member forms the Alat aquifer. Alat aquifer does not exist throughout the study area due to the pre-Neogene erosion by which it has been eroded at and around the Dammam dome. It consists mainly of skeletal, detrital and dolomitic limestones. The Alat aquifer behaves as semiconfined to unconfined near the eroded surfaces around the Dammam dome. The average transmissivity of Alat aquifer varies from $3.1X10^{-4}$ m^2/s to $2.3X10^{-3}$ m^2/s. The storativity ranges from $1.5X10^{-4}$ to $2.6X10^{-}$ indicating the confining behavior of the aquifer.

The Neogene complex is of fine to coarse clastics mainly of continental and transitional environment lies unconformably over the Eocene Formations. It is divided into Hadrukh and Dam Formations. The thickness varies from 0 to 200m in eastern province and 100-200m in Al-Hassa area where it is highly productive and provides water with artesian flow (Italconsult, 1969, and BRGM, 1977). The average transmissivity of the Neogene aquifer is about 1.3 X 10^{-2} m^2/s (Al-Mahmoud, 1987). This aquifer exists throughout the region, but not uniformly distributed. It is highly productive in AL-Hassa area.

HISTORY OF WATER EXTRACTION

The Eastern Province of Saudi Arabia depends mainly on groundwater from the Neogene, the Alat, the Khobar, and UER aquifers. The UER aquifer is the main water supply source for Dhahran and Al-Khobar cities in the Eastern Province of Saudi Arabia. The Khobar and the Alat aquifers are being used extensively in the coastal cities of Eastern Province for domestic and irrigation purposes. The Neogene aquifer is extensively used in Al-Hassa area. Large number of wells have been drilled in small areas in the province to meet the fast rising water demands within a short period of time in different aquifers. Previous studies by Italconsult 1969, BRGM 1977, GDC 1980, Abderrahman and Ukayli 1984, Rasheeduddin 1988, and Abderrahman, 1990; have shown the following history of withdrawal rates in various parts of the Eastern Province:

There are 32 major springs and thousands of private wells drilled in the Neogene aquifer, in a small area of about 200 Km^2. The total groundwater withdrawals during 1977 were about 319 million cubic meters (MCM) and increased to 608 MCM during 1990. Water demand is expected to increase to about 652.4 MCM by the end of year 2000. More than 80% of the groundwater is used for irrigation purposes and remaining for domestic and industrial purposes.

The Alat and Khobar aquifers are mainly used in the townships along the coastal belt of the Arabian Gulf, such as Dammam, Qatif, Saihat, Anak, Safwa, Umm Al-Sahik, Ras Tanurah etc. Pumping records of GDC, 1980 (Figure 3) indicates that there is a continuos increase in extraction rates since 1940 and a marked upward trend started after 1965.

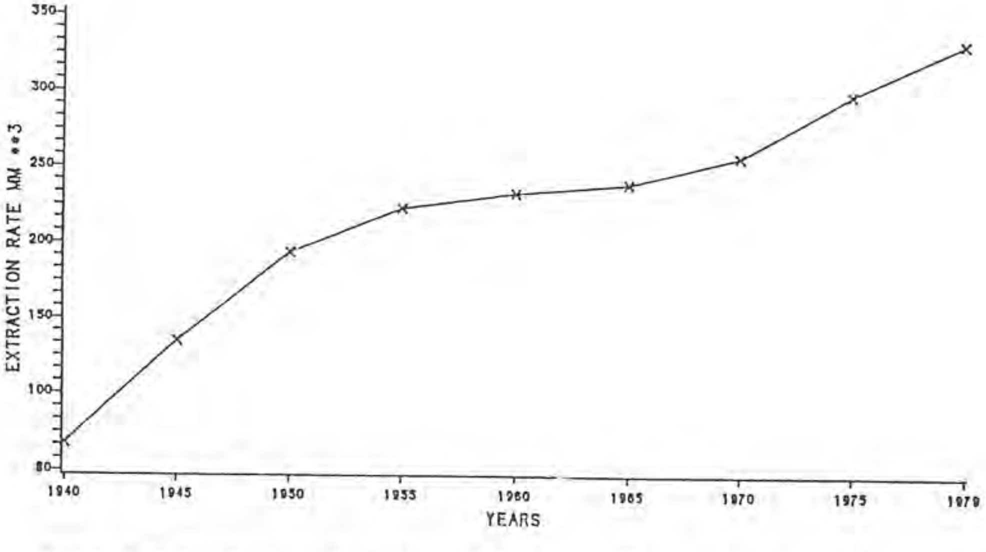

Figure 3. Extraction rates from Alat and Khobar aquifers in Coastal belt (after GDC, 1980)

The available records of pumping history show that the total pumping from Alat aquifer along the coastal belt of Arabian Gulf in the Eastern Province during the years 1967 and 1990 were around 49.3 and 101 million cubic meters (MCM) respectively. Most of the pumped water is used for irrigation and livestock purposes.

Extraction rates from Khobar aquifer along the coastal belt of Arabian Gulf including Dammam, Qatif, Anak, Saihat, Safwa, Ras Tanurah etc., were 122 and 326 MCM during 1967 and 1990, respectively. More than 80% of the pumped water is used for irrigation purposes and rest for domestic, livestock and industrial purposes. Water consumption from Khobar aquifer in Abqaiq was 3.3 MCM in 1967 and increased drastically to 28.8 MCM during 1990. In Al-Hassa about 20 MCM is being pumped from this aquifer for irrigation purposes.

UER aquifer is mostly used in the cities of Dhahran and Al-Khobar for domestic and landscape irrigation purposes. Presently several production wells are operational in the area. The annual groundwater extraction rate has increased from 17.8 to 34.38 MCM between 1967 and 1979 respectively, and to 66 MCM/year in 1990. Figure 4 shows that the extraction rates have increasing trend since 1979 due to major social, industrial and construction developments. Groundwater of UER aquifer in north and north western parts of the Eastern Province is mainly used for large irrigation schemes, such as Ash-Sharqiyah Agricultural Development Company (SHADCO). Several wells were drilled in this area to meet the growing demand of irrigation water. The production rate of the wells in this area range from 7630 to 9812 m^3 /day. The annual water extraction rate for irrigation at SHADCO is about 38 MCM/year.

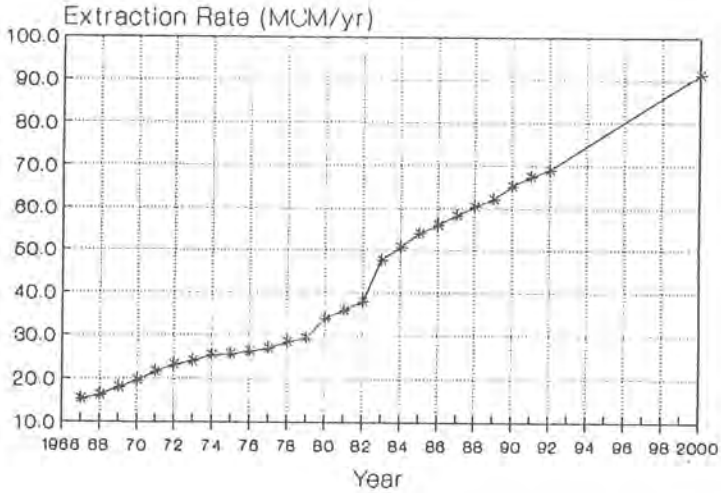

Figure 4. Extraction rates from Umm Er Radhuma aquifer in Dhahran area

IMPACTS OF HIGH PUMPING ON GROUNDWATER CONDITIONS

Based on the available data from various hydrogeological and numerical modeling studies, drawdown contour maps were constructed for different aquifers in Eastern Province (Figures 5,6,7&8).

The Neogene aquifer is being used extensively in Al-Hassa area. The artesian flow from most of the springs ceased to flow by mid 1984 (Abderrahman and Ukayli, 1984). As a result of heavy pumping from clustered wells drilled in small area of about 200 Km^2 in Al-Hassa, the groundwater level in the Neogene aquifer have been dropped significantly at the rate of 10 cm/year. The total drawdown by the end of year 2000 may reach upto more than 3m (Figure 5).

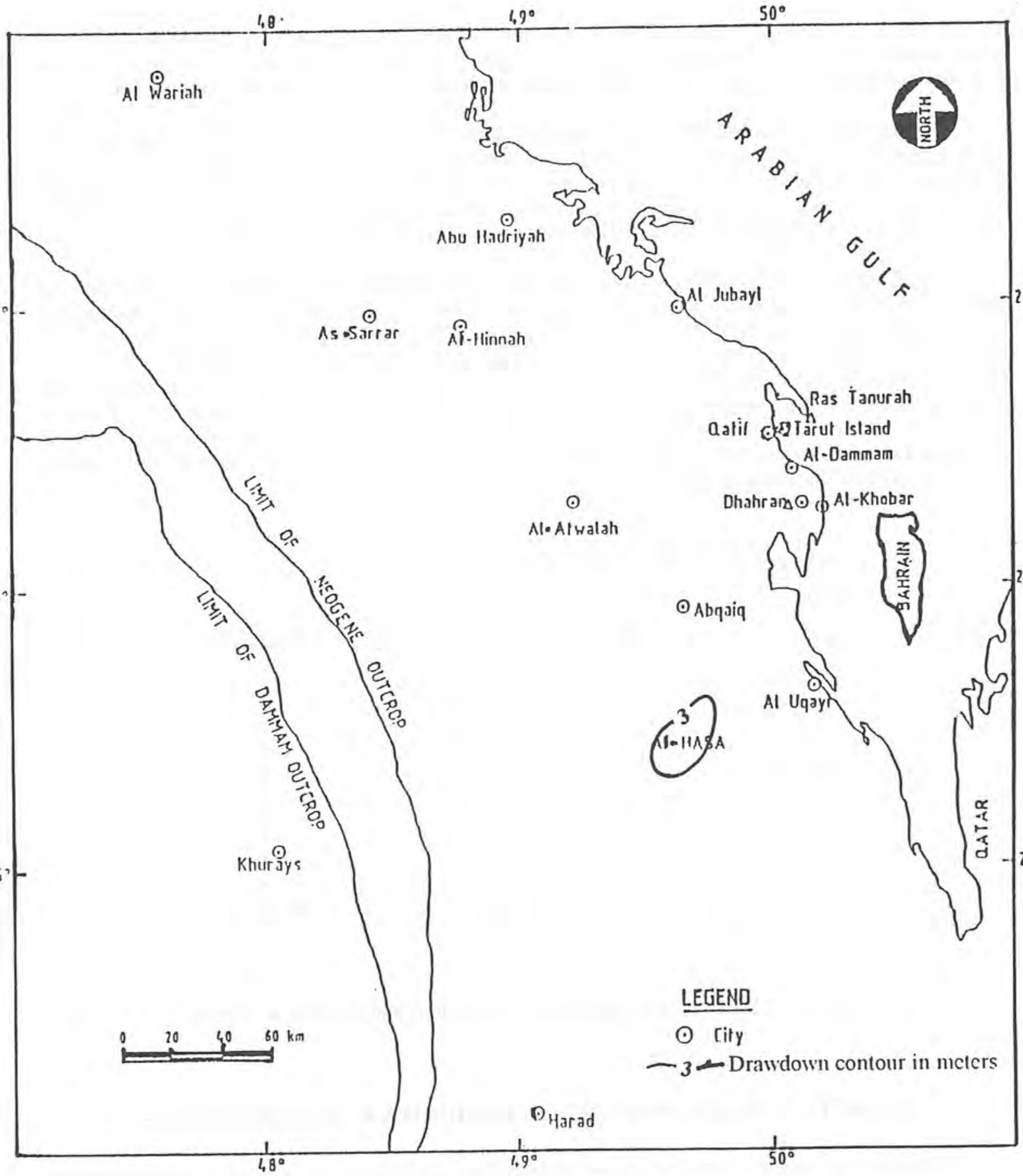

Figure 5. Drawdown contour map of Neogene aquifer for the period of 1987-2000.

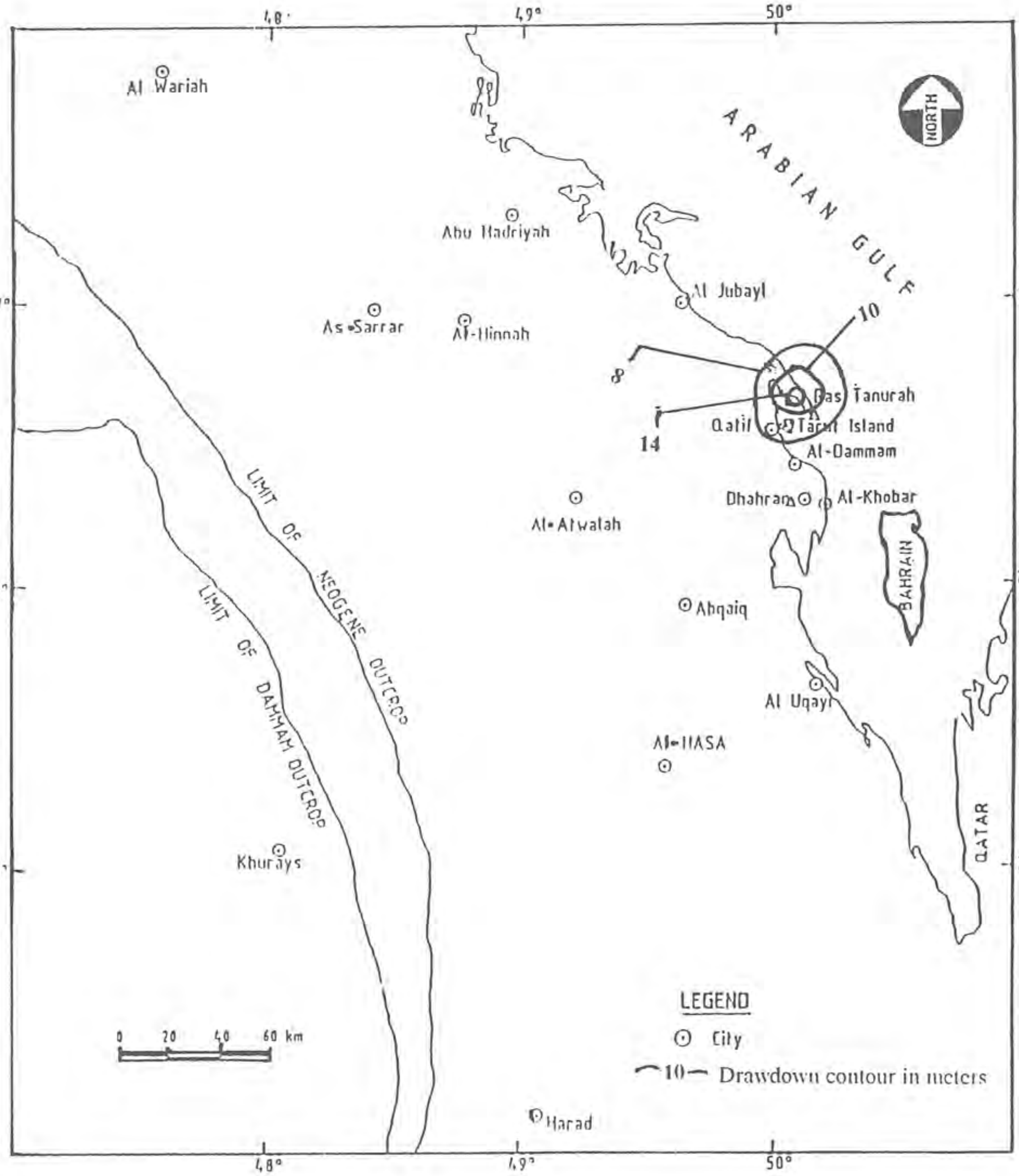

Figure 6. Drawdown contour map of Alat aquifer for the period of 1987-2000.

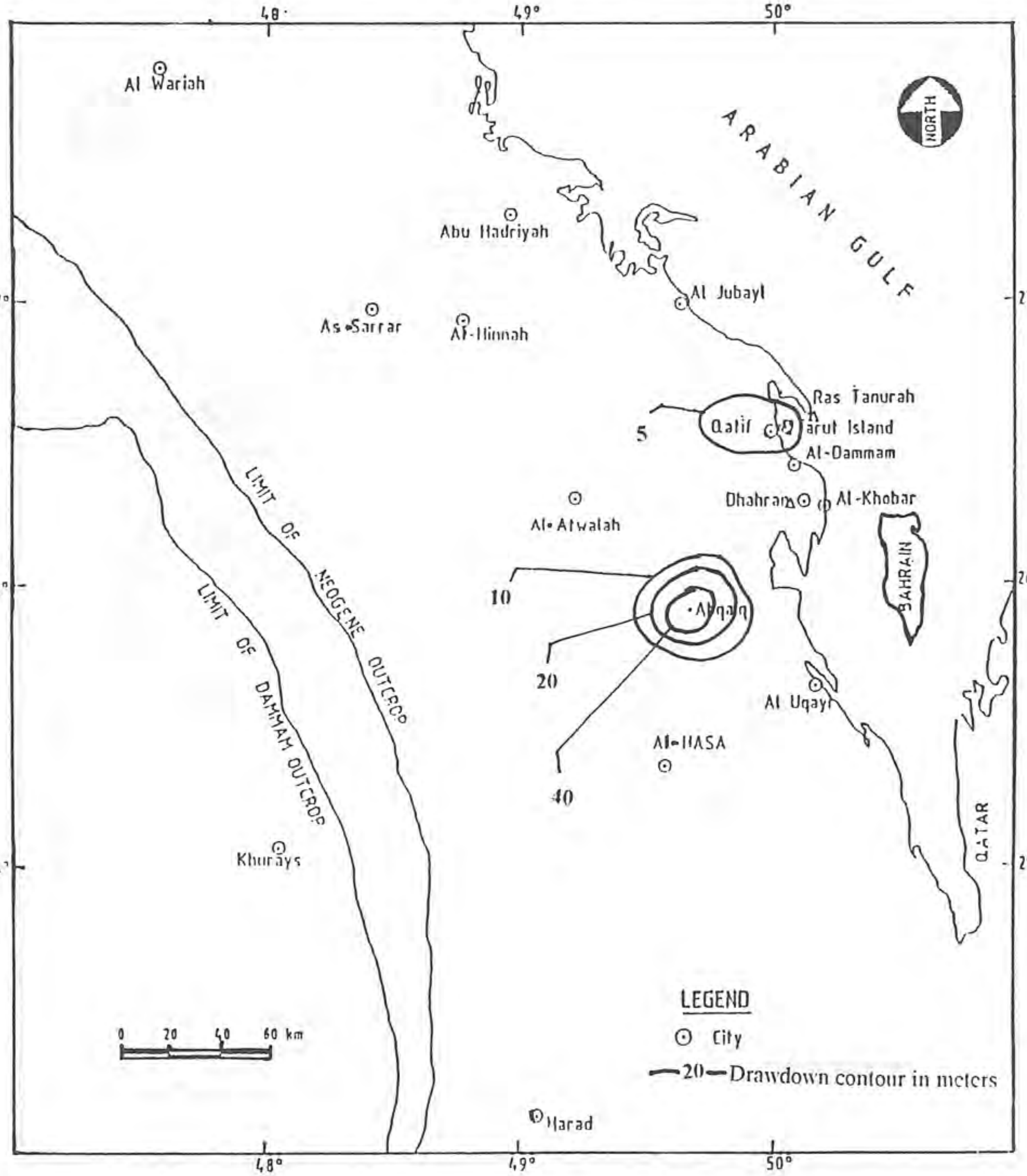

Figure 7. Drawdown contour map of Khobar aquifer for the period of 1987-2000.

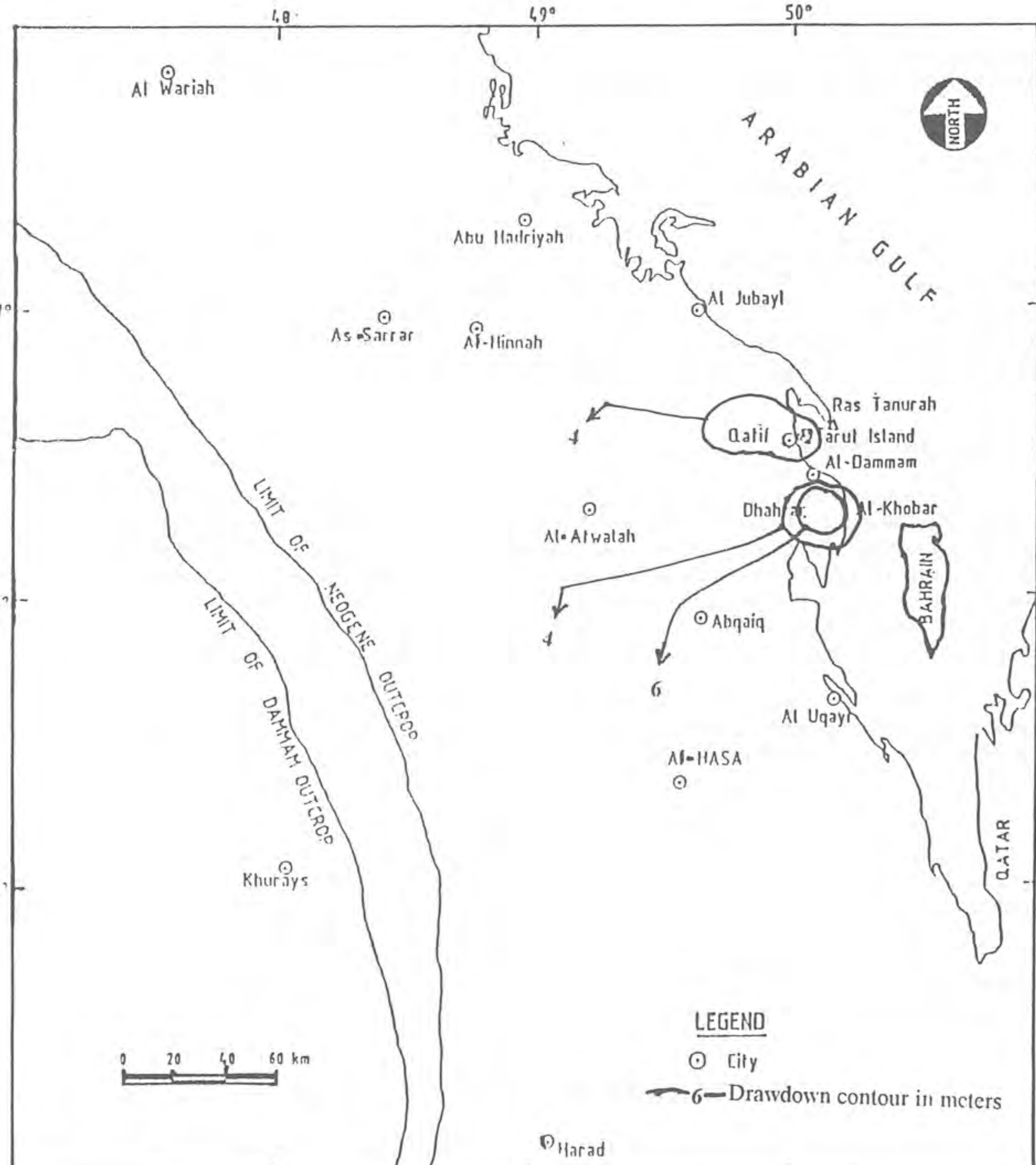

Figure 8. Drawdown contour map of Umm Er Radhuma aquifer for the period of 1987-2000.

In Alat aquifer, the drawdown map show a localized cone of depression at heavy pumping area along the coastal belt extending from Qatif to north of Ras Tanurah of Eastern Province. The expected decline in water levels between 1987 and 2000 is about 14m (Figure.6).

In Khobar aquifer the maximum drawdown of about 40m is expected at Abqaiq by the end of year 2000. This large cone of depression in this area is due to low transmissivity values and limited thickness of the aquifer. Along the coastal belt, around Qatif area the drawdown will be about 5m by the year 2000 (Figure 7).

A groundwater flow simulation model was developed for UER aquifer in Dhahran and Al-Khobar areas using MODFLOW of USGS (McDonald and Harbaugh, 1988). The results of the simulation model have indicated the development of a local cone of depression with a maximum value of about 6.0m in the well field area in Dhahran by the end of year 2000. There is another local cone of depression developed along the coastal belt around Qatif area. The development of this cone is due mainly to the contribution of groundwater from UER aquifer to the overlying Khobar and Alat aquifers by vertical flows (Figure 8).

Groundwater Quality

The average TDS value in Dhahran area as measured by GDC 1979, was about 2750 mg/l. The TDS values in groundwater of wells in UER aquifer were reviewed to assess the long term variations in salinity levels of groundwater in UER. The total annual average of TDS values in the wells was about 3545 mg/l in 1990. This shows that after 1979 the TDS average value has increased by 22% at the end of year 1990. The total increase in water extraction between 1979 and 1990 resulted in increasing the TDS value in the well fields in Dhahran by 22%. If the extraction rates continue in Dhahran area at the same annual increase rate, the TDS values will also continue to rise and may reach about 3922 mg/l by the year 2000 (Abderrahman and Rasheeduddin, 1992).

In Khobar aquifer, water quality records show that the TDS in the areas of Hofuf and Abqaiq varies from 1000 to 2000 mg/l indicating low TDS zones, and between 4000 and 7000 mg/l in the Eastern and north eastern part of the Eastern Province (Hassan, 1992). The groundwater quality in localized zones is expected to change in the future.

The total dissolved solids (TDS) of Alat aquifer in the Eastern Province of Saudi Arabia varies from 2000 to 9000 mg/l (Hassan, 1992). Maximum values of TDS are found along the coastal belt area which includes the cities of Jubayl, Qatif, Anak, Dammam etc. Since the Alat aquifer has been extensively utilized along these local areas, the TDS is expedcted to increase further in future.

The average value of total dissolved solids (TDS) in groundwater of Neogene aquifer in AL-Hassa area increased from 1414 mg/l in 1976 to 1737 in 1987. Sodium, chloride and sulphate are the major dissolved ions in this water (Abderrahman, 1990).

EFFECTS OF PRESENT PUMPING SCHEMES

The present groundwater pumping fields are concentrated in small sized areas either at heavily populated zones or agricultural farms. The wells and well fields are so closely spaced that the cones of depression are localized to small and localized areas. Increasing demands of groundwater are causing considerable drawdowns at the well field itself. Drawdown contour maps (Figures 5,6,7 & 8) show that the pumping areas such as Dhahran-Khobar, Coastal belt, Abqaiq and Al-Hassa areas are the potential sites for change in quality of groundwater. As indicated in the above figures, the groundwater conditions in undeveloped areas in western and northwestern parts of the Province are stable. Groundwater from these areas can be used for future needs.

ALTERNATIVE MEASURES FOR EFFECTIVE MANAGEMENT

To avoid the problems of localized cones of depression and to control the negative impacts on quality of groundwater it is highly recommended that the present clustering of well fields at localized areas should be reviewed; and additional new sites should be selected with sufficient spacing of wells. This should be considered on the basis of detailed hydrogeological investigations and the use of advanced numerical simulation and optimization techniques to minimize the negative effects on groundwater conditions in the Eastern Province.

A second alternative is to maintain the groundwater withdrawal from the existing wells at the present level if not reduced. This can be achieved by implementation of effective water conservation schemes. For domestic use, the in-house water consumption can be decreased by reducing the water pressures in the water supply network, introduction of new water saving devices in kitchens and bathrooms to minimize the water delivery. Landscape irrigation demands can be reduced by effective maintenance of the irrigation system, the application of the efficient irrigation schedule; and the introduction of the draught tolerant plants. Implementation of proper irrigation scheduling technology by all agricultural farms in the region will result in saving of millions of cubic meters of water and help in significant recovery of groundwater levels. Large irrigation projects such as SHADCO has implemented an irrigation scheduling program and saved about 35% of groundwater use for irrigation and water and the groundwater levels have been recovered considerably as shown in water level hydrograph (Figure 9). The measured water level during the months of June and July 1994 is higher than that of June and July 1993 (KFUPM; 1993). Implementation of effective water leakage control measures in the main water supply network will result in saving considerable water quantities. The use of the treated sewage effluent for municipal landscape irrigation in remote locations will result in reducing the landscaping water demand. The implementation of above measures is expected to reduce the water demands and groundwater withdrawal. This will result in maintaining the long-term productivity and quality of groundwater in the area.

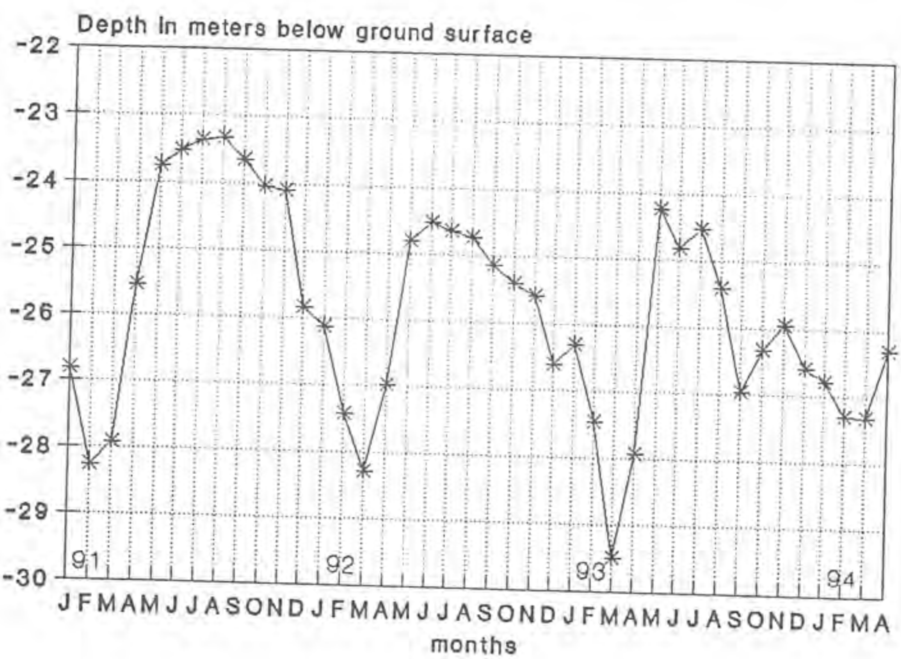

Figure 9. Hydrograh showing variations in water level at an observation wel

CONCLUSIONS

The impacts of present groundwater pumping fields and policies on local water level and quality show the need to improve the water management and conservation in the eastern province of Saudi Arabia. This study shows clearly that the drastic increase in groundwater withdrawal from the UER, the Khobar and the Alat aquifers has some negative impacts on water levels and quality in several small local areas where large number of wells have been concentrated. These impacts will persist in the future, if groundwater withdrawal continues to increase from the existing well fields. Optimal spacing between wells and well fields using advanced numerical simulation and optimization techniques will help in minimizing the negative impacts of water level and quality. The implementation of effective water management and conservation measures on local and regional levels will help in reducing the water demands and groundwater pumping as observed in SHADCO project.

ACKNOWLEDGMENT

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Computer Program for The Method of Fragments

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COMPUTER PROGRAM FOR THE METHOD OF FRAGMENTS

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ABSTRACT

Flow nets are conventionally used in the analysis of seepage problems. However, this method requires substantial effort in producing good quality flow nets. The method of fragments provides quick but approximate solutions, without drawing flow nets, with accuracy sufficient for all practical purposes. This paper presents a computer program that has been developed for the method of fragments. From the input parameters, including the geometry and the soil characteristics, the program is able to compute the critical gradient, exit gradient, quantity of discharge, uplift force, and safety factor with respect to piping.

KEY WORDS: seepage, fragments, flow net, computer program.

INTRODUCTION

Traditionally, seepage problems are solved by flow nets. The quantity of flow, exit gradient and the uplift force can be computed directly from the flow net. The major disadvantage of this conventional flow net method is the substantial effort required in producing good quantity flow net. In preliminary design stages, where several alternatives are tried, drawing separate flow nets for all configurations becomes a tedious task. Under such circumstances, it is desirable to have some methodology to estimate the necessary values quickly with sufficient accuracy. The method of fragments is a very simple and powerful technique developed by Pavlovsky (1956) in Russia and extended and brought to the attention of the world by Harr (1962, 1977). It provides quick and approximate solutions where the accuracy is quite sufficient for practical purposes. The basic assumption in this method is that the equipotential lines at selected critical points (Fig. 1) in a flow net are vertical and that they divide the flow region into fragments. The flow region in Fig. 1 is divided into four fragments.

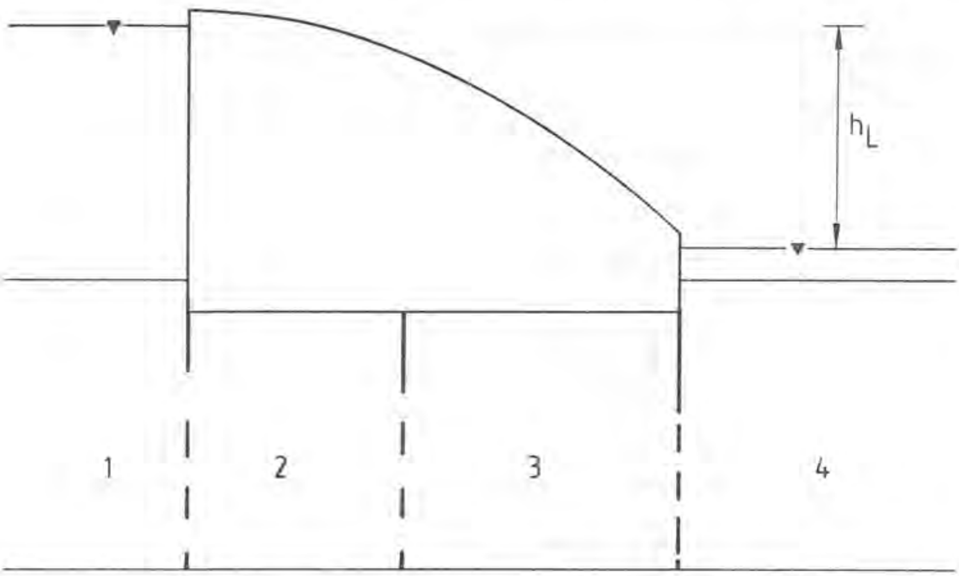


Fig. 1 Fragments in the flow region

For each fragment, a dimensionless quantity known as form factor (Φ_m) is introduced. Φ_m depends only on the geometry and the entrance and exit conditions of the particular fragment, and is defined as:

$$\Phi_m = \frac{\text{No. of equipotential lines in the } m^{th} \text{ fragment}}{\text{No. of flow lines}} \tag{1}$$

The quantity of seepage is given by:

$$Q = k\ h_L/\Sigma\Phi \tag{2}$$

where k and h_L are the coefficient of permeability and head loss respectively. $\Sigma\Phi$ is given by

$$\Sigma\Phi = \Phi_1 + \Phi_2 + \cdots + \Phi_n \tag{3}$$

Since the flow is equal in each fragment,

$$Q/k = h_m/\Phi_m \quad \text{for } m = 1, 2, \ldots, n \tag{4}$$

The fragment types and form factors, presented by Harr (1977), are shown in Figs. 2 and 3. From Eq. 4, the pressure head can be estimated at any point in the flow region. The maximum exit gradient for type II, given by Harr (1977), are presented in Fig. 4. Sivakugan and Al-Aghbari (1993)a used the method of fragments to solve several different types of confined flow problems and showed that the error is generally within 5% for predicting quantity of flow, exit gradient and the uplift force. Further, Sivakugan and Al-Aghbari (1993)b proposed to condense the six types of fragments into just two, referred to as Type A and Type B fragments, as shown in Fig. 5. Type A fragments are common in upstream and downstream, whereas, all the middle fragments are of Type B. In Type B fragments, the flow at the entrance and exit are both horizontal. In Type A, one is horizontal and the other is vertical.

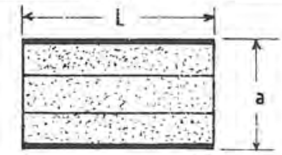
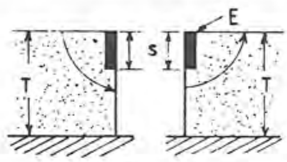
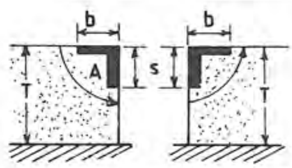
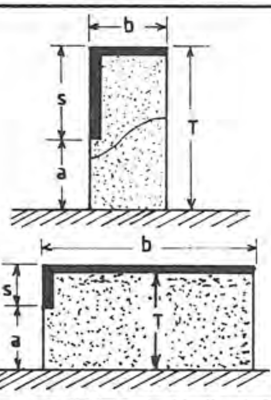
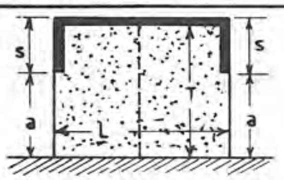
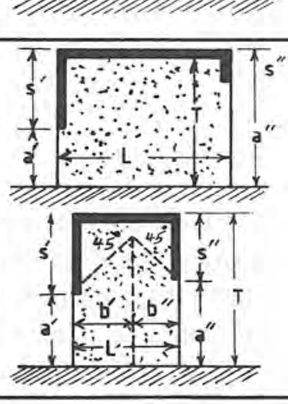
Fragment Type	Illustration	Form Factor, ϕ (h is head loss through fragment)
I		$\phi = \frac{L}{a}$
II		see Fig. 3.
III		see Fig. 3.
IV		$b \leqslant s$: $\phi = \ln(1 + b/a)$ $b \geqslant s$: $\phi = \ln(1+s/a) + (b-s)/T$
V		$L \leqslant 2s$: $\phi = 2\ln(1 + L/2a)$ $L \geqslant 2s$ $\phi = 2\ln(1+s/a) + (L-2s)/T$
VI		$L \geqslant s' + s''$: $\phi = \ln[(1+s'/a')(1+s''/a'')] + (L-s'-s'')/T$ $L \leqslant s' + s''$: $\phi = \ln[(1+b'/a')(1+b''/a'')]$ Where $b' = (L+s'-s'')/2$ $b'' = (L-s'+s'')/2$

Fig. 2 Types of fragments and form factors (after Harr, 1977)

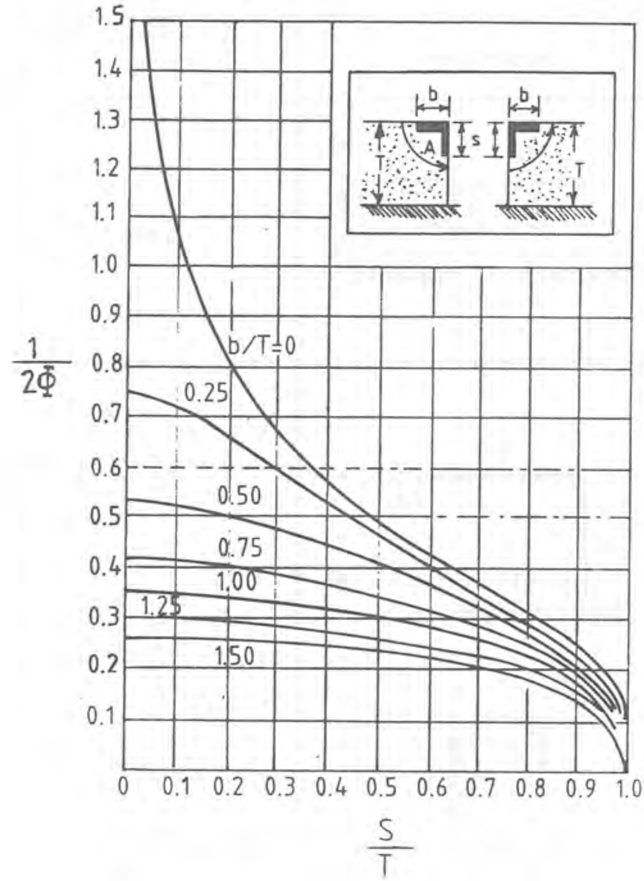


Fig. 3 Form factors for types II and III fragments (after Harr, 1977)

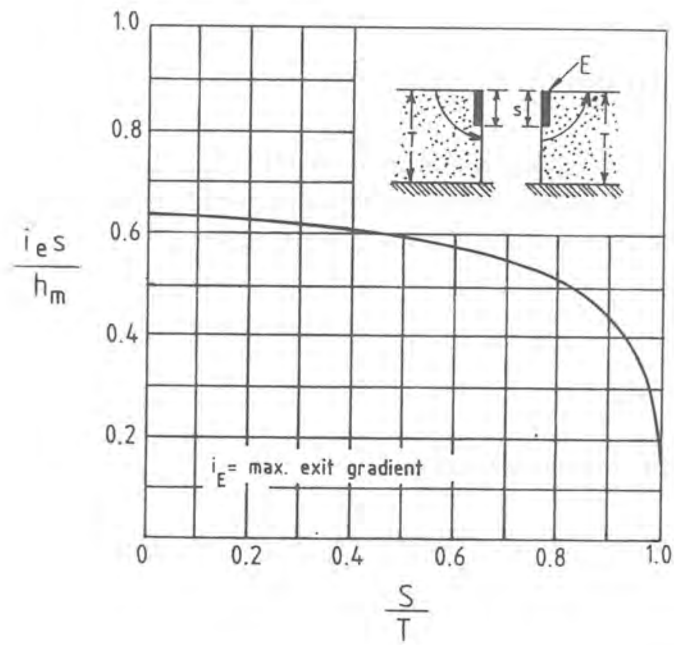


Fig. 4 Exit gradient for type II fragments (after Harr, 1977)

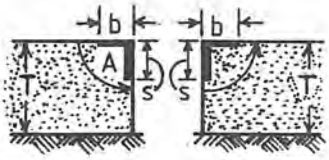
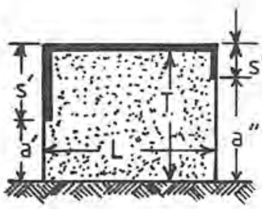
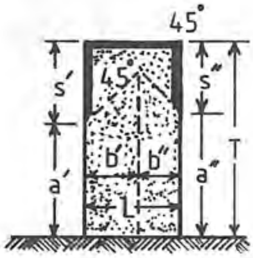
Fragment Type	Illustration	Form Factor. Ø (h is head loss through fragment)
A		See Fig. 3
B	 	$L > s' + s''$: $\phi = \ln\left[\left(1+\frac{s'}{a'}\right)\left(1+\frac{s''}{a''}\right)\right] + \frac{L-(s'+s'')}{T}$ $L < s' + s''$: $\phi = \ln\left[\left(1+\frac{b'}{a'}\right)\left(1+\frac{b''}{a''}\right)\right]$ where $b' = \frac{L+(s'-s'')}{2}$ $b'' = \frac{L-(s'-s'')}{2}$

Fig. 5 Proposed type A and type B fragments

THE PROGRAM CAPABILITIES

A user friendly and interactive program written in QBASIC for the modified method of fragments (Fig. 5) has been produced. From the input parameters, including the geometry and the soil characteristics, the program can compute the following:

- Critical gradient
- Exit gradient
- Quantity of discharge
- Uplift force
- Safety factor with respect to piping

In addition, it can plot the pressure distribution on the bottom of the dam.

INPUT

The execution of the program starts with *screen 1* which consists of six entries; the number of fragments, permeability, specific gravity, void ratio, water level in the upstream above the ground level and water level in the downstream above the ground level. All these parameters must be known and fed into the program. Then, *screen 2* appears requesting the dimensions of the upstream and downstream fragments. The program allows the user to check the input values and make any corrections needed. These dimensions are: the depth of the sheet pile; the length of the upstream blanket; and the thickness of the soil layer. The dimensions of the middle fragment are required to be entered in *screen 3*. They consist of the depths of sheet piles and the width of the dam. Similar to *screen 2*, the user can check the input values and make any corrections needed. These input screens are given below.

Screen 1

Enter the number of fragments
Input the permeability
Input the specific gravity
Input the void ratio
Input the water level in the upstream above ground level
Input the water level in the downstream above ground level

Screen 2

Input the dimensions of the upstream fragment
Input the dimensions of the downstream fragment

Screen 3

Input the dimensions of fragment No. 2

OUTPUT

At the completion of the input data, the program will show the results. *Screen 4* presents the values of the form factors and the head losses of the three fragments, the pressure distribution on the bottom of the dam, critical gradient, exit gradient, quantity of discharge, uplift force and safety factor with respect to piping. The output screen (screen 4) is given below.

Screen 4

Total head Loss
Head loss and form factor per fragment
Pressure distribution on the bottom of the dam
Critical gradient
Exit gradient
Quantity of discharge
Uplift force
Safety factor with respect to piping.

EXAMPLE PROBLEM

A typical seepage problem has been selected to show the use of the program. The dam has an upstream blanket and two sheet piles. Figures 6, 7, 8 and 9 give the complete input and output data for the example problem.

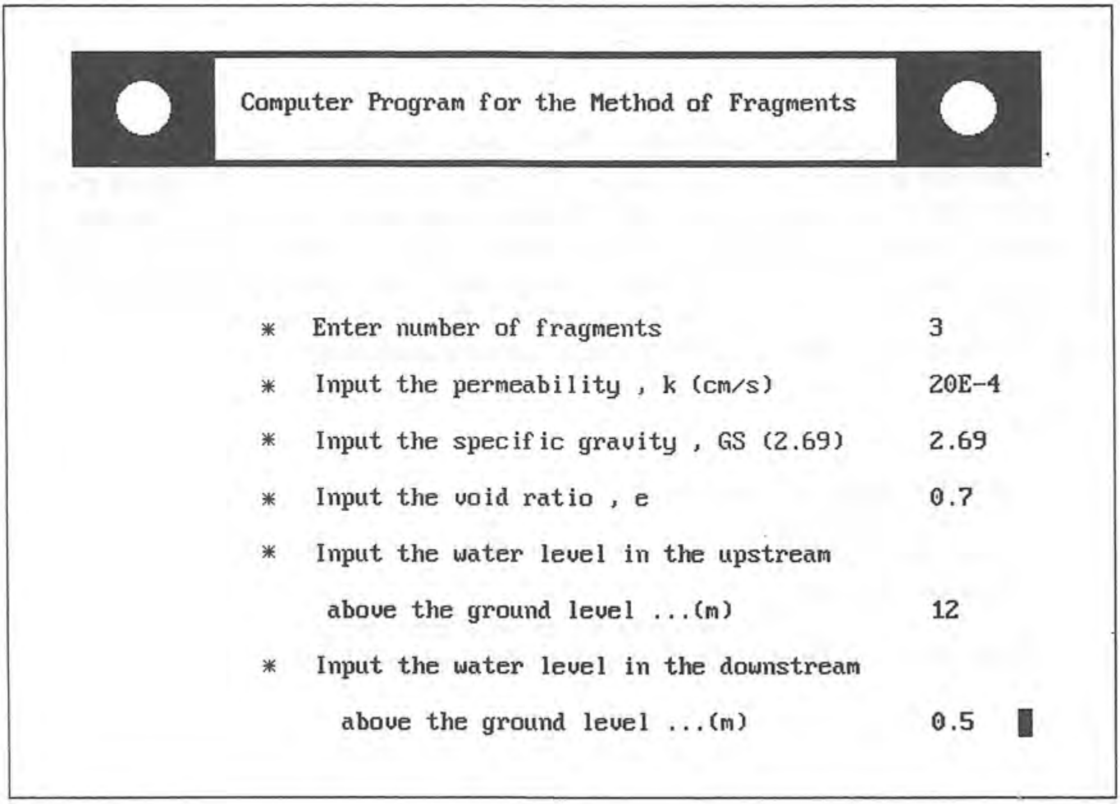


Fig. 6 Input screen 1 for example problem

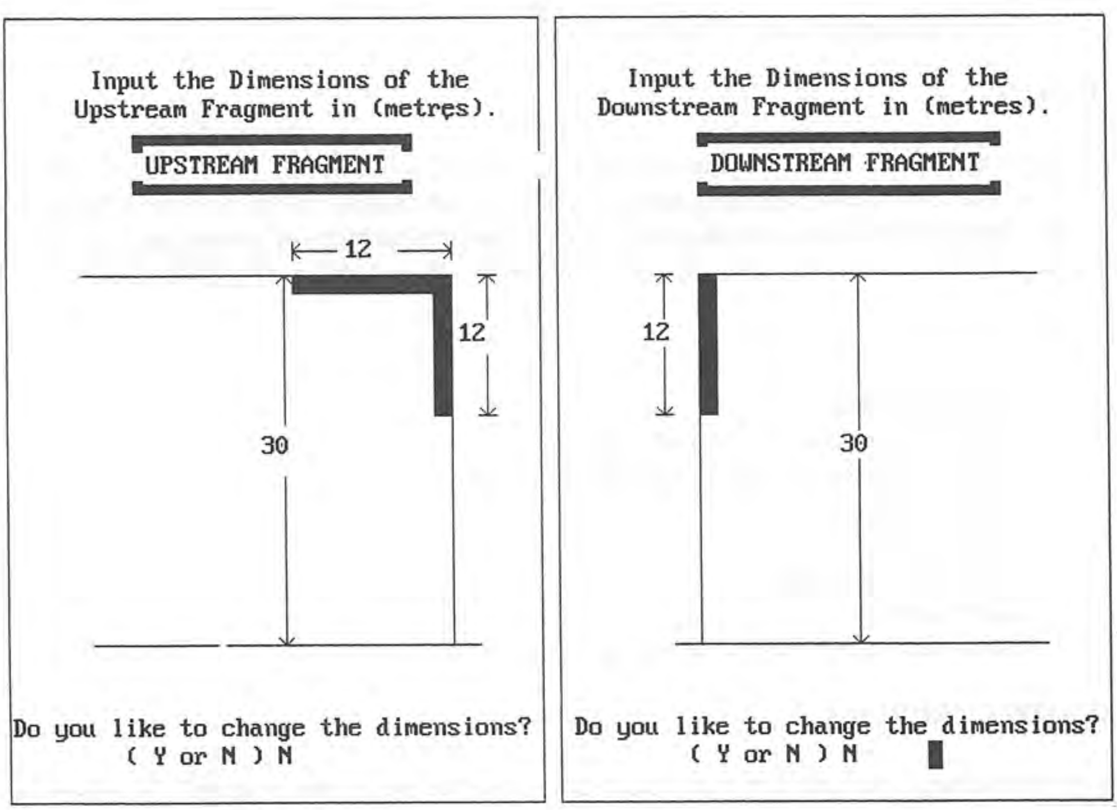


Fig. 7 Input screen 2 for example problem

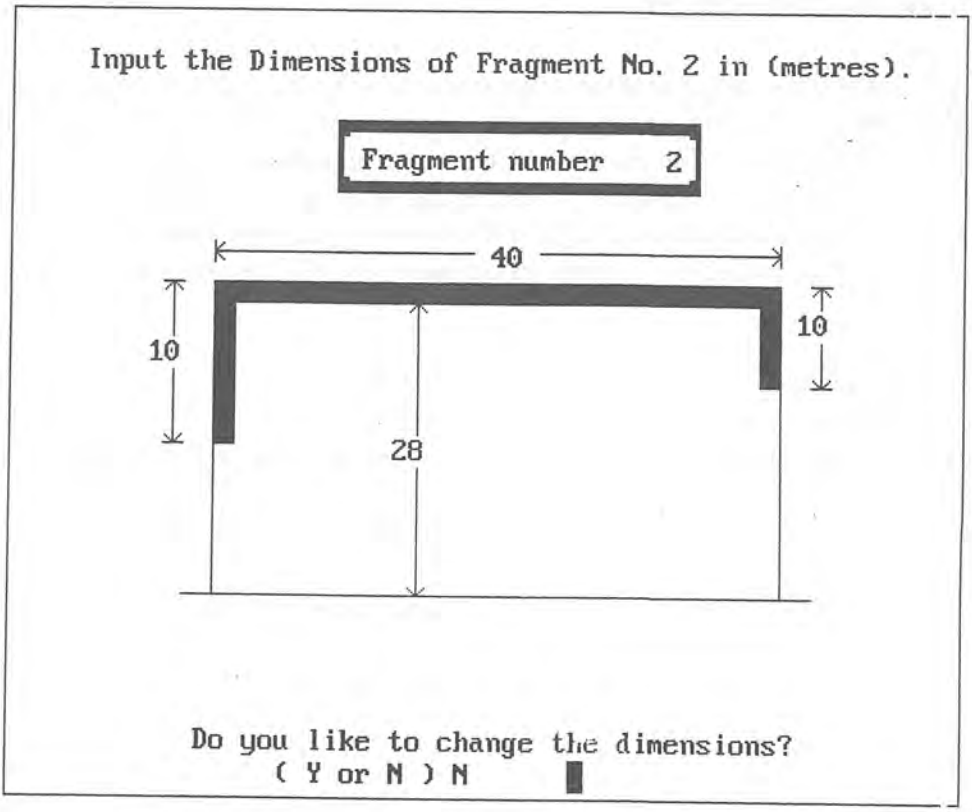


Fig. 8 Input screen 3 for example problem

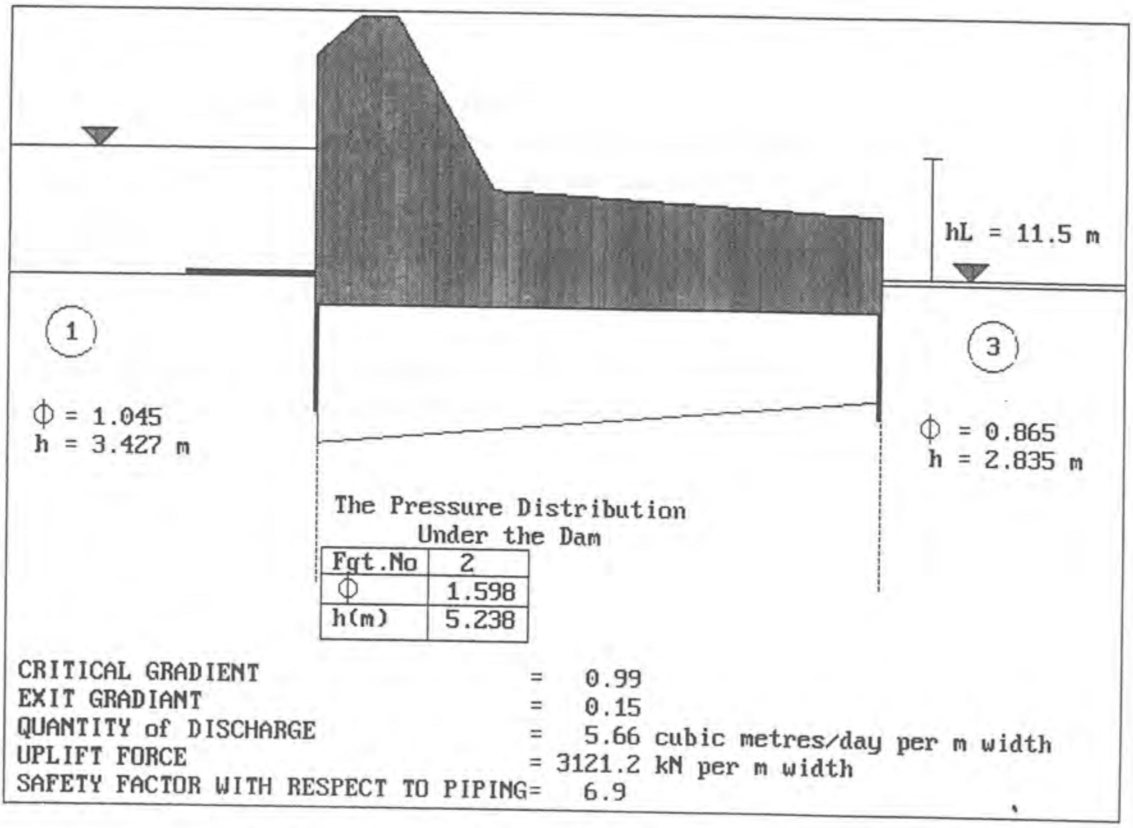


Fig. 9 Output summary (screen 4) for example problem

VALIDITY OF THE PROGRAM

Nine randomly selected seepage problems were solved by Sivakugan and Al-Aghbari (1993)a. These problems were solved by manual calculation and by the program. The analysis is based on the method of fragments. The quantity of flow, the maximum exit gradient and the uplift force are computed by both techniques. These results showed excellent agreement of the two techniques. Only three cases showed slight differences which were less than 5%. The above example problem was solved by manual calculation and the results obtained are given in below:

Critical gradient	=	0.99
Exit gradient	=	0.15
Quantity of discharge	=	5.66 m^3 /day per m width
Uplift force	=	3131 kN per m width
Safety factor with respect to piping	=	6.6

The above results showed excellent agreement between calculations done manually and by the program. The value of the uplift force is slightly different from that obtained by the computer program. However, the difference is less than 1%.

CONCLUSION

It was shown that the developed program provides quick solutions for seepage problems using the method of fragments. The procedure is simple and easy to follow. Comparative study showed excellent agreement between calculations done manually and by the program.

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Reliability Studies on Pipe Flow

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Reliability Studies on Pipe Flow

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ABSTRACT

In the past two decades there had been a substantial developments in the areas of reliability and its applications to civil engineering problems. This paper presents the results of one reliability studies carried out on gravity flow through pipes. The flow through pipe or open channel sections is generally computed using two well known formulae, Manning's and Hazen-Williams, where, the flow is expressed as a function of the section dimensions, hydraulic gradient, the Hazen-Williams coefficient and the roughness coefficient. Usually, these parameters are assumed to be constants and the flow is computed determinstically. Nevertheless, in reality, these parameters can be random variables with specific probability distributions. As a results, the flow also becomes a random variable with well defined probability distribution. The water withdrawal from a system is also generally a random variable, with a specific probability distribution. The system functions satisfactorily as long as the flow in the system is larger or equal than the water supply; the system fails when the water withdrawal is larger than the flow in the system. A reliability comparison is made between Manning's and Hazen-Williams formulae for gravity flow through circular pipe. The results are reported in this paper. The simple methodology presented in this paper can be applied to similar water-related problems as well.

Key Words

Random variables, Reliability, Manning's formula, Hazen-Williams formula

Introduction

In the past two decades, there had been significant developments in the areas of reliability and its applications on civil engineering problems. Reliability studies are increasingly becoming popular in earthquake engineering, geotechnical engineering, ocean engineering, etc. where, there is significant uncertainty in the theory and the parameters used. If it is possible to quantify the uncertainties, it will be more meaningful to go for a probabilistic approach than the conventional deterministic one. In this paper, an attempt is made to develop a probabilistic framework to study the gravity flow through pipes.

The flow through pipe or open channel sections is generally computed using two well known formulae: Manning's and Hazen-Williams, where, the flow is expressed as a function of the section dimensions, hydraulic gradient, the Hazen-Williams coefficient and the roughness coefficient. Usually, these parameters are assumed to be constants and the flow is computed determinstically. Nevertheless, in reality, these parameters can be random variables with specific probability distributions. As a results, the flow also becomes a random variable with well defined probability distribution.

The water supply available to the pipe is also generally a random variable. The actual flow through the pipe is the demand on the other end. The system functions satisfactorily as long as the supply is greater than demand it fails when the demand exceeds the capacity. The reliability of the system is simply the probability that it will work satisfactorily.

Probabilistic Model

Manning's formula for gravity flow through a circular pipe, flowing full, is given by:

$$Q=\frac{0.3115}{n}d^{\frac{8}{3}}i^{\frac{1}{2}} \tag{1}$$

Where, d is the pipe diameter in m, i is the hydraulic gradient and n is the roughness coefficient. Also, the gravity flow can be obtained by Hazen-Williams formula. For gravity flow through a circular pipe, flowing full, is given by:

$$Q=0.278\,C_{HW}\,d^{2.63}\,i^{0.54} \tag{2}$$

Where C_{HW} is the Hazen-Williams coefficient. Q is the volume of flow per unit time, in m^3/s. n, d, i and C_{HW} are random variables with known probability distributions, and in this paper, they are assumed to follow guassian normal distribution. Therefore, Q is a dependent random variable for which probability distribution is yet to be determined. Sivakugan (1994) suggested three simple methods for studying such probabilistic random variables. One of them, an analytical method, is used in this paper in studying the variability of Q.

Q in Manning's formula is a function of three independent variables, and can be written as Q = Q(n, d, i). From the simple statistical concept (Harr 1977), the mean and the standard deviation of Q can be written as:

$$\overline{Q} = Q(\overline{n}, \overline{d}, \overline{i}) + \frac{1}{2}\left(\frac{\partial^2 Q}{\partial n^2}\sigma_n^2 + \frac{\partial^2 Q}{\partial d^2}\sigma_d^2 + \frac{\partial^2 Q}{\partial i^2}\sigma_i^2\right) \tag{3}$$

$$\sigma_Q^2 = \left(\frac{\partial Q}{\partial n}\right)^2 \sigma_n^2 + \left(\frac{\partial Q}{\partial d}\right)^2 \sigma_d^2 + \left(\frac{\partial Q}{\partial i}\right)^2 \sigma_i^2 \tag{4}$$

Where, the derivatives of Q are evaluated at the same mean values of n, d and i. Thus, for a specific set of input parameters $\overline{n}, \sigma_n, \overline{d}, \sigma_d, \overline{i}, \sigma_i$, using Eqns. 3 and 4, $\overline{Q}, \sigma_Q$ can be easily determined. The same procedure can be applied for Hazen-Williams formula.

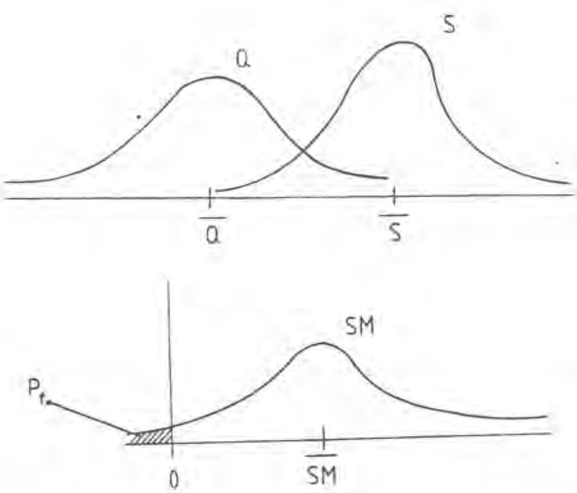

Figure 1. Probability Distributions of S, Q and SM

The water supply available to the pipe S, is assumed here to follow Gaussian normal distribution, where, the mean ($\overline{S}$) and the standard deviation (σ_S) are known. The volume of flow Q, computed above can be considered as demand, which has to be met for the system to perform satisfactorily, S should be greater than Q. The probability of failure of the system, P_f, can be defined as:

$$P_f = P[\, S < Q \,] \\ = P[\, S\text{-}Q < 0 \,] \qquad (5)$$

Where, P[ ] is the probability of event given within the parenthesis. Let us introduce a random variable, known as safety margin (SM) and defines as algebraic difference between the supply (S) and demand (Q). Thus, SM = S - Q. The probability distributions of Q, S and SM are shown in Fig. 1. The probability of failure of the system is simply the probability that the safety margin is less than zero i.e., $P_f = P[\, SM<Q \,]$. The reliability (R) of the system is defined as:

$$R = 1 - P_f \qquad (6)$$

A typical design problem

Given: for flow through a circular pipe, flowing full, the following typical parameters may be assumed.

$\bar{n} = 0.015$,	$CV_n = 30\ \%$
$\bar{i} = 0.004$,	$CV_i = 20\ \%$
$\bar{d} = 0.4$ m,	$CV_d = 5\ \%$
$\bar{C}_{HW} = 130$,	$CV_{HW} = 30\ \%$
$\bar{S} = 0.4$ m³/s,	$CV_S = 20\ \%$

Required: Find the reliability of the system.

The derivation procedure and results are summarized in tables 1 and 2 respectively.

Table 1 Summarize the derivation procedure for Manning's and Hazen-Williams formulae respectively

Term	Manning's	Hazen-Williams
Q	$\frac{0.3115}{n} d^{\frac{8}{3}} i^{\frac{1}{2}}$	$0.278\, C_{HW}\, d^{2.63}\, i^{0.54}$
$\frac{\partial Q}{\partial n}, \frac{\partial Q}{\partial C_{HW}}$	$-\frac{0.3115}{n^2} d^{\frac{8}{3}} i^{\frac{1}{2}}$	$0.278\, d^{2.63}\, i^{0.54}$
$\frac{\partial Q}{\partial d}$	$\frac{0.8315}{n} d^{\frac{5}{3}} i^{\frac{1}{2}}$	$0.731\, C_{HW}\, d^{1.63}\, i^{0.54}$
$\frac{\partial Q}{\partial i}$	$\frac{0.1559}{n} d^{\frac{8}{3}} i^{-\frac{1}{2}}$	$0.150\, C_{HW}\, d^{2.63}\, i^{-0.46}$
$\frac{\partial^2 Q}{\partial n^2}, \frac{\partial^2 Q}{\partial C_{HW}^2}$	$\frac{0.6236}{n^3} d^{\frac{8}{3}} i^{\frac{1}{2}}$	0
$\frac{\partial^2 Q}{\partial d^2}$	$\frac{1.3858}{n} d^{\frac{2}{3}} i^{\frac{1}{2}}$	$1.192\, C_{HW}\, d^{0.63}\, i^{0.54}$
$\frac{\partial^2 Q}{\partial i^2}$	$-\frac{0.078}{n} d^{\frac{8}{3}} i^{-\frac{3}{2}}$	$-0.069\, C_{HW}\, d^{2.63}\, i^{-1.46}$

Table 2 Summarize the results obtained using Manning's and Hazen-Williams formulae respectively

Term	Manning's	Hazen-Williams
$Q_{(\bar{n},\bar{d},\bar{i})}, Q_{(\bar{n},\bar{d},\bar{C}_{HW})}$	0.114	0.165
$\left.\frac{\partial Q}{\partial n}\right\|_{mean}, \left.\frac{\partial Q}{\partial C_{HW}}\right\|_{mean}$	- 7.61	0.00127
$\left.\frac{\partial Q}{\partial d}\right\|_{mean}$	0.761	1.082
$\left.\frac{\partial Q}{\partial i}\right\|_{mean}$	14.3	22.21
$\left.\frac{\partial^2 Q}{\partial n^2}\right\|_{mean}, \left.\frac{\partial^2 Q}{\partial C_{HW}^2}\right\|_{mean}$	1015	0
$\left.\frac{\partial^2 Q}{\partial d^2}\right\|_{mean}$	3.17	4.41
$\left.\frac{\partial^2 Q}{\partial i^2}\right\|_{mean}$	- 1786	- 2554
$\overline{Q}$	0.124	0.165
$\overline{S}$	0.40	0.40
$\overline{SM}$	0.276	0.235
σ_Q	0.039	0.057
σ_S	0.1	0.1
σ_{SM}	0107	0.115
$\Psi(Z)$	2.60	2.05
Z	0.4953	0.4797
P_f'	0.0047	0.0203
Reliability	99.53 %	97.97 %

Conclusion

This paper presents a simple probabilistic approach to study the reliability of pipe flow using two well known formulae: Manning's and Hazen-Williams. A typical design problem has been selected for the application. Assuming probability distributions for the water supply, demand (withdrawal) and the safety margin between supply and demand, the results indicated that:

(a) Both formulae have high reliability in the application 99.53 % and 97.97 % for Manning's and Hazen-Williams respectively;

(b) Manning's has higher reliability, but the difference is insignificant.

It is important to report that, this simple method can be extended to cover more formulae and be applied to all typical sections in gravity flow such as trapezoidal, rectangular and triangular. This method requires the evaluation of the derivatives at the mean values where this effort ranges from being simple to cumbersome to impossible, such as, for the latter, when the function is given implicitly in the form of charts, tables or as finite element solutions.

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مؤتمر الخليج الثاني للمياه

البحرين ٥ - ٩ نوفمبر ١٩٩٤

الماء في الخليج، نحو إدارة متكاملة

وثائق المؤتمر

المجلد الأول - الاوراق العربية
المجلد الثاني - الاوراق الانجليزية

جمعية علوم وتقنية المياه

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