The 7th Gulf Water Conference

مؤتمر الخليج السابع للمياه

WSTA Seventh Gulf Water Conference

Water in the GCC - Towards an Integrated Management

19 - 23 November 2005, State of Kuwait

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Vol II

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The Seventh Gulf Water Conference
Water in the GCC...Towards an Integrated Management
State of Kuwait, 19-23 November 2005

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Preface

Water is the most valuable resource on Earth. It is an important vector in the socio-economic development and for supporting the ecosystem. In the arid to extremely arid Arabian Peninsula, home of the GCC countries, the importance and value of water is even more pronounced. The GCC countries of United Arab Emirates, Bahrain, Saudi Arabia, Oman, Qatar, and Kuwait, are facing the most severe water shortages in the world. Rainfall scarcity and variability coupled with high evaporation rates have made these countries the least endowed in these resources in the world. However, the scarcity of renewable water resources is not the only distinctive characteristic of the region, inadequate management intervention and the continuous deterioration of its natural water resources have become during the past few decades equally distinguishing features as well.

In the last three decades, rapid population growth and accelerated socio-economic development in the GCC countries were associated with a substantial increase in water demands, which have escalated from less than 5 billion cubic meter (bcm) in 1970 to about 30 bcm in 2000. These demands have been driven mainly by agricultural consumptions (currently at 81% of total water used in GCC countries), and by rapid urban expansion (19%).

To meet rising demands, water authorities have focused their efforts mainly on the development and supply augmentation aspects of water resources management. Demands are being satisfied by the development of groundwater (91%), extensive installation of desalination plants (7%), expansion in wastewater treatment and reuse (2%), in addition to dams construction to collect, store, and utilize runoff. Currently, groundwater resources are being over-exploited to meet mainly agricultural water demands, which contributes less than 2% of GDP in most of these countries, and with continuous deterioration in their quantity and quality. In most of the countries, unplanned groundwater mining continues without a clear "exit" strategy and address to the "what comes after" question.

To meet domestic water supply requirement, GCC countries have turned to desalination and have become collectively the world leaders in desalination, with more than 50% of the world capacity. However, desalination remains capital intensive and costly in the region, especially that these countries do not possess this technology; while financial cost of desalinated seawater has been decreasing to around US$ 0.70 per cubic meter in the USA and other places, the average water production costs in the GCC countries remain somewhere between US$ 1 to 2 per cubic meter. In terms of wastewater treatment and recycling, the coverage rate of sewage collection and treatment system seems to be lagging far behind water supply service (20-60%), and available treated wastewaters are still not being reused to their potential and without consideration to the opportunity cost of the reclaimed water treated to a tertiary level. Although planning for full utilization of treated effluent are in the early stages, most of the GCC countries have ambitious plans for the full utilization of treated effluent.

The supply-driven approach for water management has demonstrated its inability to deliver substantial degree of water sustainability or security to the water-stressed GCC countries; despite the strenuous efforts made by these countries in this approach, they still face serious water deficits due to the continuously increasing water demands beyond the limits of their available water resources. Indeed, it is questionable if adequate supplies can be sustained in the future without heavy burden on national budget and might have expensive socio-economic impacts. In fact, the supply augmentation approach coupled with inadequate attention to improving and maximizing the efficiency of water allocation and water use have led to the emergence of a number of unsustainable water uses in these countries, such as low water use efficiency, growth of per capita water use, increasing cost of water production and distribution, and deterioration of water quality as well as land productivity. The situation was further aggravated by the lack of comprehensive long-term water policies and strategies that are based on supply-demand considerations, and was further compounded by the institutional weaknesses, fragmentation and overlap of water agencies, and inadequate institutional capacity building and enabled society.

As pressures on water converge on the region's water resources, the need for innovative approaches in water management becomes more apparent and quite urgent. The international community has recognized this fact, and over the past decade a consensus had been formed on integrated water resources management (IWRM) as an appropriate approach to address threats posed to water resources and to ensure its sustainability. Within the framework of sustainable management of water resources, IWRM takes into account a broad spectrum of social, economic, and ecological factors and their links. Effective coordination and participatory decision-making process are insured throughout IWRM. IWRM process depends on collaboration and partnerships at all levels, from individual citizens to international organizations, based on a political commitment to, and wider societal awareness of the need for water security and the sustainable management of water resources. To achieve IWRM, there is a need for coherent national and regional policies to overcome fragmentation and lack of good governance.

Fortunately, all the GCC countries have realized that efficient development and management of water resources requires water policy reforms, with more emphasis on demand management measures and improvement of the legal and institutional provisions. In essence, appropriate water sector policy reform should address the key issues of reliable assessment of water supply and demand, water quality deterioration and protection, water use efficiency and allocation, role of the private sector, pricing policies and cost recovery, groundwater mining, stakeholder participation, improved institutional support, food security and the increasing problem of water scarcity. Water policy reform needs to address these key issues, taking into consideration the specific requirements and the prevailing social, economic, and cultural conditions of the GCC countries.

Addressing the immense challenges associated with water resources management requires daring reforms to existing institutions and policies governing water resources. Far reaching and multi-sectoral approaches will be critical if we are to overcome inefficient use of water resources and make their use sustainable. This will require the establishment of a proper enabling environment that ensures the rights of users and provides the appropriate level of protection for the resource. Policies, legislation, establishment of governing bodies at various levels and knowledge management are all part of ensuring that the objectives of IWRM are met. The Seventh Gulf Water

Conference was intended to be a landmark in the water resources management in the GCC countries and to lay the foundation of IWRM policies and strategies in these countries.

As at the previous WSTA conferences (Dubai, 1992; Bahrain, 1994; Oman, 1997; Bahrain, 1999; Qatar, 2001; Saudi Arabia, 2003) the overall goals of the Seventh Gulf Water Conference are to encourage scientific studies and research in the different fields of water resources, to create a forum of open discussion and exchange experiences among the Gulf States that the WSTA engendered through the six previous conferences.

The objectives of the convening conference are: 1) Review and assess the progress made in the GCC countries towards the internationally agreed on target of adoption and implementation of comprehensive policies and strategies for integrated water resources management and their active implementation (WSSD, Johannesburg, 2002); 2) Identify main issues, challenges, constraints, as well as opportunities and lessons learned in the implementation of IWRM for sustainable development in arid regions with special reference to the GCC countries and its prevailing socio-economic, political, and environmental conditions; 3) Assess current status of natural and non-conventional water resources in the region in relation to present and future water demands in the GCC countries; 4) Promote and encourage a shift in the region from the supply-driven approach to the demand-centered and conservation approach within the framework of IWRM; and 5) Promote the development of a research and development strategy for the water sector in the region with special reference to the enhancement of existing and new technologies for water desalination and wastewater treatment and their role in enhancing water supply.

The Seventh Gulf Water Conference was held under the patronage of His Highness Sheikh Sabah Al-Ahmad Al-Sabah, the Prime Minister of the State of Kuwait, and is organized by the Water Science and Technology Association (WSTA) in cooperation with Kuwait Institute for Scientific Research (KISR) and the Secretariat General of the Cooperation Council (GCC) for the Arab States of the Gulf. The Conference was sponsored by the Ministry of Energy, Mohammed Abdulmohshin Al-Kharafi & Sons W.L.L, Kuwait Shell Limited, Kuwait Foundation for the Advancement of Science (KFAS), Prince Sultan Bin Abdulaziz International Prize for Water, National Water Technology, COMSTECH, Environment Public Authority (KEPA), UNESCO Cairo Office, Arab Center for Studies in Arid and Dry Lands (ACSAD, Syria), United Nations Environment Program (UNEP/ROWA), and United Nations Economic and Social Commission for West Asia (ESCWA, Lebanon). The Conference was supported by the Arabian Gulf University, Food and Agricultural Organization (FAO/RNA, Egypt), Arab Organization for Agricultural Development (AOAD, Sudan), International Atomic Energy Agency (IAEA), International Center for Biosaline Agriculture (ICBA, Dubai), United Nations University (UNU), European Desalination Society (EDS), International Desalination Association (IDA), and the World Bank (WB).

This conference proceedings contains 102 papers assembled into three volumes, one volume in Arabic (15 papers) and the other two in English (87 Papers). The conference papers were selected by the Conference Scientific Committee from over 150 abstracts received from the conference call of papers. Many of these were modified to meet the standards of the Scientific Committee review. The conference papers were reviewed by 42 regional and international water science and technology experts, with each paper reviewed by two reviewers. The final editing of the conference papers were made by Dr. Waleed Al-Zubari (AGU), Dr. Meshaan Al-Otaibi (KISR), and Dr. Amjad Aliewi (HEW, Palestine). Twenty five papers were invited from GCC water-related government agencies, supporting organizations and renowned regional and international experts to give scientific presentations in respective technical sessions. Conference sessions will be held on 10 topics: Water Resources Management and Planning; Development and Management of Conventional Water Resources (Groundwater and Surface Water Resources); Municipal Water Management; Domestic Wastewater Treatment and Reuse; Public Private Partnership; Water, Health, and Environment; Water and Agriculture; Desalination and Treatment Technologies; and Water Management in the Oil Industry.

The Scientific Committee wishes to express its deep appreciation to the Government of the State of Kuwait and Kuwait Institute for Scientific Research (KISR) for hosting and organizing the conference, and the many Kuwait, regional, and international conference sponsoring agencies, organizations, and companies, who kindly supported and endorsed this conference by providing their generous funds, keynote speakers, and political support.

Organization of the Gulf Water Conferences requires considerable time and effort. As in the previous WSTA conferences, individuals from various sectors (industry, government, and academia) have come forth and given generously their time. Special thanks are due to the members of the Organizing Committee and the Scientific Committee, and Scientific Papers Reviewers.

Finally, the Scientific Committee wishes to acknowledge the immeasurable contributions made by the authors and their research associates who were not only willing to rework and modify their manuscripts, but also had to meet an extremely tight time schedule. Without their efforts this document would not have been possible. We sincerely hope that this conference will achieve its objectives and is both enjoyable and rewarding for you.

Prof. Dr. Waleed K Al-Zubari
Chairman, Conference Scientific Committee
Vice-Dean, College of Graduate Studies
Arabian Gulf University

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DESALINATED & MUNICIPAL WATER

Desalination of seawater using nuclear energy

B.M.Misra

DESALINATION OF SEAWATER USING NUCLEAR ENERGY

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ABSTRACT

Water, energy and environment are essential inputs for the sustainable development of society in the coming years and are therefore aptly mentioned as life support systems. Recent statistics show that currently 2.3 billion people live in water-stressed areas and among them 1.7 billion live in water-scarce areas. In the light of this, the Millennium Declaration by UN General Assembly in 2000 set up a target to halve, by the year 2015, the world population who are unable to reach, or to afford safe drinking water. Better water conservation, water management, pollution control and water reclamation are all part of the solution to projected water stress. So too are new sources of fresh water, including the desalination of seawater. Desalination technologies have been well established since the mid 20th century and are widely deployed in many parts of the world having acute water scarcity problems. The energy for these plants is generally supplied in the form of either steam or electricity largely using fossil fuels. The intensive use of fossil fuels raises environmental concerns especially in relation to greenhouse gas emissions. The depleting sources and the future price uncertainty of the fossil fuels and their better use for other vital industrial applications is also a factor to be considered for sustainability. The desalination of seawater using nuclear energy is a feasible option to meet the growing demand of potable water. Over 150 reactor-years of operating experience of nuclear desalination have been accumulated worldwide. Several demonstration programs of nuclear desalination are also in progress to confirm its technical and economic viability under country specific conditions, with technical co-ordination or support of IAEA. Recent techno-economic feasibility studies carried out by some Member States indicate the competitiveness of nuclear desalination. This paper presents the salient activities on nuclear desalination in the Agency and in the interested Member States. Economic research on further water cost reduction includes investigation on utilization of waste heat from different reactor types for thermal desalination, pre- heat reverse osmosis and hybrid desalination systems. The main challenge for the large-scale deployment of nuclear seawater desalination is the lack of infrastructure and resources in the countries affected by water scarcity problems which are however, interested in adoption of nuclear desalination for the sustainable water resources. Socio-economic & environmental aspects and the public perception are also important factors requiring greater information exchange.

1. Desalination as an Alternate Source of Fresh Water

Water, energy and environment are essential inputs for the sustainable development of society in the coming years and are therefore aptly mentioned as life support systems. These are therefore current issues of deliberations at national and international forums. Seventy percent of the planet is covered with water but only 2.5% of that is fresh water. Nearly 70% of this fresh water is frozen in the icecaps of Antarctica and Greenland. Most of the rest is in the form of soil moisture or in deep inaccessible aquifers, or comes in the form of monsoons and floods that are difficult to contain and exploit. Less than 0.08% of the world's water is thus readily accessible for direct human use, and even that is very unevenly distributed.

Recent statistics show that currently 2.3 billion people live in water-stressed areas and among them 1.7 billion live in water-scarce areas, where the water availability per person is less than 1000 m^3/year. In fact, the situation is going to worsen further, statistics show that by 2025 the number of people suffering from water stress or scarcity could swell to 3.5 billion and 2.4 billion of them are expected to live in water-scarce regions. Water scarcity is a global issue, and every year new countries are affected by growing water problems (1).

In the light of this, the Millennium Declaration by UN General Assembly in 2000 set up a target to halve, by the year 2015, the world population who are unable to reach, or to afford, safe drinking water. Vision 21, shared vision for Hygiene, Water supply and Sanitation has a target to provide water, sanitation & hygiene for all by 2025 (2).

Better water conservation, water management, pollution control and water reclamation are all part of the solution to projected water stress. So too are new sources of fresh water, including the desalination of seawater. Desalination technologies have been well established since the mid-20th century and widely deployed in the Middle East and North Africa. The contracted capacity of desalination plants has increased steadily since 1965 and is now about 36 million m^3/d worldwide, as shown (3) in Figure 1. This capacity could roughly cater 6 litres a day per capita of fresh potable water to the world's population.

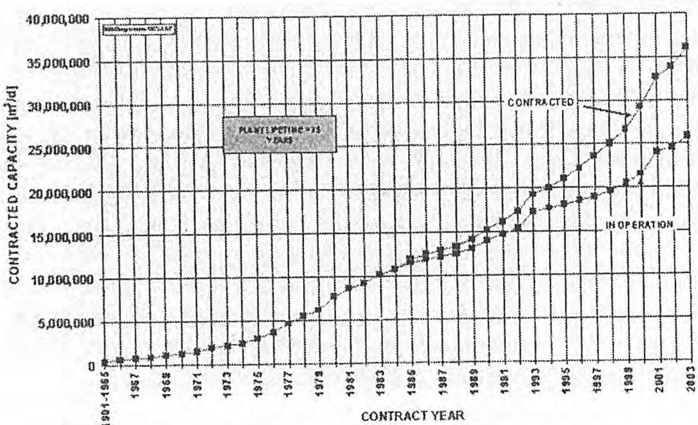

Fig. 1: Cumulative worldwide desalination capacity; the top line shows total operating and contracted capacity and the bottom line shows only the operating capacity.

Large-scale commercially available desalination processes can generally be classified into two categories: (a) distillation processes that require mainly heat plus some electricity for ancillary equipment, and (b) membrane processes that require only electricity. In the first category (distillation) there are two major processes: multi-stage flash (MSF) and multi-effect distillation (MED). In both, seawater is heated; the steam that evaporates is condensed and collected as freshwater; and the residual brine is discharged. In the second category (membranes) is the reverse osmosis process (RO), in which pure water passes from the high-pressure seawater side of a semi-permeable membrane to the low-pressure freshwater permeate side. The pressure differential must be high enough to overcome the natural tendency for water to move from the low concentration freshwater side of a membrane to the high concentration seawater side, in order to balance osmotic pressures. The salient characteristics of various desalination processes are shown in Table 1.

Table 1: Key data of processes

	MSF	**MED**	**MED-TVC**	**MVC**	**RO**	**ED**
Operating temperature (°C)	< 120	< 70	< 70	< 70/1/	< 45	<45
Form of main energy	Steam (heat)	Steam (heat)	Steam	Mechanical (electrical energy)	Mechanical (electrical energy)	Electrical energy
Thermal energy consumption (kWh/m³) /4/	12	6	21	Not applicable	Not applicable	Not applicable
Electrical energy consumption (kWh/m³)	3.5	1.5	1.5	8-14	4-7	1.0
Typical salt content of raw water (ppm TDS)	30,000 – 100,000	30,000 – 100,000	30,000 – 100,000	30,000 – 50,000	1,000 – 45,000	100 – 3,000
Product water quality (ppm TDS)	< 10	< 10	< 10	<10	< 500	< 500
Current, typical Single-train capacity (m³/d)	5,000 – 60,000	500 – 12,000	100 – 20,000	10 – 2,500	1 – 10,000	1 – 12,000

2. The Role of Nuclear Power in Desalination

The world energy requirements are presently met from oil, coal, gas, hydro, nuclear and renewable in that order as shown in Table 2.

Table 2: Percentage of World Energy Use (4)

Fuel	Percentage (%)	Present trends
Oil	39	Building of more fossil fuel plants
Coal	25	
Gas	22	Short-term – greater burning of oil, coal and gas resulting in more CO_2
Hydro	7	
Nuclear	6	
Renewables	1	Greater energy efficiency – increased renewable sources of energy: geothermal, wind, solar, bio-mass

There is general agreement that there will be an increase in the world's requirement for electricity over the next few decades. The present trend towards meeting this demand includes the building of fossil fuel plants, particularly combined cycle gas fired plants. The spiralling increase in greenhouse gas emissions has resulted in setting the emission targets in Toronto, Rio and the last one in Kyoto. The International Atomic Energy Agency (IAEA) predicts that emissions would be 36-50% higher by 2010 compared with those in 1990. Many commentators, therefore, feel that the only viable alternative to fossil fuel is nuclear energy to reduce the rate of increase of greenhouse gases, particularly, carbon dioxide. Another incentive for nuclear power is to maintain diversity of supply. A national strategy limited to one particular form of energy (fuel) will be vulnerable to reduction of other fuel costs.

Nuclear power is a proven technology, which has provided more than 16% of world electricity supply in over 30 countries. More than ten thousand reactor-years of operating experience have been accumulated over the past 5 decades. In recent years, the option of combining nuclear power with seawater desalination has been explored to tackle water shortage problems. The desalination of seawater using nuclear energy is a feasible option to meet the growing demand for potable water. Over 150 reactor-years of operating experience on nuclear desalination have been accumulated worldwide. Several demonstration programs of nuclear desalination are also in progress to confirm its technical and economical viability under country-specific conditions, with technical co-ordination or support of IAEA.

There are many reasons which favour a possible revival of the nuclear power production in the years to come: the development of innovative reactor concepts and fuel cycles with enhanced safety features which are expected to improve public acceptance, the production of less expensive energy as compared to other options, the need for prudent use of fossil energy sources, and the increasing requirements to curtail the production of greenhouse gases (GHG), toxic gases, particulates and acid rain, which are all associated with the combustion of fossil fuels.

It is thus expected that this revival would also lead to an increased role of nuclear energy in non-electrical energy services, which, at the moment, are almost entirely dominated by fossil energy sources. Among various utilizations of nuclear energy for non-electrical products, using it for the production of freshwater from seawater (nuclear desalination) has been drawing broad interest in IAEA Member States as a result of acute water shortage issues in many arid and semi-arid zones worldwide.

The issue has been repeatedly stressed at the General Conference and supported by many Member States including most members of a Group of 77. The support stems from their expectation of not only its possible contribution to the freshwater issue but has been motivated by a variety of reasons that include; likely competitiveness of nuclear desalination in areas lacking cheap hydropower or fossil resources, energy supply diversification, conservation of fossil fuel resources and spin-off effects of nuclear technology for industrial development.

2.1. Nuclear Desalination

Nuclear desalination is defined to be the production of potable water from seawater in a facility in which a nuclear reactor is used as the source of energy for the desalination process. Electrical and/or thermal energy may be used in the desalination process on the same site. The facility may be dedicated solely to the production of potable water, or may be used for the generation of electricity and production of potable water, in which case only a portion of the total energy output of the reactor is used for water production.

The design approaches for a nuclear desalination plant are essentially derived from those of the nuclear reactor alone, with some additional aspects to be considered in the design of a desalination plant and its integration with the nuclear system. All nuclear reactor types can provide the energy required by the various desalination processes. The amount of energy (heat or electricity) needed for desalination can be readily supplied by tapping the low-grade steam and/or electricity produced by the nuclear plant. In this regard, it has been shown that Small and Medium Reactors (SMRs) offer the big potential as coupling options to nuclear desalination systems. The development of innovative reactor concepts and fuel cycles with enhanced safety features as well as economics are expected to improve the public acceptance and further the prospect of nuclear desalination.

The coupling with nuclear system is not difficult but needs some consideration in (a) avoiding cross-contamination by radioactivity, (b) providing an alternative source in case the nuclear system is not in operation for refuelling and maintenance, (c) providing certain limits (in case the reactor system is small) in the amount of heat supply to the desalination system considering its effect on the nuclear system from the abrupt shut down of the latter system.

2.2. Experiences and new plans (5)

The desalination of seawater using nuclear energy is a demonstrated option having over 150 reactor-years of operating experience worldwide of which Japan now has over 125 reactor-years. Kazakhstan (Aktau fast reactor BN-350) had accumulated 26 reactor-years of producing 80, 000m^3/day of portable water before shutting down in 1999.

Table 3 summarizes past experience as well as current developments and plans for nuclear-powered desalination based on different nuclear reactor types. Most of the technologies in Table 1 are land-based, but the table also includes a Russian initiative for barge-mounted floating desalination plants. Floating desalination plants could be especially attractive for responding to temporary demands for potable water. Figures 2, 3, and 4 show some view of nuclear desalination plants at Aktau (Kazakhstan), Ohi (Japan) and Kalpakkam (India) and Fig 5 shows the KANUPP site for the proposed nuclear desalination demonstration project in Pakistan.

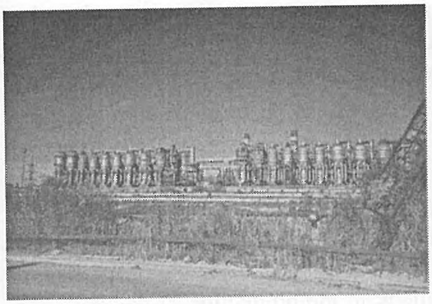

Fig. 2. Evaporators at Aktau, Kazakhstan

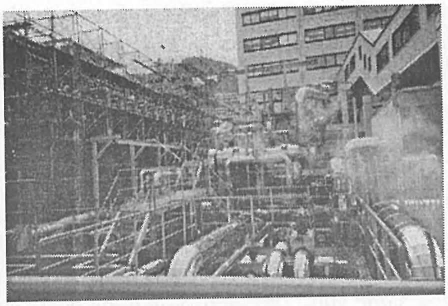

Fig. 3. Operating plant: Ohi, Japan

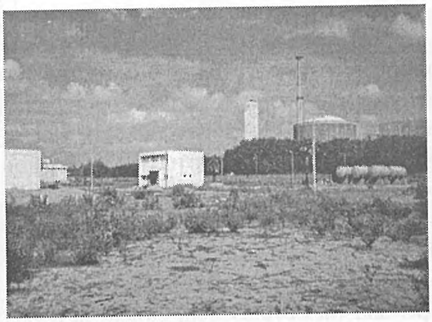

Fig. 4. Hybrid (MSF+RO) plant, Kalpakkam, India

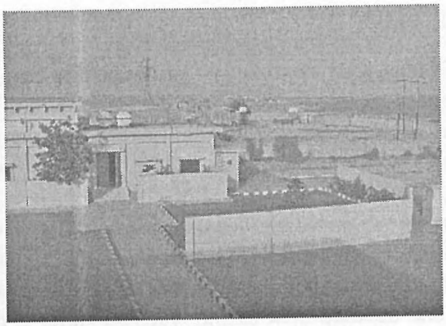

Fig. 5. SWRO Plant at KANUPP, Pakistan

Table3: Reactor Types and Desalination Processes

Reactor Type	Location	Capacities (cu.m. /d)	Status
LMFR	Kazakhstan (Aktau)	80,000	In service till 1999
PWRs	Japan (Ohi, Takahama, Ikata, Genkai)	1,000-2,000	In service with operating experience of over 125 reactor-years.
	Rep. of Korea Argentina	40,000 12,000	Under design
	Russia		Under design (floating unit)
BWR	Japan (Kashiwazaki)		Never in service following testing in 1980s, due to alternative freshwater sources; dismantled in 1999.
PHWR	India (Kalpakkam)	6,300	Under commissioning
	Canada		Under design
	Pakistan (KANUPP)	4,800	Under design
NHR	China		Under design
HTGR	South Africa, France, The Netherlands		Under consideration

The followings provide additional detail on the new developments in the Member States:

- Argentina has identified a site for its small reactor (CAREM), which could be used for desalination. Depending on financing, construction could begin in the near future.
- Canada has embarked on a three-year project to validate its innovative reverse osmosis (RO) system design concepts.
- China is proceeding with several conceptual designs of nuclear desalination using NHR type heating reactor for coastal Chinese cities.
- France is carrying out detailed nuclear desalination studies as part of CEA's own R&D programme or under international programmes with the EU, the IAEA and as bilateral arrangements with India and some other North African countries.
- Egypt has completed a two-year feasibility study for a nuclear co-generation plant (electricity and water) at El-Dabaa. Based on the results, government approval to proceed towards implementing the project is sought.
- India is building a demonstration plant at Kalpakkam using a 6300 m^3/d hybrid desalination system (MSF-RO) connected to an existing PHWR. The RO

plant is already commissioned and operating successfully. India expects to commission the full plant in 2006.

- The Republic of Korea is proceeding with its System-integrated Modular Advanced Reactor (SMART) concept. Work is in the basic design phase. The project is designed to produce 40,000 m^3/d of potable water. Construction project for a SMART-P plant with one-fifth scaled power and a MED plant was launched in 2002 and it will be in operation by 2008.
- Morocco, in June 2000, halted a demonstration project at Tan-Tan originally intended to produce 8000 m^3/d of potable water using an NHR-10 of Chinese design. Possible next steps are being studied.
- Pakistan is considering coupling of a MED thermal desalination plant of 4800 m^3/d capacity with existing PHWR at KANUPP.
- Russia is progressing with the design and licensing of a floating co-generation plant, based on a Nuclear Floating Power Unit (NFPU) with KLT-40C reactors and is looking for the international cooperation for setting such a plant.
- Tunisia has undertaken several studies to select a suitable desalination process and to identify what process could be coupled to a nuclear reactor. La Skhira site has been identified in the southeast part of the country for further study.
- USA will include in its Generation IV roadmap initiative a detailed discussion of potential nuclear energy products in recognition of the important role that future nuclear energy systems can play in producing fresh water.
- Further R&D activities are also underway in Indonesia and Saudi Arabia. In addition, interest has been expressed by Brazil, Iran, Iraq, Italy, Jordan, Lebanon, Libya, Philippines, Syria and UAE in the potential for nuclear desalination in their countries.

3. IAEA Activities on Nuclear Desalination

IAEA has been providing guidebooks technical documents, and computer programs on nuclear desalination as well as technical assistance through the framework of technical co-operation programs

A number of technical co-operation projects have assessed the feasibility of particular projects. In 1999 the IAEA launched an interregional technical co-operation project "Integrated Nuclear Power and Desalination System Design". The project is designed to facilitate international collaboration between technology holders and potential end-users for the joint development of integrated nuclear desalination concepts, aiming at the demonstration of the viability of nuclear desalination at a specific site or sites. Under the IAEA regional technical co-operation framework, several international collaboration activities are underway, for example, between the Republic of Korea and Indonesia; and France and Tunisia.

A CRP on "Optimization of the Coupling of Nuclear Reactors and Desalination Systems", (1998- 2003) covered a review of reactor designs suitable for coupling with desalination systems, the optimization of this coupling, possible performance improvements and advanced technologies of desalination systems for nuclear desalination. A TECDOC covering the salient studies of the CRP is under publication.

A new CRP on "Economic Research on, and Assessment of, Selected Nuclear Desalination Projects and Case Studies" has started early 2002 with the participation of 11 institutions from 11 Member States. It will deepen the economic aspects of nuclear desalination plants and give more confidence in this option.

IAEA's DEEP computer code has been widely used by engineers and researchers for preliminary economic evaluation of desalination by a wide range of fossil and nuclear energy sources.

A Web page on nuclear desalination, describing the current status of nuclear desalination activities in the Member States and the IAEA is available at http://www.iaea.org/nucleardesalination

4. Economics of Nuclear Power/ Desalination

Current trends

The economics of desalination using nuclear energy is obviously linked to the economics of nuclear power. Table 4 gives the average generation costs from nuclear, coal and gas based power plants. Nuclear generation costs are most sensitive to discount rates. Nuclear power is competitive at 5% discount rate but looses its competitive margin at 10%. Availability of funds on soft loan or grant would be helpful in considering a nuclear desalination project.

Table4: Average generation cost ($US per kWh)

Generator	5% Discount	10% Discount
Nuclear	0.034	0.051
Coal	0.038	0.048
Gas	0.040	0.044

However, fuel costs for nuclear plants are generally low in comparison with other energy producers (Table 5). Further, the nuclear fuel cost is not affected by price uncertainties and of course is more sustainable

Table 5: Fuel costs

Generator	Cost (% of generation)
Nuclear	<25
Coal	~40-50
Gas	~75-80

In case of old reactors, the lifetime extension costs are known to be lower than building newer nuclear plants and these are comparable to the new combined cycle plants (Table 6). Attempts are therefore directed in some countries (India and Pakistan) to utilize the existing reactors for setting up demonstration desalination plants to produce fresh water from seawater economically.

Table 6: Lifetime extension versus new building costs ($US per kWe)

Lifetime extension	210 - 840
New nuclear plants	≡ 2000
New combined cycle units	700 - 900

A number of studies have been carried out on coupling of different types of reactors with desalination processes in many Member States. Preliminary techno-economic studies conducted in China for NHR-200 coupled 160,000 m³/d VTE-MED plant estimated the desalted water cost to be around 0.80 US$/m³. Similar economic evaluation of the integrated SMART-MED desalination plant of 40,000 m³/d capacity indicate the water cost ranging from 0.70 to 0.90 US$/m³. The projected cost of water from Russian KLT-40 floating reactor based nuclear desalination plants is also in the above ranges. These costs are comparable with desalination costs using locally available fossil fuels.

Economic comparisons thus indicate that water costs (and associated electricity generation costs) from nuclear seawater desalination are generally in the same range as costs associated with fossil-fuelled desalination. Given the conclusion that nuclear and fossil-fuelled desalination are broadly competitive with each other, any particular future investment decision will depend on site-specific cost factors and on the values of key parameters (fuel price, interest rate, construction time, etc.) at the time of investment. Higher fossil fuel prices would of course favor nuclear desalination; higher interest rates would favor less capital-intensive fossil-fuelled options.

Pre-feasibility studies have been carried out recently for the proposed nuclear desalination projects at Madura, Indonesia and La-Skhira, Tunisia under the IAEA technical cooperation inter-regional project (1999-2004). These indicate economic competitiveness of nuclear desalination over fossil based plants under the specific conditions in their countries (6).

Future developments

There have been many improvements and innovations in the desalination technology in recent years. The energy requirement in the seawater reverse osmosis is reducing progressively due to better energy recovery systems and the membrane costs; resulting in low cost of water from RO plants. This is reflected in the installation of many large size RO plants worldwide in recent years. Pre-heat RO is another concept likely to further reduce the water cost from RO plants. Fig 6 shows the likely reduction in applied pressure in RO with increasing temperature of seawater of different salinities (35,000 to 45,000 ppm) and for different product recoveries, ranging from 40 to 50 %. This results in reduced water cost due to savings in energy or membrane cost for a pre-heat RO plant (7). Some studies suggest the use of the condenser cooling seawater in a contiguous RO plant installed in a nuclear reactor can make the process even more economical.

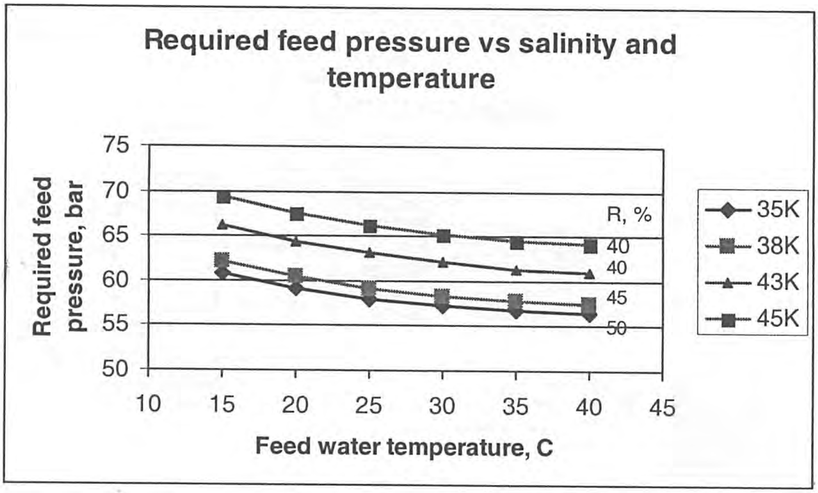

Fig.6: Required feed pressure vs. salinity and temperature

The water reuse after treatment of domestic effluents for agricultural and other uses is also being considered in recent years to augment the water resources for non-potable applications. These would bring further challenges on the economics of nuclear desalination, although care must be taken on the difference in the quality of water produced in the two cases.

Hybrid desalination systems are known to produce two qualities of water, which can be blended to make desired quality of potable water without any chemical addition. These plants have also the advantage of combined pre-treatment of seawater and the post treatment of product water. There also exists the advantage of pre-heat RO in hybrid systems. The life of the RO membranes could be augmented for a UF-RO hybrid system. These advantages are likely to improve the economics of seawater desalination

The possibility of waste heat utilization from the PHWRs is being investigated in India for seawater desalination. Nearly 100 MW (th) of moderator waste heat is available in a 500 MWe PHWR. Research is being conducted to develop thermal processes, which can utilise such low-grade waste heat. The efficiency of these desalination systems is however lower requiring larger heat transfer area and hence higher investment even though energy cost is minimal. Fig.7 shows a desalination unit set up in 2004 at the CIRUS research reactor, Trombay, India producing high quality water from seawater, meeting the reactor makes up water needs (9). Design of a large capacity desalination plant has been carried out for the proposed AHWR in India.

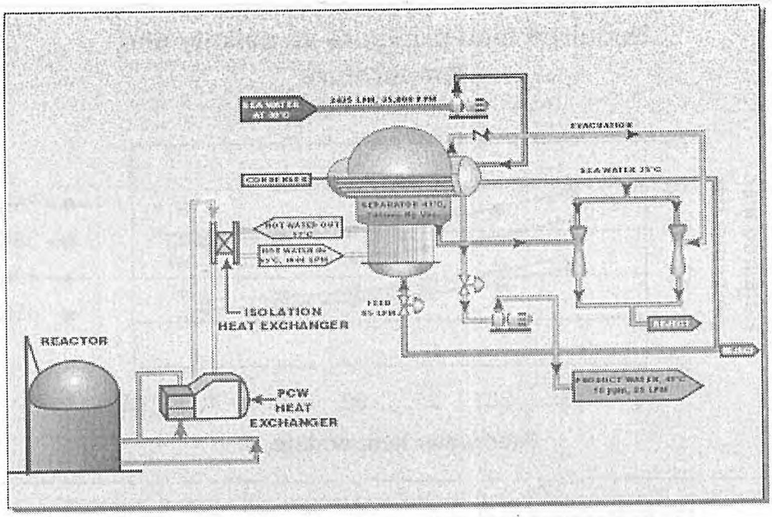

Fig. 7: Integrated LTE nuclear desalination system, Trombay (India)

The use of reject heat from High-Temperature Gas-cooled Reactor (HTGR) design, one of the candidate reactor designs well suited for economical desalination, is being investigated. While traditionally, emphasis has been on HTGR suitability for high-temperature applications such as hydrogen production, the potential of this plant design for low-temperature applications such as nuclear desalination is attracting. According to French study, 300MWth HTGR plant can produce 25,000m^3/day of water without sacrificing electricity production.

Water production cost estimated for a MED plant located at the proposed high temperature gas cooled reactor is given (10) in Table 7. It shows the possibility of reducing the water cost to half compared to the present costs.

Table 7: Estimated water production cost for a MED plant located at a HTGR

MED module cost (M$)	20
Annual water production (m^3)	6.935x10^6
Amortization rate (%)	8
Amortization (cents/ m^3)	23
Steam cost (cents/ m^3)	0
Electricity cost (cents/ m^3)	7
Chemical cost (cents/ m^3)	3
Operation & maintenance (cents/ m^3)	12
Total water production cost (cents/ m^3)	45

5. Challenges

The following lists up possible challenges facing nuclear desalination:

a) Disparity

Countries suffering from scarcity of water are, generally speaking, not the holders of nuclear technology and infrastructure for product water distribution. The utilization of nuclear energy in those countries will require infra-structure building and other institutional arrangements for such things as financing, liability, safeguards, security and will also require preparation for the fuel cycle including upstream and downstream. The concept of multi-national fuel cycle centre, as is proposed by IAEA, could be used to assure a supply of nuclear material to legitimate would-be users under control of sensitive parts of the nuclear fuel cycle.

b) Public perception

The design of nuclear desalination plants normally concern with various safeties related aspects. The possibility of radioactive contamination of product water, however, is a very important issue to be considered for the nuclear desalination plants. The dissemination of data from the existing co-generation facilities in many countries would go a long way to alleviate the concern and improve the public perception for the nuclear desalination plants. Sharing of relevant information in this area from Member States involved in nuclear desalination and co-generation facilities will be greatly welcome.

c) Socio-environmental aspects

The socio-environmental aspects of nuclear desalination need greater attention for its large-scale adoption. Setting up of desalination plants at nuclear reactors for providing the much needed freshwater to the public will no doubt add to its social acceptance. However, this calls for providing assured quantity and quality of the fresh water round the year. The intake/ outfall of the nuclear desalination plants are to be designed keeping in view the locals use of the area for fishing and other socio-cultural activities. Protection of the marine environment near the desalination plant site, particularly the flora and fauna, need to be considered in great length. This would lead to enhanced acceptance of the nuclear desalination plants. The use of the reject brine from the desalination plants for pisciculture or other uses such as production of useful minerals is a possibility worth consideration.

6. Conclusion

The availability of vast resources of seawater on earth has attracted interest in seawater desalination as a possible source of fresh water in the water scarce arid and semi-arid areas of the world. The present desalination capacity of about 36 million cubic meters per day worldwide meets a very small fraction of the world's fresh water needs. There is however a significant potential of desalination and water reuse technology for rapid expansion in the future to augment the fresh water resources in water scarce areas. Use of fossil fuels for the large capacity desalination plants could lead to large emission of undesired green house gases into the environment. The future price uncertainties of fossil fuels and their sustainability is also an important issue. Use of energy from nuclear reactors for desalination is a demonstrated option and it is an environment friendly and a sustainable source. Feasibility studies carried out recently indicate that present costs of water produced from nuclear desalination plants are similar to that of fossil fuel based desalination plants. Use of future generation nuclear reactors, which would produce large amount of available heat for desalination in addition to providing cheaper power, is likely to reduce the water cost substantially. Nuclear desalination will be an important option for safe, economic and sustainable supply of large amounts of fresh water to meet the ever-increasing world-wide water demand.

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Domestic Water Demand Management in terms of quality and Quantity in Gaza Strip / Palestine

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B.M.Misra

DOMESTIC WATER DEMAND MANAGEMENT IN TERMS OF QUALITY AND QUANTITY IN GAZA STRIP/ PALESTINE

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ABSTRACT

Gaza Strip is located on the extreme edge of the shallow coastal aquifer that borders the eastern Mediterranean Sea. There is little rainfall and no reliable riparian flow; hence water supply for Gaza residents (about 1.3 millions inhabitants) is limited to that available from the part of the coastal aquifer that underlies its 365 km^2 of land, of which 165 km^2 are still under occupation. Over exploitation of the coastal aquifer has resulted in continuous lowering of regional water levels and the worsening of water quality. The greatest threats to existing water supplies are seawater intrusions and up coning of deep brine fossil water. There are serious water quality problems in the Gaza Strip Aquifer. Less than 10 percent of the aquifer's yield is water meeting World Health Organization (WHO) drinking standards. The population in Gaza Strip will grow to over two million by 2020, and the demands for water will exceed the sustainable capacity of the aquifer. Continuous urban and industrial growth will place additional stress on the aquifer system, unless appropriate integrated planning and management actions are instituted immediately. It is evident that drastic action must be taken quickly to support its people in the future. In recognition of this worsening situation, the Palestinian Water Authority (PWA) and the United States Agency for International Development (USAID) have jointly developed and begun implementation for managing the domestic needs. This paper presents overall guidelines for the management until year 2020, with associated investment requirements for infrastructure facilities to meet all goals and objectives. It has been estimated that a capital investment program of about US$ 1.5 billion is needed to finance the implementation of such plans. It has been concluded that implementation of a domestic management plan through sea water desalination as well as brackish water desalination are the main components of domestic water management and that will have overall beneficial impacts on the socio-economic aspects.

1.0. Introduction

Water is the most precious and valuable natural resource in the Middle East in general and in Gaza Strip in particular. It is vital for socio-economic growth and sustainability of the environment. Gaza Strip is, in particular, facing a critical situation that requires immediate and concerted efforts to improve the water situation in the term of quality and quantity. Demand greatly exceeds water supply. In addition, water quality is very poor and the aquifer is being over pumped. Very limited water supplied for domestic use is potable. About 70% of the total pumped water is used for agricultural purposes.

This paper presents an overview of the water resources as well as the current and future water balance for the Gaza coastal aquifer through efficient and sustainable water resources management in terms of quality and quantity. Because the problem is expected to grow new resources to minimize the water deficit and to improve the ground water quality are needed. Producing additional water through desalination facilities becomes eminent. Through various studies, which tackle strategic planning for water resources, it is concluded that Gaza will need to develop both seawater and brackish groundwater desalination to meet the demand up until the year 2020.

2.0. Water Resources

Gaza's water resources are essentially limited to that part of the coastal aquifer that underlies its 360km^2 area (Fig.1). The coastal aquifer is the only aquifer in the Gaza Strip and is composed of Pleistocene marine sand and sandstone, intercalated with clayey layers. The maximum thickness of the different bearing horizons occurs in the northwest along the coast and decreasing gradually toward the east and southeast along the eastern border of Gaza Strip to less than 10m (Fig.2). The base of the coastal aquifer system is formed of impervious clay shade rocks of Neogene age (Saqiyah formation) with a total thickness ranging between 500 and 1000 m. Depth to the water level of the coastal aquifer varies between a few meters in the low land area along the shoreline and about 70 m along the eastern border. The coastal aquifer holds approximately $5x10^9$ m^3 of groundwater of different quality. However, only 1.4 $x10^9$m^3 of this is "fresh water", with chloride content less than 500 mg/l. This fresh groundwater typically occurs in the form of lenses that float on the top of the brackish and/or saline groundwater. That means that approximately 70% of the aquifer are brackish or saline water and only 30% are fresh water.

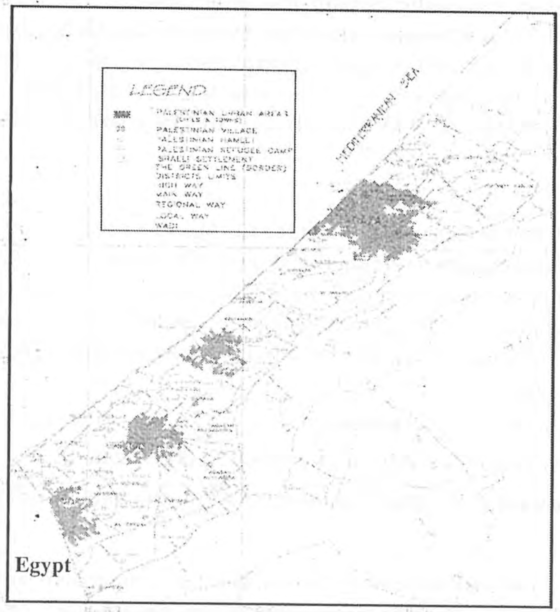

Fig.1: Location map of Gaza Strip

The major source of new groundwater in the aquifer is rainfall. Rainfall is sporadic across Gaza and generally varies from 400 mm/yr in the North to about 200 mm/yr in the south. The total rainfall recharge to the aquifer is estimated to be approximately 45 m^3/yr. The remaining rainwater evaporates or dissipates as run-off during the short periods of heavy rainstorms.

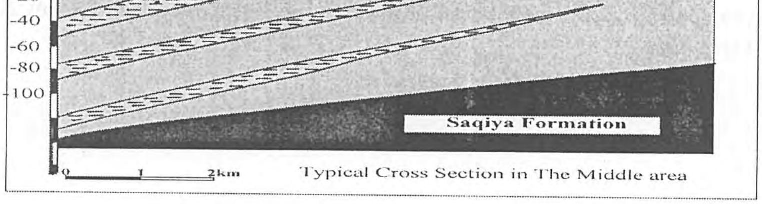

Fig.2: Typical hydrogeological cross section in the middle area of Gaza Strip

The lateral inflow to the aquifer is estimated to be between 10 - 15 x$10^6 m^3$/yr. Some recharge is available from the major surface flow (Wadi Gaza). But because of the extensive extraction from Wadi Gaza in Israel, this recharge is limited to, at its best, 1.5- 2x$10^6 m^3$ during the ten or 50 days in which the Wadi actually flows in a normal year. As a result, the total fresh water recharge at present is limited to approximately 56.5-62 x$10^6 m^3$/yr.

3.0. Water Balance

The water balance of the Gaza coastal aquifer has been developed based on the estimate of all water inputs and outputs to the aquifer system. The Gaza coastal aquifer is a dynamic system with continuously changing inflows and flows. The present net aquifer balance is negative, that is, there is a water deficit. Under the defined average climatic conditions, total abstraction and return flows, the net deficit is about 40-50 MCM/yr. Implication of the net deficit includes:

- Lowering of water level (documented).
- Reduction in availability of fresh groundwater (documented).
- Seawater intrusion (documented), and potential up-coning of deep brines (partly documented).

It is estimated that only 10 percent of the total aquifer volume may be considered fresh, meeting WHO drinking water standards. This corresponds to a total of about 500 MCM. The time frame for complete depletion of fresh groundwater will depend on continued abstraction volumes and patterns. Using a rate of aquifer depletion of about 40-50 MCM/yr, it can be theoretically calculated that depletion would occur in 10-13 years. The net deficit has led to a lowering of the water table in the past 30-40 years and inland migration of seawater. Of these two factors, seawater intrusion accounts for a greater fraction of the volume loss, but it is less visible and thus tends to lessen the perception of the worsening aquifer evolution.

4.0. Demand Projections

Mainly the population growth and socio-economic development control water demand for the different uses. The annual population growth rate in Gaza was recorded at 5.9 and 6.8% between 1980 and 1996, at the time Gaza had a population of 963,000 inhabitants.

Based on PCBS census Dec, 1997 Gaza's population was recorded as 1,020,080. Using a conservative growth rate of about 3.5% and assuming an influx of 50,000 returnees by 2010, the estimated population in 1999 was slightly over 1,100,000 and the population was forecasted to be 2,140,000 by 2020. That means that the population is expected tdouble after 20 years. Fig.3 shows population projections in 5-year increments from 2000 to 2020 for Gaza Strip.

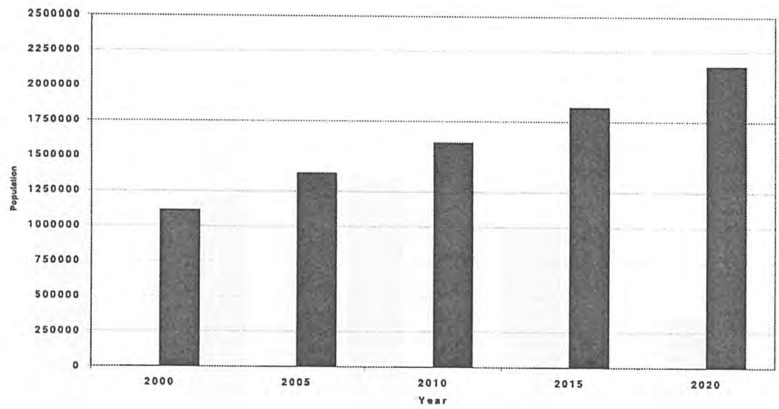

Fig.3: Population growth projection in Gaza assuming a growth rate of 3.5% and an influx of 50,000 returnees by the year 2010

In 2003, it was estimated that approximately $150x10^6$ m^3/yr of water was pumped from about 4100 wells of which about $90x10^6m^3/yr$ of water was used for irrigation and $60x10^6$ m^3/yr were pumped for domestic and industrial purposes from 100 municipal wells.

WHO recommends an average of 100 liters per capita per day (l/c/d) as a minimum standard for individual water use. In 2003, it was estimated that 80 l/c/d were actually made available to consumers. On the other hand, only about 13 l/c/d meet WHO quality standards. As social development occurs, the demand for water will increase to meet the average WHO recommendation of 150 l/c/d in future years.

These facts make it evident that the Gaza Coastal Aquifer is in extreme danger of becoming unusable for drinking water and irrigation. Over exploitation of the aquifer has resulted in salt water intrusion and continuous decline in groundwater levels has been observed in most of the areas of Gaza Strip since the mid-1970s. The ability of the aquifer to sustain life for the increasing population and a basic agriculture industry will be destroyed in twenty years if no action is taken.

4.1. Domestic and Industrial (D&I) Water Demand

Population growth, the changing water needs of households and industry and changing demands of agriculture will shape in the future (D&I) water demand.

The projected (D&I) demand for the next 20-years is graphically presented in Fig.4.

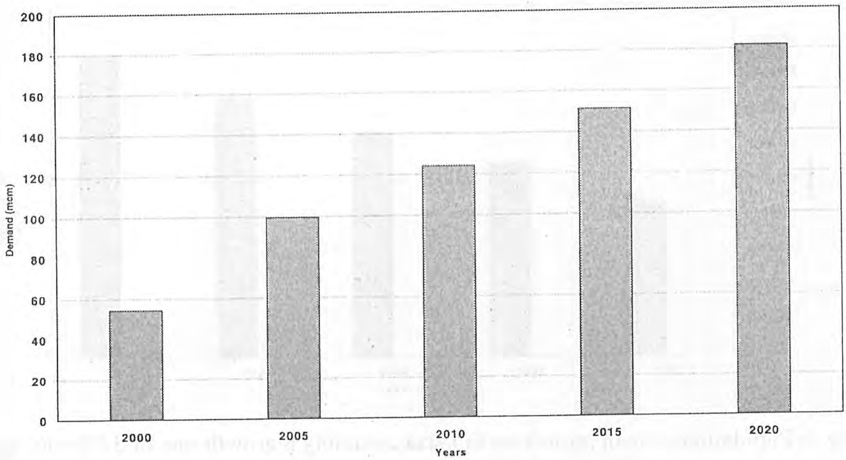

Fig.4: Projected Domestic and Industrial Water Demand until the Year 2020.

The per capita consumption is assumed to reach 150 l/d and the population increase is in accordance with Fig.3. The D&I demand include net demand for domestic, industrial, public customers and livestock water supply. Water losses through transmission pipeline and water distribution system are included. Therefore, D&I demand presents the quantity of water at the water supply source that should be delivered to the D&I customers. It is clear that the total D&I water needs will reach to about 182 mcm by 2020 assuming an overall efficiency of 20%.

4.2. Agricultural Water Demand

If the demand for irrigation is calculated on the basis of the food requirements of the growing population, it appears that it will increase from the present usage of about 90 $x10^6 m^3/yr$ to $185x10^6 m^3/yr$ by2020. However, these figures are not realistic projections for Gaza, because neither the water nor the land to support an increase in agricultural activity exist. Therefore, the estimated future demands for agriculture are based on the actual water amounts of today. Fig.5 illustrates the continuing decreasing trend in the agricultural water demand reflecting the decreasing use of both irrigated and rain fed agricultural land area in Gaza.

That is occurring as a result of the growth of urban areas, which expand onto agricultural land. This encourages farmers to bring what had been marginal land into production. It also means that farmers are turning to more intensive methods of agriculture, which require expensive inputs.

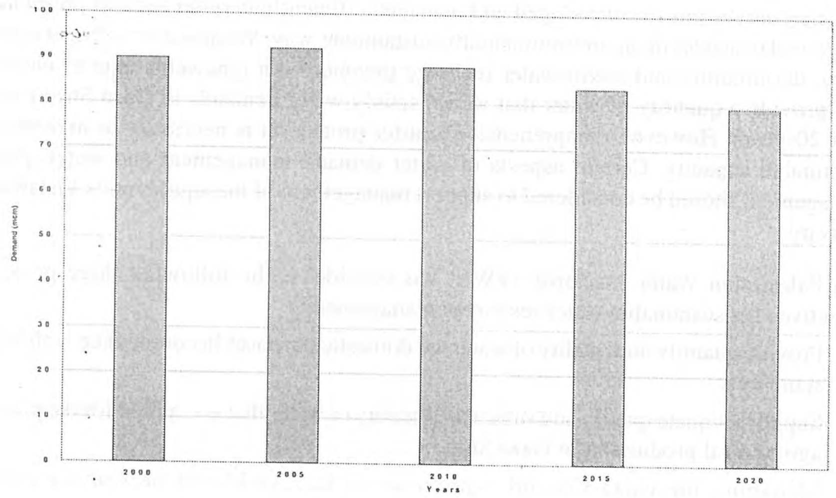

Fig.5: Projected Agricultural Water Demand in Gaza until the Year 2020

4.3. Overall Water Demand

Generally, the overall water demand in Gaza Strip is estimated to increase from about $150x10^6 m^3/yr$ to about $260x10^6 m^3/yr$ in 2020, as shown in Fig.6. This includes D&I demand at water supply sources and agricultural demand.

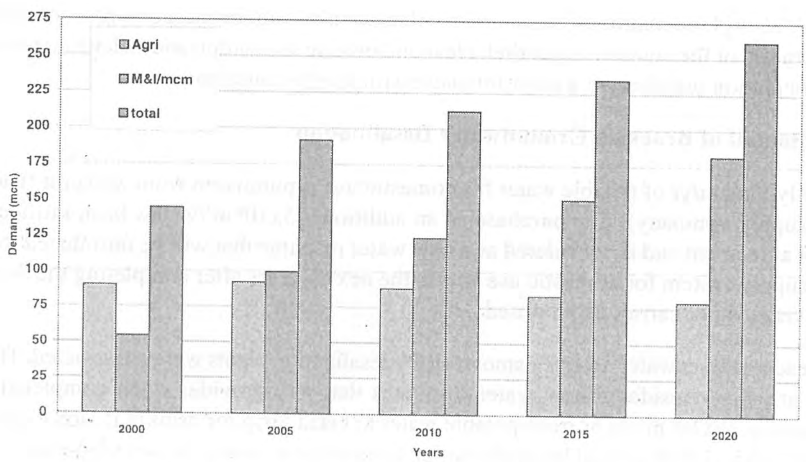

Fig.6: Overall Water Demand in Gaza until the Year 2020

5.0. Palestinian Water Resources Policy

Water resources must be developed and managed efficiently in order to meet present and future water needs, in an environmentally sustainable way. Wastewater reclamation and reuse, desalination and storm water recharge together with renewable aquifer capacity will provide a quantity of water that would satisfy water demands in Gaza Strip for the next 20-years. However, comprehensive aquifer protection is necessary to maintain its sustainable capacity. Certain aspects of water demand management and water quality management should be considered to support management of the aquifer at its sustainable capacity.

The Palestinian Water Authority (PWA) has considered the following three principal objectives for sustainable water resources management:

- Provide quantity and quality of water for domestic purposes in compliance with WHO standards.
- Supply adequate quality and sufficient quantity of water that is required for the planned agricultural production in Gaza Strip.
- Managing the Gaza Coastal Aquifer at its safe yield and preventing further deterioration of the aquifer water quality.

The accomplishment of these principal objectives is based on the following fundamental promises:

- Reclamation of wastewater and maximum use of the reclaimed water for agriculture.
- Introduction of new water resource(s) into the Gaza Strip water sector as soon as possible to meet the projected water demands.
- Improve pumped groundwater quality needed for domestic use by desalination facilities.

Successful implementation of these issues will maintain water balance and prevent further deterioration of the aquifer. In parallel, clear and precise legislation and strict water sector implementation policies are a must for successful implementation.

6.0. Potential of Brackish Groundwater Desalination

Presently 5 mcm/yr of potable water for domestic use is purchased from Mekerot (Israeli water supply company). The purchase of an additional $5x10^6$ m^3/yr has been ratified by Oslo-II agreement and is considered as a new water resource that will be introduced to the water supply system for domestic use within the next 3-years after completing the North-South Gaza water carrier as proposed.

Two beach well seawater reverse osmosis (RO) desalination plants were constructed. These plants are also considered new water resources that will provide, when completed, an additional $2.25x10^6$ m^3/yr of fresh potable water to Gaza Strip for drinking. This means, a total of 12.25 $x10^6$ m^3/yr will be available for domestic use during the next 3-4 years from both Mekerot and beach well desalination plants.

The available fresh groundwater resources in the Gaza Coastal Aquifer that can be used for domestic and fit with the recommended WHO guidelines in terms of water quality is in the range of 15% of the total aquifer capacity. At present, most of the domestic municipal water produced is far away from the acceptable level. Figs. 7&8 show both chloride and nitrate concentration of the pumped water from the municipal wells in Gaza Strip for domestic and drinking purposes.

As mentioned before the total projected domestic demand will reach about 182 mcm/yr by 2020. Considering the total of about 12.25 mcm/yr that will be supplied through Mekerot and beach well RO desalination plants. The remaining quantity required will be more or less $170 x 10^6 m^3/yr$. Considering the water balance as well as the groundwater yielding capacity, it has been assumed that a total $128 x 10^6 m^3/yr$ can be produced from the groundwater aquifer for domestic purposes by 2020. About 20% of the total domestic water demand ($37 x 10^6 m^3/yr$) can be produced from the aquifer and mixed directly with about $100 x 10^6 m^3/yr$ of treated or/desalinated groundwater through RO desalination facilities in order to fulfill domestic water compliance with WHO water quality standards. Fig. 9 shows the total domestic water demand projection to be produced from the aquifer and the groundwater desalinated quantity required.

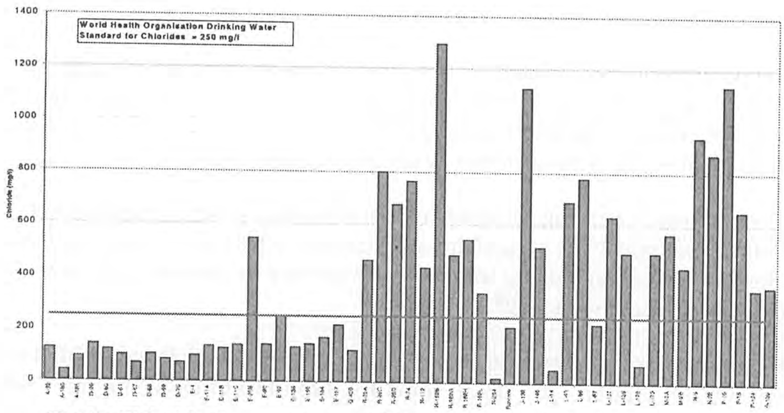

Fig.7: Chloride Concentration of Domestic Municipal Wells in Gaza

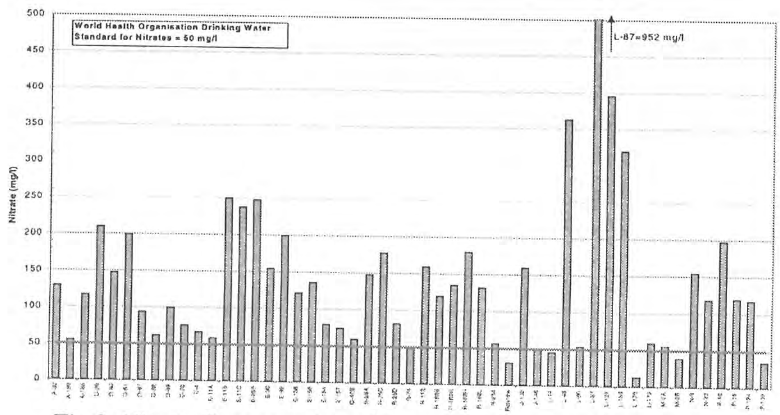

Fig.8: Nitrate Concentration of Domestic Municipal Wells in Gaza

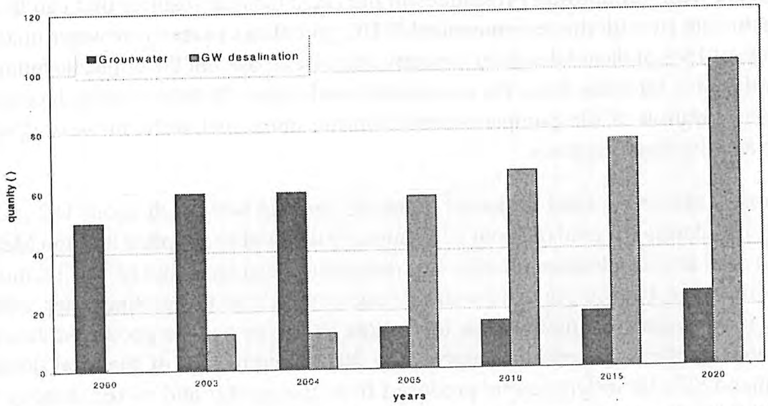

Fig.9: Originated Water (fresh & desalinated brackish groundwater) Supply Projection until the Year 2020

7.0. Prospect of Seawater Desalination

In order to maintain the water balance to the positive condition and to fulfill the domestic water demand in terms of quality and quantity, a new water resource should be introduced into the Gaza Strip water sector as soon as possible. These new water resources will relief stress on the aquifer and prevent further deterioration of its water quality.

With the assumption of efficient comprehensive wastewater management in terms of reuse the reclaimed wastewater for agriculture and recharge the surplus wastewater into the aquifer, the proposed water balance will be improved relatively and the total water deficit will be of about 40 mcm/yr in 2020 (Fig. 10).

Many alternatives have been examined to minimize the water deficit and fulfill domestic water demand, but seawater RO desalination has been identified as the most realistic option. Following this concept a large scale RO seawater desalination plant with four different production phases is proposed as follows:

- Phase-1: 60,000 m^3/d, in operation by 2004
- Phase-2: 60,000 m^3/d, in operation by 2008
- Phase-3: 20,000 m^3/d, in operation by 2014
- Phase-4: 10,000 m^3/d, in operation by 2017

The total desalination capacity will be about 150,000 m³/d (~55x10⁶ m³/y) by 2020.

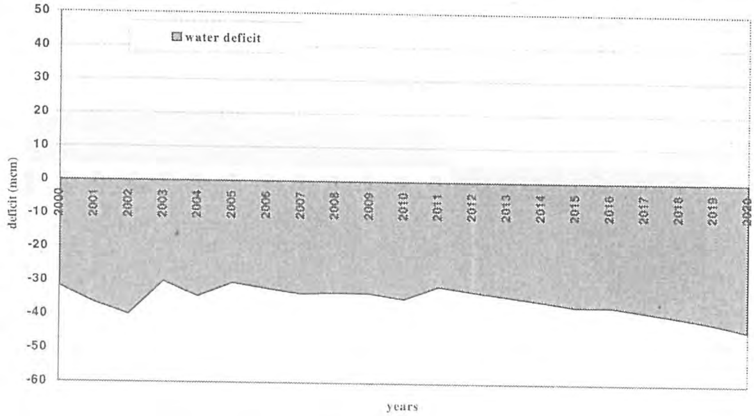

Fig.10: Water Deficit with Efficient Reclaimed Water Reuse and without Seawater Desalination

The total seawater desalination quantity in conjunction with the brackish groundwater desalination, Mekerot water supply and beach well desalination plants will be able to cover completely the domestic water demand (D&I) compatible with WHO standards in terms of quantity and quality. Fig. 11 shows the domestic water quantity that can be available from the different sources up to year 2020.

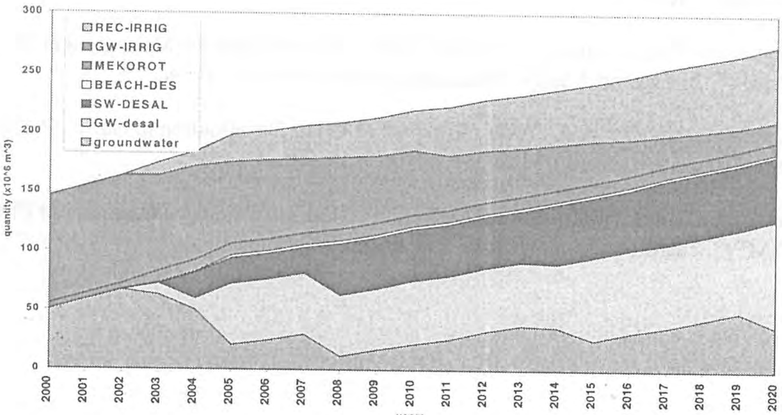

Fig.11: Projected Water Supplies for Various Demands in Gaza

With the option and/or assumption to construct a large seawater RO desalination plant as described earlier, the aquifer over drafting will decrease. As a result, it is expected that seawater will be pushed back (transgression) toward the sea preventing further deterioration of the aquifer water quality. Ultimately, an approximation of about 10 mcm/yr-aquifer water balance will be maintained starting by year 2008 as shown in Fig.12.

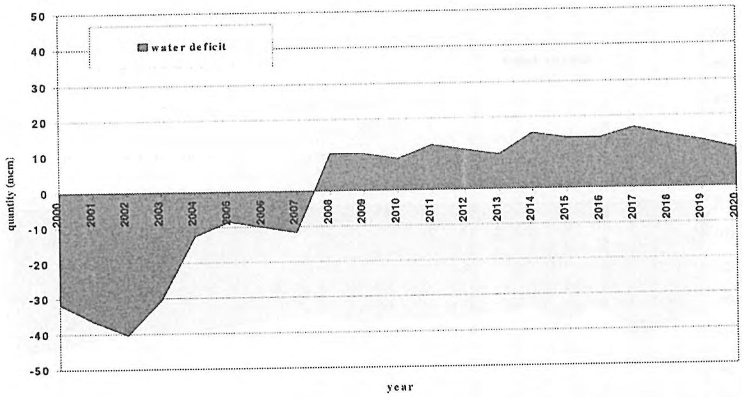

Fig.12: Water Balance with Seawater RO Desalination Plant and Efficient Reclaimed Water Reuse

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Reducing Unaccounted for Water Experiences in the Mega-City of Tehran

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REDUCING UNACCOUNTED FOR WATER EXPERIENCES IN THE MEGA-CITY OF TEHRAN

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ABSTRACT

Per capita water has been continuously decreasing as a result of population growth and increasing water consumption during the past 40 years. Tehran is facing a potential water crisis. The latest drought period started from 1998 and continued till 2001, during which the existing resources were progressively consumed and Tehran became 270 million m^3 water short in early 2001. The methodology used to achieve UFW reduction involved scientific research, many pilot studies and investigations, and selection of the most cost-effective and economic methods. Facing the intense limitations of water resources, reducing UFW is not only an economical solution, but it has also an inane value as a potential water supply resource. The experiences gained in this work would certainly benefit other nations as they lead to great savings on time and money. In this paper, these experiences and information on reducing UFW in the mega-city of Tehran, in which water plays a vital role, is illustrated in a clear form using figures, charts and necessary documents.

Keywords: UFW, Causes, Solutions, Tehran.

Introduction

Tehran is the largest city in Iran, having more than 7 million inhabitants and around 2 million commuters. It is also the capital of Iran and so it has a special significance both from the domestic and international aspects. In the mega-city of Tehran, for which over 920 million cubic meters of water are supplied annually, the costs of transferring, treating and monitoring of water are very high. Therefore the supplied water should not be wasted making the reduction of water losses an important issue. In this article the practical results of reducing Unaccounted for Water (UFW) in Tehran are presented. Heavy investments have been made for the city's water supply. In the past losses made up one third of the water supply. Initiatives to reduce these losses began 6 years ago after a large investment resulting in drop in UFW in the mega-city of Tehran from 33.3% to 30.1% during the 3rd development plan (2000-2004) [1]. Figures 1 and 4, show the drop in UFW in the province of Tehran and mega-city of Tehran.

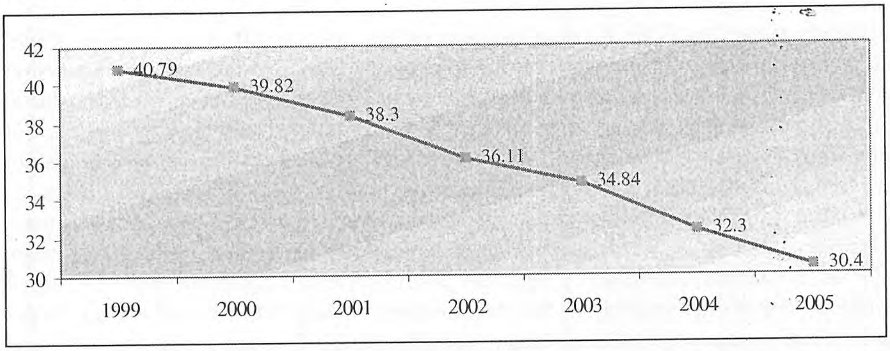

Figure 1: Progress chart of efforts to reduce UFW in the province of Tehran (1999-2005)

The main issues stressed in this article are:

- The importance of UFW in urban water supply, particularly in large cities (Tehran experience).
- The level of success of UFW reduction programs in Tehran.
- Solutions tested in successful practice and programs.
- Justification and stress on planning and implementing programs to reduce UFW and water losses.

Definition and Calculation of UFW

UFW in a supply and distribution system is equal to the volume of water produced in the system less the volume of consumption, either paid for or known. UFW is calculated as follows [2]:

$$UFW = S - (M + A \times P) \qquad (1)$$

Where:

S = Total of water produced in the system

M = Consumers

A = Coefficient of non-metered per capita consumption

P = Population coverage in persons

Types of UFW

UFW is divided into physical and non-physical groups. The physical group concerns the volume of water lost through leakage in the system, while the non-physical group is related to losses in revenues caused by inaccurate meters, illegal connections and non-payment of bills (by legal customers). Both groups are further divided into sub-groups, as shown in Figure 2.

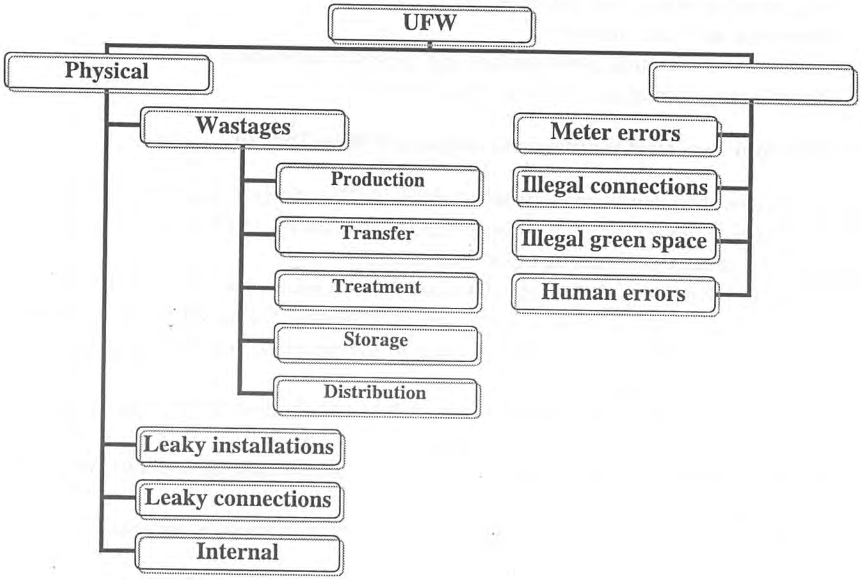

Figure 2: UFW types

As observed, UFW is present in various degrees, throughout nearly the whole supply and distribution cycle. Therefore it is necessary to undertake UFW reduction activities in every aspect of the cycle. However priority is given to distribution and customer systems (Figure 3).

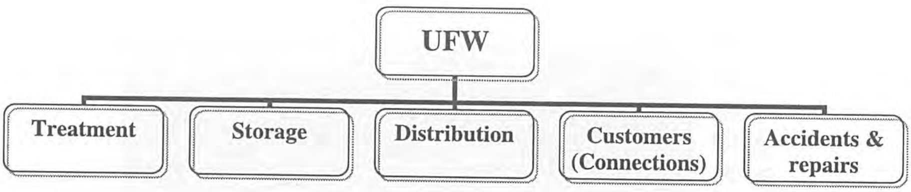

Figure 3: Relation of UFW cycle with systems

Causes and Reasons for UFW

Urban UFW is undoubtedly one of the costliest types of water, as it involves a lot of expenses. Therefore, for an effective and economical initiative towards an appropriate solution the identification of its reasons is a must. Some of the main reasons for UFW occurrence are presented below:

- The age of pipes, valves and other components and their leakage.
- Valves being either buried or broken down.

- Shortage of suitable and up to date maps of potable water distribution network.
- Lack of correct zoning to maintain water pressure equilibrium in the network.
- Management and operational problems of the network.
- Neglecting the reasons of repeated accidents.
- Diversity of network from the point of view of design and pipe material.
- Low quality joints and fittings in the network.
- Inappropriate pipes and valves in the laterals.
- Inappropriate water meters.
- Existence of marginal communities and illegal connections.
- Water losses in treatment systems (backwash).

Research and Practical Solutions to reduce UFW in Tehran

Due to successful measures in place since 1999, UFW is declining, dropping from 34% in 1999 to 30.1% in 2004 (Figure 4). This trend is expected to continue reaching about 20% by 2021. The most significant of these measures are:

- Undertaking pilot studies of the distribution network in six zones in Tehran.
- Setting up and equipping emergency posts to shorten the accident repair time.
- Maintaining appropriate water pressure in the network and adjusting the pressure zones to local altitudes.
- Replacing the old and broken pressure valves and the use of new diaphragm valves instead of weight operated pressure-reducing valves.
- Leakage detection along 9,300 km of the water distribution network in the mega-city of Tehran.
- Rehabilitation of worn-out segments in the water distribution network.
- Detection and cutting – off illegal connections.
- Replacing old and worn out Class B meters with new Class C ones (mainly of piston type).
- Installing precision instruments to measure the volume of raw water intake from surface and ground resources.
- Reducing water losses during production, transmission, treatment and distribution.
- Preparing the maps of installations in GIS environment.
- Unearthing and conducting valve maneuvers and maintaining the readiness of distribution network.

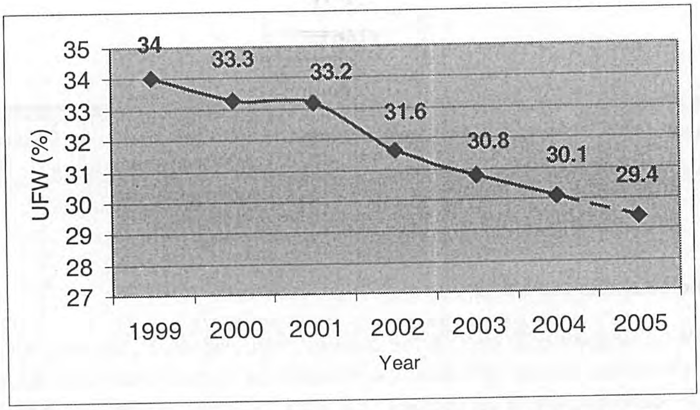

Figure 4: Results of initiatives taken to reduce UfW in Tehran (1999 – 2005)

Pilot Research to Reduce UFW

To make best use of time and adopt effective measures to reduce UFW in the mega-city of Tehran, which was over 34% in 1999, we opted for implementation of a pilot study to accurately identify the components of UFW and the effects of each one on the rate of water losses or the non-physical problems. This led to very interesting results, and showed us better methods with which to continue our works. For this purpose the mega-city of Tehran was divided into 6 zones. Taking into account the expanse of the network, which covers an area in excess of 51,000 hectares, as well as the variety in type and size of the pipes and the pressure difference in the network caused by changes in altitude in the city, the pilot plants were created with the objective of obtaining a more accurate picture of UFW in Tehran and identifying the best solutions to deal with.

The scope of the pilot plants was approximately 3,470 hectares, including 109,000 connections in total. The studies were undertaken by 6 research teams from 5 consulting engineering firms resulting in a very realistic outcome. Table 1 and Figure 5 show the assessment of UFW in the area covered by the six aforementioned pilots. It is noticeable that various pilot studies in all 6 different areas of the city of Tehran were selected based on the following criteria:

- Age of pipes (usually its more than 15 years).
- Type of pipes (ductile iron).
- Density of subscribers.
- Heavy traffic in the area.

Furthermore, from the geological point of view, the soil is different from one area to another; it varies from clay to fine sand to hard limestone.

Table 1: Assessment of UFW in six zones of Tehran in 2002

Type of activity	No. of cases	Average volume of water losses m^3/d	Ratio of UFW reduction per day (%)	Total volume in six months $10^6 m^3$
Replacement of meters	109,000	4,700	0.2	0.9
Valves	4,480	1,840	0.07	0.35
Leakage in network	360	5,760	0.25	1.1
Leakage from connections	600	7,800	0.31	1.45
Illegal connections	1,100	2,400	0.1	0.5
Green space connections	60	1,500	0.07	0.3
Total		24,000	1	4.6

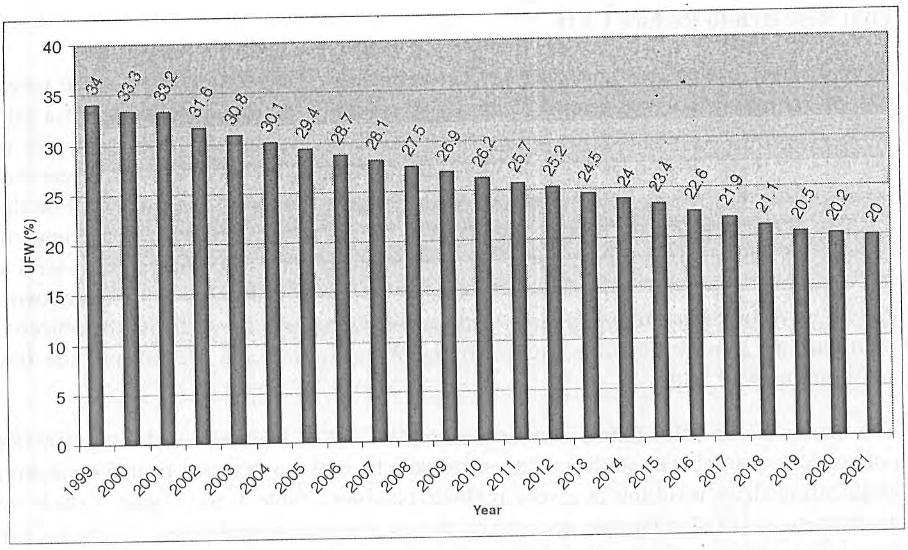

Figure 5: Predicting the trend of UFW in mega-city of Tehran (1999-2021)

Actions taken during pilot projects to reduce UFW

- Correcting the 1:2000 and 1:500 plans of the network and connections.
- Design and completion of the form on customer data and its transfer into a computer database.
- Site survey of distribution network with the objective of identifying valves and their correlation with maps.
- Survey of customers in the pilot zones according to consumption type and processing the generated data.
- Collecting data on accidents and mishaps in the pilot zones and inspection of the network pressure.
- Measuring the pressure in the network and preparing the co-pressure curves.
- Inspection and maneuvers of the valves for adequate operation, and measuring the incoming and outgoing flows to and from the pilot zone.
- Implementing leakage detection plans in the pilot zones.
- Dividing the pilots into different isolated zones.
- Monitoring the installation of meters at the inlet point of each pilot zone.
- Processing the water production and consumption in each pilot zone.
- Hydraulic analysis of the network using the collected data.
- Testing 10% of the meters in the laboratory and determining the error correction coefficient.

Calculation of non-physical losses included: applying the error correction coefficient for inaccurate meters, identifying illegal connections and faulty meters.

Calculation of physical losses included: detection of invisible leaks in the transmission line, the distribution network and the connections, as well as detecting small leaks such as from air conditioning systems and household faucets.

Necessary actions to reduce urban UFW

- Respecting the design standards.
- Monitoring the implementation of plans.
- Complying with technical standards while making installing connections.
- Improving the technical knowledge of operators.
- Implementing budgetary policies.
- Implementing leakage control policies.
- Applying operational standards.
- Enhancing meter precision and using quality materials.

Results of pilot projects

Based on the results of the six pilot researches on UFW in Tehran, the volume of physical UFW in the year 2000 was around 49% and the volume of non-physical UFW was approximately 51%. The studies showed that water losses in the distribution network and from connections composed the most significant parts of UFW. This finding had an important message, because prior to that, about 200 km of old pipes in the Tehran network were replaced annually with no clear results.

Therefore, it was decided to conduct a leakage detection program using precision instruments and then proceed with repair and rehabilitation of the network on the basis of leakage detection results. Moreover, the old water meters, which were over 7 years old and had exceeded the operating age limits, were included in the replacement programs. They numbered close to 650,000 throughout Tehran. Accordingly, the necessary actions for leakage detection and repair of the network as well as the exchange of ineffective and worn out meters began simultaneously.

The leakage detection in the 9400 km network of Tehran lasted for more than 3 years and about 5,500 major leakage cases and 11,000 leakage cases in total were detected and repaired. The total costs of leakage detection and rehabilitation programs were equivalent to 15.5 billion Rials (1.9 million USD). From a total of 650,000 old meters around 550,000 were replaced in a span of three years with better quality and class piston type devices (Class C). The total cost of these operations amounted to 140 billion Rials or equivalent to 16 million USD. In fact, the results of the pilots were used to prioritize time and investment for more effective actions. As for the network, the focus was on rehabilitation of areas, which had more technical weaknesses and therefore a higher rate of UFW, instead of concentrating on the age of the network.

Table 2: Summary of actions taken for leakage detection and exchange of meters 2000-2004

Description	Unit	Work amount
Network length	km	9,400
Amount of leakage detection from 2000 to 2004	km	9,400
Total number of major leaks detected in the network	Case	2,550
Total number of major leaks detected in the connections	Case	2,976
Total number of small leaks in the network and connections	Case	5,500
Total cost of leakage detection from 2000 to 2004	Thousand Rials	7,000,000
Cost of repair operations from 2000 to 2004	Thousand Rials	8,330,000
Number of connections in Tehran	Connections	860,000
Number of worn-out meters	Unit	650,000
Number of meters replaced	Unit	550,000
Cost of meter replacement	Billion Rials	140

Solutions to reduce UFW

Reducing UFW requires a number of different actions, heavy investment and a relatively long period of time. Therefore related programs should be implemented within the frame of short term and long term phases. Some of the programs that need to be implemented during the short term phase are:

- Updating the maps on distribution network, customers and technical records.
- Regular inspection of the network's main meters.
- Quantity and quality assessment of data on the volume of water produced, consumed and unmeasured.
- In situ testing of meters' precision after installment.
- Inspecting meters to ensure their diameter's compatibility with consumption flow, their location and their performance.
- Site visits to systems controlling the network and monitoring the methods of network operation.
- Adjusting pressure in the network, zonal meter reading at nights and regionalizing leakage detection activities.
- Controlling the repairs, accident and leakage records and upgrading software.
- Keeping a record of accidents in each pipeline.
- Weekly control of pressure reducing valves to repair and replace them when necessary.
- Replacing volumetric meters once every two years, and household meters once every five years.
- Color coding metering according to year installed.
- Maneuvering valves to ensure correct open and shut action, determining the number of turns needed and the direction for shutting them.
- Detection of illegal customers.

Long term programs are also necessary to sustain and supplement short term actions. The long term programs to reduce UFW include:

- Installing main meters in reservoirs and production resources.

- Creating facilities to ease the processes of measuring water production and consumption and testing meters.
- Creating facilities to isolate tanks for sealing and flushing tests.
- Designing the necessary valves and equipment at the inlet and outlet of the reservoirs and constant control of their water level.
- Installing sufficient valves to isolate pipelines in the network expansion plans.
- Creating facilities such as covers for main pipes for installing pressure control devices and flow meters.
- Undertaking water quality control test related to corrosion.
- Preparing and updating manuals on methods for installing and testing pipes, disinfection, installing meters, maneuvering ordinary and pressure reducing valves and their periodical replacement, repair and maintenance.
- Creating facilities for regional meters and installing valves to isolate the network into smaller segments.
- Installing meters of high standard in the network.
- Economical reviews of rehabilitation or replacement of the main pipes and laterals.
- Supply of fire hydrants in sufficient number in the network to assist its flushing (in addition to their main purpose).
- Supply and installation of modern state of the arts domestic and foreign equipment to control the quality and volume of water.

Conclusion

- Pilot research projects are recommended to reduce UFW.
- Unaccounted for Water is expensive and needs investment to be reduced. This matter in arid and semi-arid countries is equivalent to new water supply resources.
- Identification of priority measures and the use of appropriate technical tools are the main conditions in reducing UFW.
- The experience in the mega-city of Tehran proved leakage detection in the network and replacing old meters to be the main priorities in reducing UFW.
- Charging the real water price would greatly assist in reducing UFW.
- Reducing UFW is time consuming and costly (for example, in Tehran it can only be reduced by 15% in a span of 25 years).
- Exchange of world experiences on UFW, provides the means for more effective and economical measures in reducing UFW.

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An ARCGIS Database for Water Supply/ Demand Modeling and Management in Abu Dhabi Emirate, UAE

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Moll Macdonald, Mike Brook, Huda Al Houqani

AN ARCGIS DATABASE FOR WATER SUPPLY/ DEMAND MODELING AND MANAGEMENT IN ABU DHABI EMIRATE, UAE

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ABSTRACT

A strategic environmental goal of the Environmental Research and Wildlife Development Agency of Abu Dhabi is a management regime for water resources. The first stage of a water resources management plan development program includes the compilation of all data and information pertaining to water resources in an ArcGIS – SQL Server database containing supply and demand data and resource monitoring information for all sectors of water use. Demand data relates to water use for domestic, industrial, agricultural, forestry, and amenity purposes. Water sources comprise groundwater, desalinated water, treated wastewater, and water imported to the Emirate. A supply-demand balance model has been developed, linking demand centers to supply sources in order to predict future water shortfalls and surpluses. In this paper the development of the GIS/Database and the supply-demand model, and their use as an important tool for future assessment, monitoring, and management of water resources are described. The developed model was used to predict the future gaps between water demand and supply for any eastern and central region in the Abu Dhabi Emirate up to the year 2020. The results indicated an increase of about 100%, 132%, 136% and 269% in water demand due to future development in, agriculture, forestry, amenity and domestic sectors respectively

Keywords: GIS database, Supply-Demand Modeling, Water Resources Management, Water Demand Centers, Water Supply Sources, Groundwater Management, and Abu Dhabi Emirate.

1. Introduction

Water shortages and the degradation of water supplies threaten the development activities and health of people in many parts of the world. This is particularly true in GCC countries that are experiencing rapid population and development growth but have limited water resources and poor water resources management. Water management is the efficient and effective use of the water resource available by minimizing wastage, promoting recycling, and increasing water quality alongside sustainable economic development. Water management is a crucial issue to the survival of humans and all living things in the present era as it is a resource, which is getting scarcer. The amount of water we need and the availability is unbalanced. With proper water management, we could minimize the effect of drought and thus famine being faced by developing countries.

In order to manage water better it is crucial to have an inventory of the water available, how it is being managed, the drainage area, and the demand and supply of water. GIS has been extremely beneficial in mapping and data analysis, and thus greatly aids in the understanding and decision-making involved in water resource management. Linked GIS-databases and water resources supply/demand modeling plays a vital role in providing the detailed information for water resources and demand centers to develop the solutions for improving the efficiency of managing the available water resources. Abu Dhabi Emirate has invested heavily in GIS and asset databases, but until recently, these have not been in the field of water resources management and modeling. A GIS-water database together with an integral supply/demand model (SDM) has now been developed by the Environmental Research and Wildlife Development Agency (ERWDA), to provide decision makers with good quality, timely and reliable data. A well-designed GIS database can significantly reduce the time needed for data preparation and presentation during the modeling process. Recently, the use of GIS has grown rapidly in water resources assessment and management. The use of GIS in water management modeling is still at an early stage, but some successful applications have already been developed such as those done by Maidment (1994), Fedra (1994 and 1995), Furst (2001), Kharad et al (2002), Sarma and Saraf (2002), Singh and Prakash (2003), and others.

The ability to link GIS to water supply/demand models can also provide a useful tool, enabling rapid and accurate prediction for future water management and planning scenarios. Simulation for future water management and planning scenarios allow decision makers to estimate the required investments in the water sector without disrupting supply or affecting the progress in the various development sectors due to any unpredicted gaps in water resources (Ali et al., 2003).

Recently many researchers have concentrated on using GIS technology in mapping and data analysis, thus greatly aiding in the understanding and decision making in water resource management (Tremblay et al., 1994). Schultz (2000) discussed the use of Arc-GIS as a tool for water resources management through producing digital maps and digital elevation models which can be processed together with remote sensing and other data within GIS databases thus increasing the potential for working with multi-temporal imagery. In addition, he discussed how the combination of remote sensing with other information leads to new data types that allow integrated planning of water resources systems.

This current study is being undertaken to determine the water availability and demand in various development sectors. The main objectives of this study are:

- To develop an understanding, on a regional basis, of the relationship between the available water resources in the eastern and central regions, with an extension to cover the whole Abu Dhabi Emirate at a later stage.
- To develop projections of water supply and demand, including water demand for domestic, agriculture, forestry and industrial/commercial sectors, and the implications of inter-sectoral competition for water on the development in these sectors.
- To analyze the future alternatives for water supply and demand taking into account all the factors affecting the increase in water demand.
- Based on this analysis, to assess the impact of alternative water availability scenarios on water demand, taking into account policy reforms and investments in water and irrigation management.

  Since the mid 1960s, the Emirate of Abu Dhabi has undergone major development, underpinned by large oil revenues and the commitment of the former ruler Sheikh Zayed to agriculture and to policy of 'greening of the desert'. Pre-development, a small population relied entirely upon groundwater within superficial aquifers. In the east of the Emirate, fresh groundwater was exploited by shallow wells and by natural aflaj, for potable use and for traditional oasis agriculture; westwards, the aquifer contained brackish to saline groundwater. At that time, total abstraction did not exceed 200 Mm^3/y (including falaj flows) of which agriculture consumed 163Mm^3/y and forestry, less than 1 Mm^3/y (USGS, 1996). By contrast, year 2002 water use is estimated at over 3200 Mm^3/y (ERWDA, 2002). Reasons for this enormous increase in demand are:

- Very high per-capita potable water consumption. Potable demand is increasing by 8%pa concurrently with a 6%pa population growth;
- Continued expansion in the area under irrigation, comprising amenity planting, forestry and agriculture farms;
- Few, if any constraints on water use.

Demand now far exceeds the capacity of the superficial aquifer and Abu Dhabi must increasingly use new water sources, specifically desalinated Gulf seawater and desalinated water imported from Fujairah on the Gulf of Oman. Development has led to environmental concerns in particular the local over-abstraction from the superficial aquifer, aquifer salinization and possible aquifer contamination from chemicals used in the agriculture sector. The proliferation of desalination plants along the Gulf, and the Emirates increasing reliance upon desalination, leads to both environmental and supply security concerns.

In order to accomplish the research objectives, a water supply demand model has been developed that attempts to project and analyze how water availability and demand evolve over the next two decades (from a base year of 2000), taking into account the availability and variability in water resources, water supply infrastructure, and irrigation and non-agricultural water demands, as well as the impact of alternative water policies and investments on water supply and demand. The developed GIS-database and SDM allow ERWDA to store and interrogate water resource data, model and test existing and future water demand and water supply scenarios, and to contribute towards water policy formulation and the sustainable management of water resources.

2. Study Area Location

The location of the project area is shown in Figure (1). It comprises the Eastern & Central Regions of Abu Dhabi and is bounded by the Dubai Emirate in the north, by the Oman border in the east, and by the Saudi Arabian border in the south. The study area is both under the jurisdiction of the Al Ain Municipality (Eastern Region including Al Ain City) and the Abu Dhabi Municipality (Central Region including Abu Dhabi City).

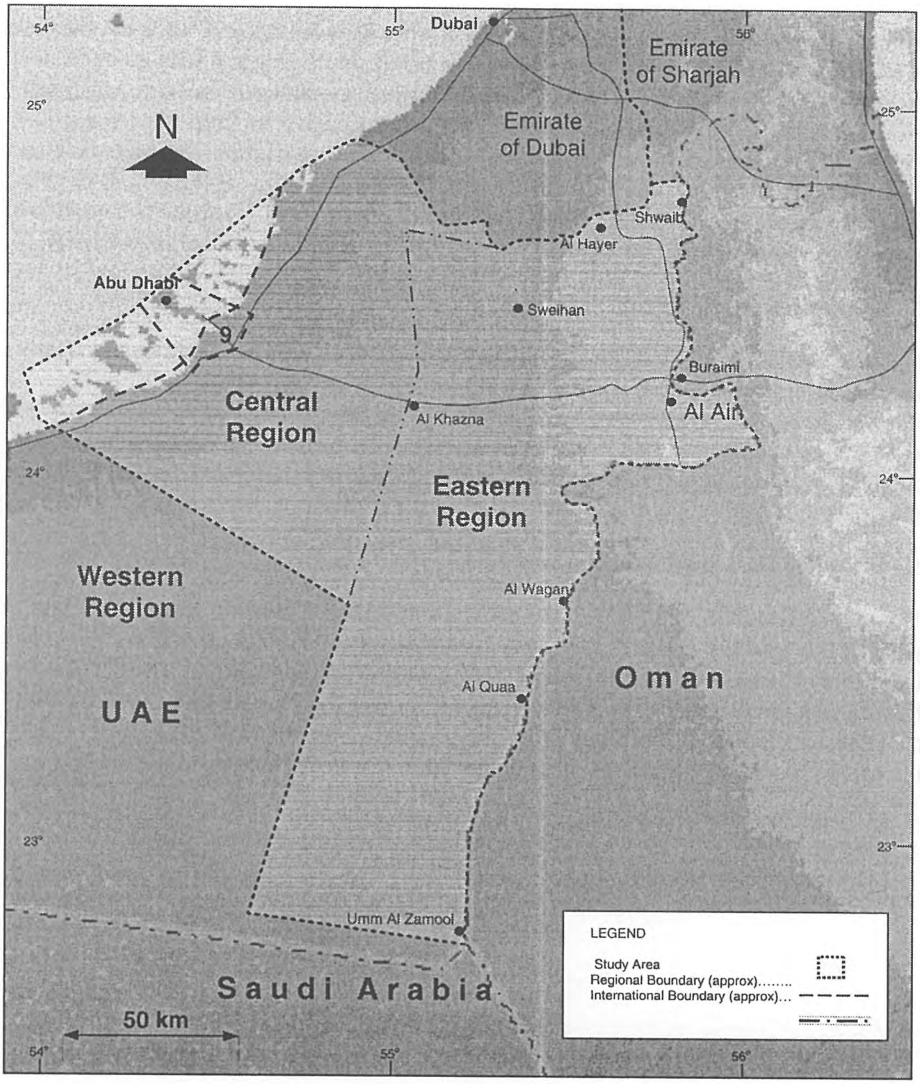

Figure 1: General Location Map for the Study Area.

3. Water Supply Sources

3.1 Groundwater

The superficial aquifer contains freshwater storage in the east of the Emirate while elsewhere, it contains brackish to saline groundwater. No regionally significant deep freshwater aquifers are known. The freshwater storage has long been over-exploited in particular by potable water well fields in the region of Al Ain; its use is increasingly constrained by falling well yields and increases in salinity. Brackish to saline groundwater is found in the shallow aquifer through much of the Emirate, and it is utilized for almost all irrigation of farms and forest. In the eastern region of Abu Dhabi, year 2002 farm well surveys (Mott MacDonald, 2004) show that agriculture is irrigated with mostly brackish water; of over 24000 farm wells sampled, 65% of wells have water of EC more than 6000 mS/cm while in some areas forests are irrigated with groundwater exceeding 30,000 mS/cm. This use of brackish-saline groundwater in farm and forest irrigation leads to increasing difficulties in soil/salt management, crop yield reduction and constraints on tree growth. Continued expansion of agriculture and forestry may require the introduction of desalinated water into the irrigation systems will clearly involve major infrastructure costs and government policy support. The total groundwater production from the eastern and central regions is about 1430 Mm^3/y.

3.2 Treated Sewage Effluent

The total treated sewage effluent production is currently about 87.3 Mm^3/y and this is all used in the irrigation of amenity and road verges plantations both in the cities and along major highways (ERWDA, 2002). USGS (1995) provided figures for TSE discharge at Al Ain up to 1994, starting in 1982. From 1995, TSE quantities were based on actual outputs for 2002 and assumed upgrades to the sewage treatment works in 2006, 2015 and 2020.

3.3 Seawater Desalination & Imports

The total desalinated water use in the eastern and central regions is about 365.0 Mm^3/y out of this quantity about 132.4 Mm^3/y is imported from Fujirah plant to Al Ain City. Desalinated seawater is largely used for domestic supplies and its current use in farming is limited. Potable water supply deficits in Al Ain will shortly be alleviated with desalinated supplies to be piped from the Fujairah I desalination plant (effectively imports to the area). Figure (2) presents the current water resources in the study area.

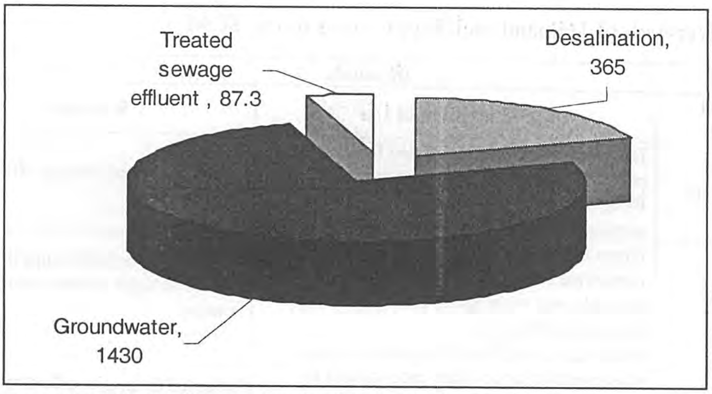

Figure 2: Available Water Resources in the Study Area.

4. Demands for Water

4.1 Domestic Demand

Domestic demand is determined from population and per-capita water consumption, both specified for individual years of the simulation period. Bulk demand for industries is not included as a separate demand centre but is located within the respective settlement and is specified as a daily rate for individual years as a bulk demand. Per capita demand is specified in l/d and demand is thus derived from:

$$DWS = POP * PCD * 10^{-6} + BULK \quad (1)$$

Where:

DWS : requested demand for water supply [tcmd]

POP : population

PCD : per capita daily gross water need [l/d]

BULK : bulk demand [tcmd]

Demand constraints relate to the capacity of the water supply distribution and storage system to deliver the requested demand to the customers. This constraint is expressed as a value [tcmd] for the years of the simulation period. It is normally equal the design capacity of the distribution and storage system. Table (1) illustrates the data specification. Current consumption in the domestic sector in the eastern and central regions is estimated at 450 Mm^3/y (ERWDA, 2002). Per capita consumption is very high according to world standards, between 300-500 liter/capita/day (ADWEA, 2001). Unmetered connections or illegal use and wastage can locally increase consumption to 2000-5000 liter/capita/day. The SDM will allow future policy options, where this level of consumption is constrained by engineering, metering or price measures, to be tested.

Table 1: Overview of Demand and Supply used in the SDM

Demands		
Demand Center	**Determination of Use**	**Remarks**
Settlements	Domestic demand determined from population and per-capita consumption. Bulk demand is determined as a daily average quantity	Links to treated sewage effluent are defined
Amenity	Gross demand is determined from water requirements for a mix of trees and groundcover vegetation, taking account of irrigation efficiency	Amenity is normally supplied from treated sewage effluent supply sources
Agriculture	Water use for different vegetation classes (representing up to three crop mixes) is based on net requirements, cropped area and irrigation efficiency and is amalgamated for individual farm units into Agriculture demand centers	Irrigated from groundwater sources but desalinated water is being introduced.
Forestry	Individual forest areas are grouped into forestry demand centers. Demand is based on forest water requirement, taking into consideration the maturity of forest trees and the efficiency of irrigation.	Irrigated from groundwater sources although use of desalination water may occur in future
Industry	Demand depends on the nature of the industry and its size. Demands are specified as average daily quantity for individual years	Sources of water include groundwater and desalination water
Supplies		
Supply Source	**Determination of Use**	**Remarks**
Groundwater	The supply source is defined as a group of individual wells (a well field). Installed capacity determines maximum output, while actual output is controlled by demand and by constraints described above.	Well fields serve Settlement, Agriculture and Forestry demand centers
Desalination	The design capacity of desalination plant(s) are user specified. Design capacity is constraint by age, while water delivery to demand centers may be reduced by leakage from conveyance systems	Individual desalination plants may be grouped into one supply source
TREATED SEWAGE EFFLUENT	The maximum output from a sewage treatment works is determined by its design capacity. Actual output is a function of sewage input and treatment process	Input to sewage treatment works is linked to Settlement water use
Imports	Import capacity is user specified for individual years	

4.2 Amenity

Amenity areas include parks, golf courses, road verges and central reservations. They comprise a mixture of trees and groundcover vegetation (grasses and flowers). In many cases these areas are irrigated with tertiary treated sewage effluent supplied from sewage treatment works. Demand is determined from net vegetated area, from the mix of trees and groundcover vegetation, from the crop water requirement (crop and climate dependent), the efficiency of irrigation and the extent of 'under irrigation'. The latter represents the moisture stress imposed on the vegetation and reduces crop consumptive use. The demand is calculated as follows:

$$DA = NAA * (PT * CUT + (100 - PT) * CUG) * (0.0001 / eff) \qquad (2)$$

Where:

DA	:	demand for amenity area [tcmd]
NAA	:	net irrigated area [ha]
PT	:	percentage of net area covered by trees
CUT	:	net water requirement for trees [mm/d]
CUG	:	net water requirement for groundcover vegetation [mm/d]
eff	:	irrigation efficiency

Current consumption in amenity, parks, road verges, and gardens in eastern and central regions is estimated at 219 Mm^3/y (ERWDA, 2002).

4.3 Agriculture

The water requirements for farm demand centers are calculated in a similar manner to forest areas, except that more crops are considered. From net crop consumptive use of individual crops, percentages of the crops within a crop mix and irrigation efficiency, the gross water requirement follows from:

$$DAG = \acute{O}\ \{Max\ (0,\ \acute{O}\ [(Qnet * p) * Area * (0.01 / eff)]\} \qquad (3)$$

Where:

DAG	:	farm water demand as the summation of individual farm units [tcmd]
Qnet	:	net irrigation requirement for individual crops [mm/d]
p	:	percentage of crop within crop mix [%]
Area	:	area of farm unit [ha]
eff	:	irrigation efficiency [%]

Up to three crops are allowed for each crop mix, with percentages of individual crops defined for each crop mix. In the last 35 years, there has been a major expansion in agriculture principally through development of non-traditional new farms or Citizens Farms. Over 72000 ha of farms are estimated to consume 1692 Mm^3/y or 61% of all water used in the Emirate, a 345 % increase on 1994 estimates (ERWDA, 2002 and Mott MacDonald, 2004). Water use is unregulated although it may be constrained by aquifer capacity.

4.4 Forestry

The water requirement for forests depends on the following:

- Area of trees cover (digitised from Landsat imagery)
- Net consumptive use of trees, depending on climatic factors, species and maturity of trees

- Gross irrigation requirement, which depends on:
  - Net consumptive use
  - Irrigation efficiency, which is a function of irrigation technique, frequency of irrigation, soil properties, leaching requirements, etc.

The net crop water requirement is specified as a function of maturity as follows:

$$DF = Ó\ (Max\ \{f1,\ Min\ [(year_i - year_0),\ N]\ /\ N\}*\ Qfmax\}\ *\ Area\ *\ 0.01\ /\ eff) \quad (4)$$

Where

DF : forest water demand as summation of individual forest units [tcmd]
f1 : factor determining minimum water requirement at early stage of forest
Qfmax : net irrigation requirement for mature forest [mm/d]
$year_i$: year for which gross water requirement is calculated
$year_0$: year when forest was planted
N : number of years when maturity is reached
Area : area of farm unit [ha]
eff : irrigation efficiency [%]

The demand for a forest unit is zero for years prior to year of first planting.

In the SDM only area and gross irrigation requirement is specified, both as annual values for each forest demand centre. This is necessary to accommodate both forest expansion and the variability of the factors controlling gross water requirement. Irrigated forest plantations have expanded from less than 250 ha in 1969 to over 250,000 ha at present, under a national policy of 'greening the desert'. The forests in eastern and central regions are estimated to consume over 124 Mm^3/y (ERWDA, 2002).

5. GIS Database Development

5.1 Design and Structure

The database design incorporates:

- Input of water related non-geographical or attribute data (such as names, designations, entity types and data specific to that entity. Entities considered include, for example, well field and borehole sources and a discrete irrigated area with a water demand).
- Input of water related time-series data for these entities (such as well water levels or demand changes in response to a changing population). All water-resource specific entities have a time component, which allows water demands and supplies to vary over the study period adopted.
- Links with ArcGIS to allow the spatial attributes of the entity (geographical position, shape etc), to be linked with attribute data for that entity.
- The means to represent links and link-constraints (typically pipeline conveyance systems) between water sources and water demand centers.
- User selection of specific supply sources and demands, and the ability to allow some supplies and demands (e.g. forests and farms) to be grouped into larger groups (e.g. forestry and agriculture), for ease of manipulation in the SDM.

- User selection of Global Settings, used within the SDM to calculate water use by irrigated crops, forest and amenity planting. These include farm/forest type defined by cropping/planting pattern, crop/tree water requirement and irrigation efficiencies.
- Output screens.

The overall structure and data flows across the Database-GIS are shown in Figure (3). Information held in the database is employed within the SDM and the database structure essentially mirrors the SDM whose structure is shown in Figure (4). Table (1) provides an overview of demand and supply incorporated in the model. The objectives of water demand and supply balance modelling are twofold, namely (a) to assess the current status of supply sources and demand centers, in terms of their current capacity to supply water (sources) and their current requirements for water (demand centers) and (b) to provide a planning tool to allow rapid judgment of the future impacts of changes in supplies and demands on the supply/demand balance.

Supply sources include groundwater, desalinated water and treated sewage effluent. *Demand centers* are user-defined areas of defined type that require a quantified supply of water of a required quality from linked supply sources. The demand centers included are listed in Section 4. For the chosen assessment area, supply to demand links are defined, generally on the basis of the existing or planned conveyance systems.

5.2 Rules and Constraints related to the SDM

The following rules and constraints are built within the database and the SDM:

- The SDM uses yearly time steps with both supply and demand quantities expressed as average daily totals for the respective years.
- There are constraints associated with the links between supply sources and demand centers. For example, treated sewage effluent supply sources cannot be linked with Settlement, Industry and Agriculture demand centers.
- Constraints can be attached to both demand centers and supply sources. They may, for example, include the capacity of a distribution/storage network in a Settlement demand centre (constraint linked to the demand centre), or the reduction in output from a well field when brackish groundwater requires desalination (constraint linked to the supply source).
- Constraints may, in reality, also relate to transmission facilities between supply sources and demand centers. Wherever possible, however, these constraints are linked to either demand centers or supply sources. Constraints are summarized in Table (2).

5.3 Model Output

The content of a SDM scenario output screen is user specified; typically it will comprise a GIS map of the supply sources and demand centers examined in the scenario, tabulated results and a graphical output summarizing forecast supply-demand variations over the chosen time period. Figure (5) displays the results of a modeled scenario for a demand centre, the city of Al Ain. The figure is annotated to show the salient features of model output. A priority can be assigned to demand centers and this affects the allocation of water from linked supply sources. The concept of prioritization is illustrated in Table (3).

Table 2: Demand/Supply Constraints

Demand Centre	**Potential Constraints**	**Remarks**
Settlement	Distribution and storage network	It is likely that the distribution network and storage facilities have been designed to a specified capacity.
Supply Source	**Potential Constraints**	**Remarks**
Groundwater	Pump capacity	Output cannot be more than the capacity of the pumps.
	Q/h relationships	Pump capacity (Q) is a function of pumping head (h) and generally reduces when the pumping head increases.
	Groundwater levels	Groundwater levels directly affect the pumping head. If levels continue to drop, the pumping level in the well may drop below the intake level of the pump, thus reducing output.
	Water quality	Water quality may constrain the use of the water for certain purposes or require treatment, thus losing output in the process.
	Environmental constraints	These could include groundwater level constraints related to environmentally sensitive areas.
Sewage Treatment Works	Treatment capacity	Output cannot exceed the treatment capacity of the works.
	Wastewater supply	Output is constrained by the quantity of wastewater supplied.
Desalination Plants	Treatment capacity	
Imports		Imports relate to water supply from outside the project area

The example indicates an Al Ain requirement of 409 tcmd in 2002, although this is constrained by the capacity of the distribution system to 300 tcmd. Demand is met from five supply sources (two well fields, 2 desalination plants and imports from Fujairah). In 2002 and 2003, a supply deficit is evident while desalinated water import from Fujairah contributes to a notional oversupply for year 2004. The contribution of groundwater from the Al Ain well fields is shown to reduce to zero over the period 2002 to 2007, as desalinated water replaces problematic well field sources. The deficits and oversupplies can be minimized by use of the SDM optimization routine or by manual intervention by reallocation of supply. The increase in constrained demand (to 500 tcmd in 2004) reflects distribution system improvements.

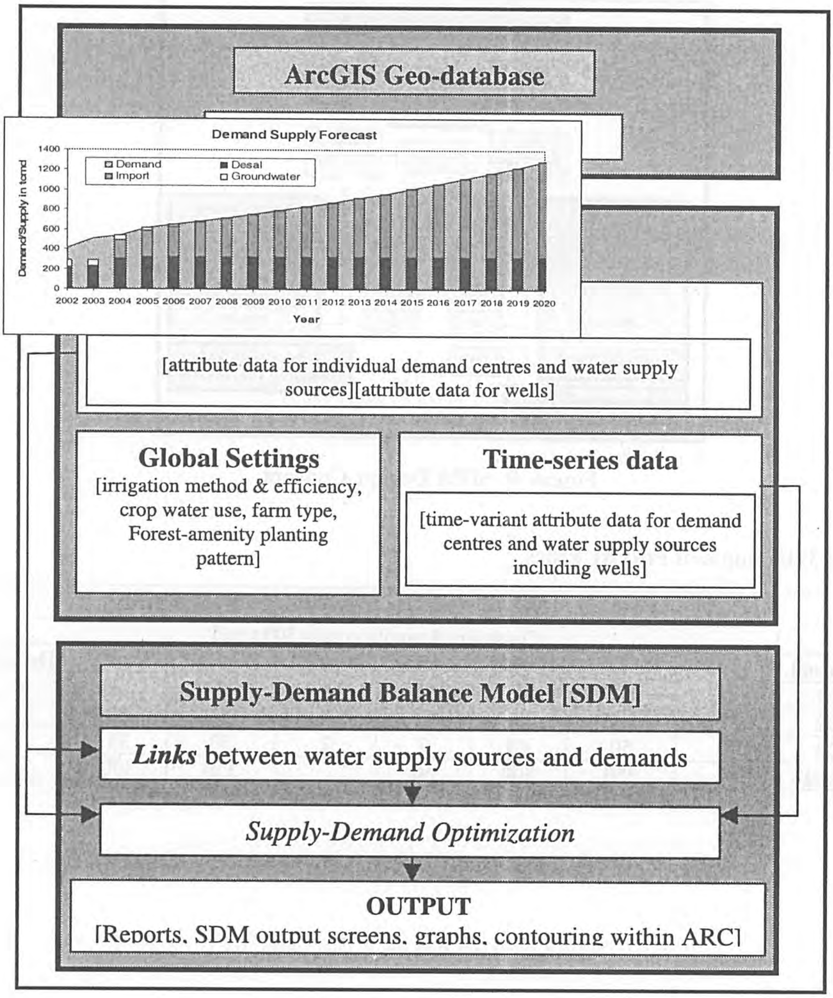

Figure 3: Database-GIS Structure

6. Model Application

The model has been applied to estimate the water demand for eastern and central regions of Abu Dhabi Emirate by year 2020. Table (4) shows the modeled results for year 2002, 2010, 2015 and 2020. The results indicated an increase of about 100%, 132%, 136% and 269% in water demand due to future development in agriculture, forestry, amenity and domestic sectors respectively. This increase in water demand will increase the pressure for using the desalination water in agriculture and amenity sectors, possibly blended with existing brackish groundwater. The increase in amenity planting water demand can be met from expansion in TSE output.

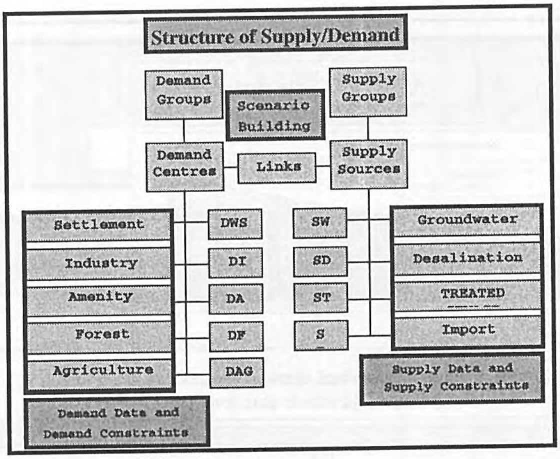

Figure 4: SDM Design Concept

Table 3: Example of Priority Rules

	Without priority				With priority			
	Constrained supply equals 300 tcmd							
Demand	Priority	Demand	Supply	Deficit	Priority	Demand	Supply	Deficit
D1	1	200	171	29	1	200	200	0
D2	1	100	86	14	2	100	67	33
D3	1	50	43	7	2	50	33	17
Total		350	300	50		350	300	50

Table 4: Modeled Increase in Water Demand for the Eastern and Central Regions of Abu Dhabi Emirate by year 2020.

Year	2002			2010	2015	2020		
Sector	Area [ha]	Present Use [Mm^3/y]	Groundwater Pumpage [Mm^3/y]	Predicted Use [Mm^3/y]	Predicted Use [Mm^3/y]	Area [ha]	Predicted Use [Mm^3/y]	Groundwater Pumpage [Mm^3/y]
Agriculture:	59,807	1,692	1,430	2,453	2,728	130,050	3,385	1,310
Forestry:	59,000	124	115	211	250	112,000	288	139
Amenity	6,480	219	57	344	423	15,320	518	57
Domestic	n/a	451	22	772	1,135	n/a	1667	0
Totals	**125,287**	**2,486**	**1,624**	**3,780**	**4,536**	**257,370**	**5,858**	**1,506**

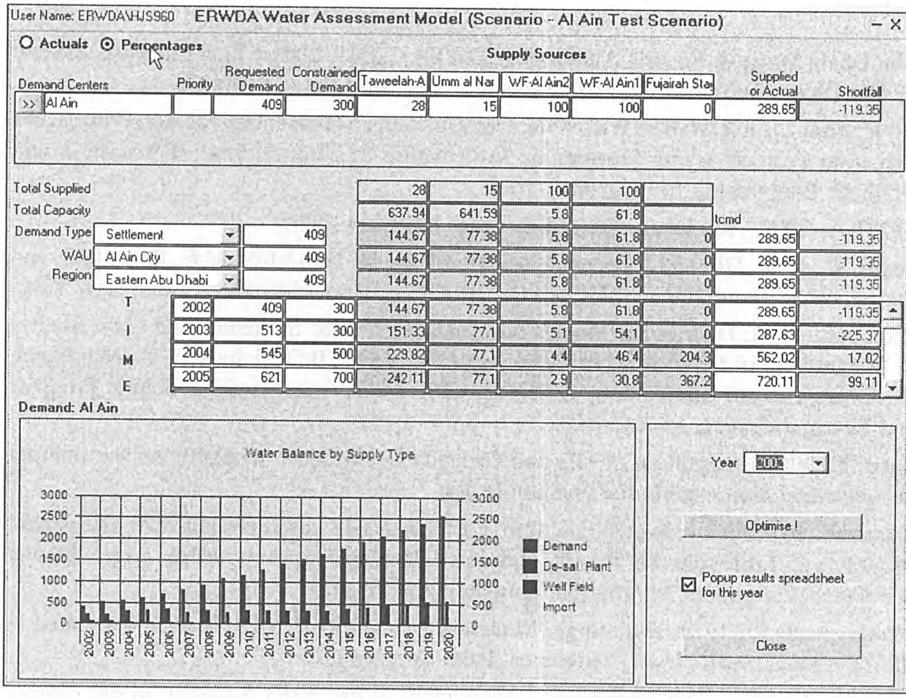

Figure 5: Model Output Screen

7. Conclusions

The Database-GIS and SDM was developed in the period 2002-2004. ERWDA uses this tool in resource data acquisition and assessment, resource modelling and the development of a water resources strategic management plan for Abu Dhabi. Despite major development over the last 35 years, little systematic collection of resource data has occurred; in particular, very little is known of irrigation water use yet over 70% of water use is by the agriculture and forestry sector. The assumptions embedded in the system (in particular the Global Settings) will demand revision by ERWDA as data improves. Nevertheless, the tool is flexible and will allow any rational supply-demand scenario to be tested. System use, and calibration with reliable data, will inevitably point to data uncertainty and elements which need field investigation and measurement.

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Annex 1: List of Acronyms and Abbreviations

ADWEA	:	Abu Dhabi Water & Electric Authority
EC	:	electrical conductivity
ERWDA	:	Environmental Research and Wildlife Development Agency
GIS	:	geographical information system
Ha	:	hectare
lcd	:	litter/capita/day
Mm^3/y	:	million cubic meters per year
SDM	:	supply-demand model
tcmd	:	thousand cube meters per day

Simple yet significant: water auditing as demand management tool

A. Fadlelmawlat and E. Azrag

SIMPLE YET SIGNIFICANT: WATER AUDITING AS DEMAND MANAGEMENT TOOL

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ABSTRACT

Water auditing can be a significant tool in any water conservation scheme. In efforts to demonstrate its benefits, the Kuwait Institute for Scientific Research has conducted an audit of its own water consumption. In this paper, the efforts and results of the water audit are presented. The audit was conducted at campus level with emphasis and detailing of consumption at the main building (MB). A total of 49 water meters were installed to subdivide the system and direct readings of consumption. The total daily consumption of the campus was estimated at 866 m^3. The main consumers at the campus recorded daily consumptions as follows: the Green Houses area 370 m^3, the Old Area 52 m^3, the Water Resources and Agriculture building 45 m^3, and the Tissue Culture building 38 m^3. The calculated input to the MB according to the water balance is 361 m^3. Direct measurements and water balancing for the MB indicated the following consumptions: the restrooms are the main consumers with 130 m^3/d, the cafeteria with a significant 51 m^3/d, the management floor consumed 22 m^3/d and the chillers only 4 m^3/d. Balancing the input against the consumption, the laboratories consumptions and the unaccounted water was estimated at 30 m^3/d. Leakage losses at the different segments of the supply system and within the MB were estimated by analyzing the consumption temporal trends and/or direct measurements of consumption during minimum activities periods. While the indoors leakage losses were not alarming, the losses from the storage tanks and the pipelines were estimated at 36% of the total campus consumption. Savings are foreseen in maintenance of the green houses' humidifiers, replacement of main pipeline, recycling at the Tissue Culture facility and installation of aerators and pressure regulators. The last two saving opportunities were perused further where a conceptual design of the recycling system was prepared and pilot application of aerators and pressure regulators were carried out, both of them proved to be very promising. Overall, the efficiency of water auditing as a demand management tool is manifested in a recommended program of water saving that has potential reduction of up to 46% of the campus consumption.

Key words: Water saving, Leakage losses, Water balance, Recycling.

Introduction

The acute lack of natural water resources in Kuwait is indisputable. With essentially limited groundwater resources as the only natural water source, the majority of the country's demands are met by seawater desalination putting an ever-increasing burden on the Kuwaiti economy. Ironically, analysis of the water utilization in Kuwait clearly indicates that water consumption per capita is excessively high (MOE, 2004; Fadlelmawla and Al-Otaibi, 2004) indicating that an aggressive demand management plan should significantly improve the balance in favor of the resources. One fundamental element in demand management planning is water auditing (PUB, 2002).

Water auditing is conducted at a variety of scales; national, facilities, or households. At the national scale the Working Group on Environmental Auditing (WGEA) of the International Organization of Supreme Audit Institutions (INTOSAI) has identified water quality, rivers and lakes, flooding, drinking water and sanitation, and marine environment as the issues of national audits (INTOSAI, 2004). Overall, the main purpose of these audits is to help raise the consciousness towards the relevance of water problems and needs and to improve the performance of the responsible programs or institutions in solving these problems.

At the scale of facilities or households, auditing the water consumption is essentially similar to financial auditing. In such an audit one tries to balance the incoming water (from the main water supply, water trucks, pumping from wells, etc.) against that utilized. Through this process, water auditing identifies the losses/leaks and provides ways to reduce them. It also gives an overall view of the water allocation within a facility, which enables improving the water utilization efficiency through process modifications, reuse, usage of water saving devices, etc. Such a level of water auditing has become one of the essential elements of national water conservation plans in many countries. While households auditing can only be encouraged, in Singapore (PUB, 2002), facilities using 5,000 m^3/month are considered large consumers and are subjected to obligatory regular water audits. It is regarded as a service provided by the government (since it gives the consumers solutions to reduce their consumptions hence reduce their expenditures on water); nonetheless, penalties may be given to consumers failing to install the mandatory equipment (Singapore mandates the usage of reduced flushing toilets, for example, in public facilities). Other countries such as the USA and Canada though do not obligate auditing; they highly encourage it and promote it as an excellent way to reduce water expenditures. Also businesses and products are labeled as environmentally friendly based on reduced water consumption. In this regard, large consumers often hire consultants to carry out such audits.

In this paper, the experiences and results of an internal water audit conducted by the Kuwait Institute for Water Research (KISR) are presented. The study was conducted with the aim of setting the example and demonstrating the benefits of water auditing.

Materials and methods

Campus scale water supply system: Figure 1 is a schematic presentation of the freshwater network from its input at the main gate till it branches out of the main building to the rest of the campus. The fresh water is delivered to KISR in a 4-inch diameter pipeline. Inside KISR's premises the freshwater pipeline branches into two lines. One line connects to the main building while the other line supplies the green houses area (GH). The line feeding the main building supplies its water to an underground equalization tank then to the basement. The line then branches into three other lines, the first leaves the main building to feed the Water Resources and Agriculture Building (WRA), the second also leaves the main building to feed the Tissue Culture Department building (TC), and the third branch proceeds into the main building (MB) supplying the different needs of the building then leaves the building to feed the rest of the campus or what is commonly known as the old area (OA).

MB distribution network: The MB main line runs in the raceways of its basement with six risers that supply the water to the different areas at the ground and first floor of the building. The internal piping system of the MB consists of various diameters piping that include ½", ¾", 1½", 2", 2½", 3" and 4". The most common are the ¾", which are used for the cold and hot water supply to the rest rooms, cafeteria and laboratories.

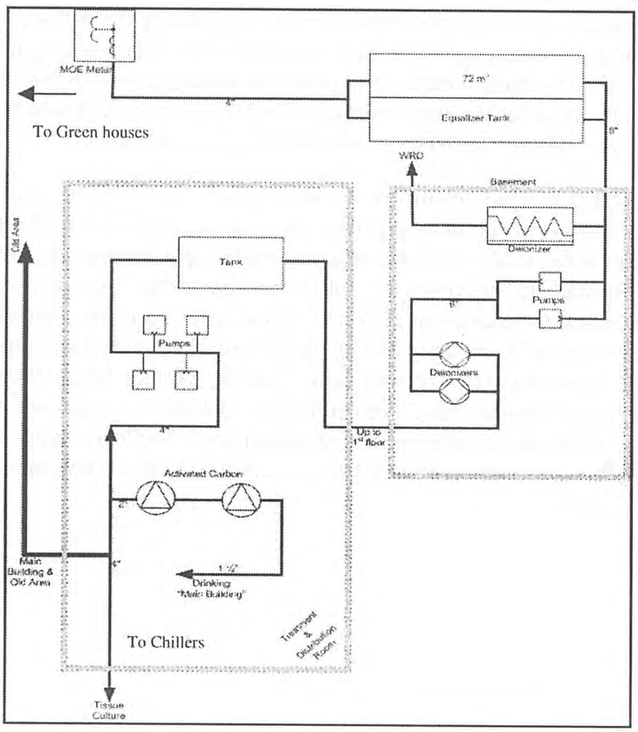

Fig. 1: Schematization of inputs and outputs to the main buildingTo Chillers

Gauging the system: The consumption at the campus scale was divided into five areas in accordance with the supply lines namely GH, WRA, TC, MB and OA. In addition to the Ministry of Energy (MOE) meter recording the total input to KISR, four mechanical displacement propeller meters of various diameters (1½", 2", 2½", and 3") were installed on the lines feeding the GH, WRA, TC and OA. Since it does not have its own supply line, the MB consumption had to be estimated from water balance. On the other hand, a total of 45 mechanical displacement propeller meters of ½" and ¾" diameters were installed to subsection the activities of the MB. Those activities are categorized as rest rooms, laboratories, cafeteria and the chillers.

Water balance formulation: A balance between inputs and outputs for the whole campus was constructed. The balance was used for losses estimation. The balance equation is expressed as follows:
MOE meter – Water Resources and Agriculture (WRA) building meter – Old Area (OA) meter – Tissue Culture (TC) building meter – Green Houses and workshops (GH) meter = Consumption at the main building (MB) + losses at the MB + losses in pipeline between MOE meter and MB + losses from equalization tank and treatment units.

Losses estimations: The main principles underlying the approaches used for losses estimation is that leakage losses are a continual process with minimum variations and that losses are less or equal to the recorded flows during minimum activities periods. Since the activities (i.e. not the total consumption of the building) of the MB is comprehensively gauged, direct measurements of flows for essentially each of the significant water consuming activities was possible. Additionally, activities of the MB are mostly during the working hours, which emphasize that recorded flows after working hours would be mostly leakages. Accordingly, flows were recorded during periods of minimum activities (after working hours and during the weekend). The leakage rate was taken as the minimum recorded flow rate during the two above mentioned periods.

Having the losses at the MB quantified, the water balance was used to calculate the losses in the buried portions of the main pipeline between the main gate gauge (MOE gauge) and the MB. These losses also include leakage from the equalization tank and the treatment units. Bulk estimations of losses for individual buildings were based on temporal consumption trends. Consumption after and during working hours were estimated from consumption records at 9 am, 2 pm and again at 9 am of the next day. The consumptions were averaged as rates over two periods namely during working hours (9 am to 2 pm) and after working hours (2 pm to 9am). The continuous 24-hours activities were identified for each building and its consumption measured or estimated. The losses were then calculated by subtracting the consumption of the 24-hours activities from the minimum consumption after working hours.

Results and discussions

Inputs and outputs at campus scale: Table 7 shows the various components of the water balance. According to this balance the water consumed at the main building along with the associated losses is estimated at 361 m^3/day.

Percentage wise, it is clear that most of the input water to the campus is consumed or lost at the stretch between the main gate to and within the MB and at the GH area. The meters recorded that 84% of the total input is consumed or lost at those two areas. The WRA, TC and OA consumptions were very close ranging from 4.3 to 6%.

Table 7: Balance of input and output over the Showuikh campus

Line	Volume discharged in 24 hours (m^3)
Inputs	
Ministry of Energy meter	866 (measured)
Outputs	
Water Resources and Agriculture (WRA) building meter	45 (measured)
Old Area (OA) meter	52 (measured)
Tissue Culture (TC) building meter	38 (measured)
Green Houses and workshops (GH) meter	370 (measured)
Consumption at the main building (MB) + losses at the MB + losses in pipeline between MOE meter and MB + losses in tanks	361 (calculated)

Analysis of MB consumption and losses: At the MB the extensive gauging enabled the detailed recording of consumption. The consuming activities were categorized as rest rooms, campus cafeteria, the chillers, management floor, and laboratories. Categorization was based on type of activity or access to piping system. Table 2 presents the average daily consumption per category for the MB.

Table 2: Daily consumption per category at the main building

Consuming category	Measured consumption (m^3)
Rest rooms	130
Cafeteria	51
Management floor	22
Chillers	4
Laboratories	30 (estimated)
Total	237

The twelve rest rooms of the building are responsible for 62% of the measured consumptions. The cafeteria has, also a significant share in the measured consumption, as its daily consumption is 22% of the total measured consumptions. The management floor consumption is unexpectedly high at 9% of the measured consumptions. The chillers recorded only 4 m^3/d due to an efficient closed circuit system for heat exchange, which results in minimum water losses (i.e. in comparison to the volume of circulated water).

Based on flow measurements during two different minimum activities periods, losses per room was estimated as illustrated in Table 3 and Figure 2. A leakage loss at the MB, which was detected in four locations only, is estimated at 13.5 m^3/d. This estimated leakage is about 6% of the total measured consumption at the MB.

Balancing the calculated lumped estimate of the MB consumption, losses in the MB and losses between the MOE and the MB against the measured values of consumption and losses at the MB, the losses in the line between the MOE meter and the MB, which also includes the ground tanks, are estimated to be 110 m^3/d. Having this last value estimated, Figure 3 presents an anatomy of the water flow to the MB.

Table 3: Leakage estimations at the MB

Room number	Leakage estimation (m^3)			
	During AWH	During weekend	Selected estimate	Estimated over 24 hrs
1112-1113	0.00	0.2	0.00	0
2005-2006	3.00	0	0.00	0
1012-1011	0.00	0.09	0.00	0
2011-2010	9.26	0.3	0.3 per 4 hrs	1.44
1259-1258	6.58	11.37	6.58 per 18.5 hrs	8.54
2259-2258	0.31	0.61	0.31 per 18.5 hrs	0.40
DG Right	2.44	2.92	2.44 per 18.5 hrs	3.17
DG Left	0.00	0.98	0.00	0.00
Cafeteria kitchen	7.20	0	0.00	0.00
Cafeteria toilet	0.80	0	0.00	0.00
Cafeteria basin	0.50	0	0.00	0.00
Cafeteria total	8.49	0	0.00	0.00
Total estimated leakage	30.08	16.47	NA	13.55

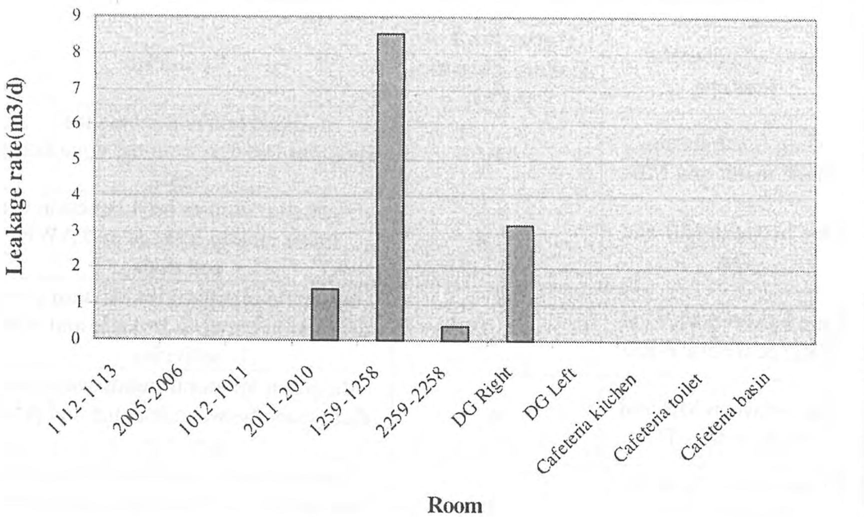

Fig. 2: Estimated leakages per room at the main building

Losses at campus scale: Analyzing the temporal trends in water consumptions, as described in the materials and methods section, the losses were estimated at the other buildings and areas. Table 4 summarizes the estimated leakage losses for the different locations of the campus.

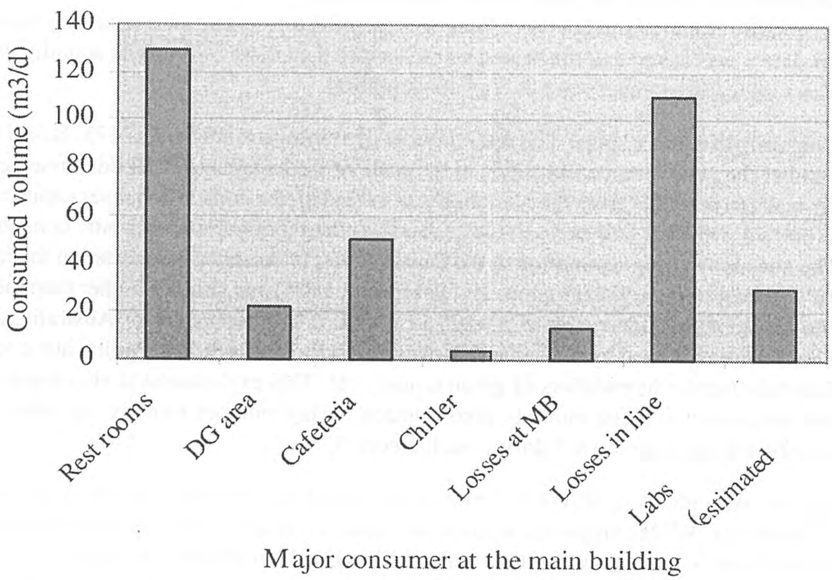

Figure 3: Anatomy of recorded flow to the MB

Table 4: Estimated leakage potential at various areas of campus

Area of potential leakage	Upper limit of leakage potential (m^3/d)	Remarks
Main line between MOE meter and MB	110	Based on water balance. Includes leakage from the equalization tank
Line between MB and OA	27.8	Overestimation as no distinction was made among leakage and AWH activities
Line between MB and WRD & within WRD	1.7	Based on temporal trends with good distinction between leakage and AWH activities
Line between MB and TC & within TC	9	Based on temporal trends with good distinction between leakage and AWH activities
Branch from main line to GH area & its connections in the area	205	Expected to be significantly less than this number as it includes unaccounted AWH consumptions
Within the MB	13.6	Based on the least measured consumption during different minimum activities periods

Excess energy consumption: One of the features of the piping system at the MB is the inclusion of loops for circulating the water from and to the boiler. The purpose of this loop is to maintain the temperature of the hot water in the pipes. Nonetheless, observations such as hot water in toilet cabinets indicated the presence of short-circuiting. The implication of this observation that water returning to the boilers for reheating is not necessarily coming form hot water lines or even going there again after reheating. Inspecting the data it was found that the heated water is more than three folds of the actually used hot water indicating unnecessary energy consumption.

Consumption per capita: The total number of employees in KISR is 673. Based on this number the per capita consumption at the scale of the campus is 1280 l/d. Nonetheless, if the leakage potential from the main line was excluded, the consumption per capita reduces to 600 l/d, which is still excessively higher than the reported non-domestic consumption. The non-domestic consumption in the United States of America is reported in the range of 40-400 l/capita/d (Tchobanoglous and Schroeder, 1985) and slightly higher than the upper end of the Australian range of 35-595 l/capita/d (Commonwealth of Australia, 2002) . This can be explained by the outdoor consuming activities such as irrigation at the research facilities and the humidifiers of green houses, etc. This explanation is also supported by the increase in average monthly consumption during summer months regardless of the significant decrease in staff during such a period.

The records indicate a total of 417 employees allocated to the MB. Assuming an additional 100 persons (KISR employees and outside visitors) are also using this building on a daily bases. The per capita consumption is about 399 l/d, which matches the reported maximum of the non-domestic use in the United States while compares well to the Australian range of the same category. It is noteworthy that the Australian range is more relevant to the climatic conditions of Kuwait.

Saving opportunities: The measurement and analysis of the water use in the campus highlighted situations that may be utilized in reducing water consumption. Table 5 lists those situations and the expected savings.

- The TC Department is utilizing a considerable volume of water in backwashing their deionizing resin system. Also their distillation units are working at a ratio of 1:9 of production to refusal. The estimated combined volume of water disposed off during those two processes is 43 m^3/week. In the mean time, the department is using more than 80 m^3/week in irrigation and other uses within the building. Accordingly, recycling this water may result in savings of about 30% of the department's consumption.

- About 90 m^3/day is used for cooling the green houses. Inspecting the site of the green houses, the humidifiers appeared to be in dire condition (Figure 4) with water falling out of the recycling trench causing losses in water. An investment in modern, more water efficient humidifiers should pay well in water savings at this area.

- Usage of water at the MB is taking place mostly at the rest rooms and the cafeteria. Tap water constitutes almost 50% of this usage. Also the water consumption in the laboratories is essentially through tap water. Hence, reduction in consumption of this category would result in considerable savings. The utilization of pressure regulators, aerators and similar water saving devices should have a significant impact on water consumption at the MB.

- Leakage from the equalization tank, the main line and its branch to the GH area proved to be very high. Eventhough, the estimates are rough, a potential of about 315 m^3/day, which is 36% of the total campus consumption, is significant even if overestimated. Excavation investigations followed by replacement of the identified stretches can have a major reduction in the consumption of the campus.

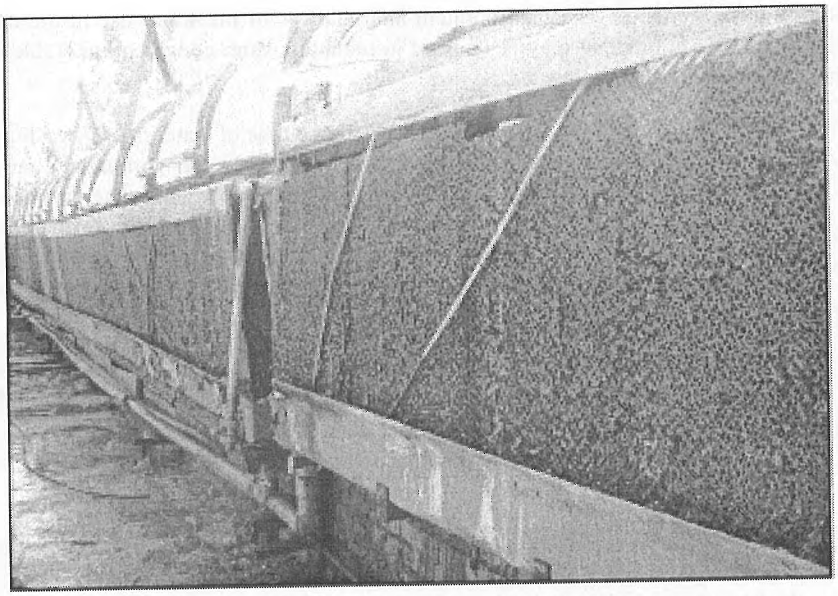

Figure 4: Apparent losses from the humidifiers

Table 5: Saving opportunities

Area/Situation	Recommended action	Saving potential	% Reduction in activity's consumption	Remarks
TC building	Recycling of deionized and backwash waters	7 m^3/d	30%	• Direct measurements • Solves septic tank flooding problems • Conceptual design is available
GH area	Replacement of humidifiers	18 m^3/d	20%	• Direct measurement/estimation
MB restrooms and cafeteria	Use of aerators and pressure regulators	60 m^3/d	30%	• Pilot application • Applicable campus wide
Main line between MOE meter and MB + equalization tank	Pipeline replacement and tanks maintenance	110 m^3/d	30%	• Water balance product • Additional metering should proceed excavations
Branch from main line to GH area & its connections in the area	Pipeline replacement	205 m^3/d	55%	• Water balance product • Involves considerable estimations and unidentified activities
Total		**400 m^3/d**	**46%**	

Conclusions and recommendations

A study on the water consumption at KISR's main campus was conducted. The study gave an assessment of the water consumption at the campus and its main consumers. Emphasis was put on the main building of the campus where the consumption was assessed in details. The measured average daily consumption of the campus is 866 m^3. The main consumers at the campus recorded daily consumptions as follows: the Green Houses area 370 m^3, the Old Area 52 m^3, the Water Resources and Agriculture building 45 m^3, and the Tissue Culture building 38 m^3. It is noticeable that the combined input to the MB and the GH area constitutes 85% of the total input to the campus.

Direct measurements and water balancing for the MB indicated the following consumptions: the restrooms are the main consumer with 130 m^3/d, the cafeteria with a significant 51 m^3/d, the management floor consumed 22 m^3/d and the chillers only 4 m^3/d. Balancing the input against the consumption, the laboratories' consumptions and the unaccounted water use was 30 m^3/d.

Leakage losses at the different segments of the supply pipeline and within the MB were estimated by analyzing the consumption temporal trends and/or direct measurements of consumption during minimum activity periods. While the indoors leakage losses were not alarming, the losses from the pipelines and the equalization tanks were estimated at 36% of the total campus consumption.

The efficiency of water auditing as a demand management tool is manifested in a recommended program of water saving that has potential reduction of up to 46% of the campus consumption. The recommended program includes recycling of disposal water, use of aerators and pressure regulators, maintenance of equalization tanks and pipeline replacement operations.

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WSTA Seventh Gulf Water Conference

Water in the GCC - Towards an Integrated Management

19 - 23 November 2005, State of Kuwait

DOMESTIC WASTEWATER TREATMENT & REUSE

Wastewater Treatment and its Applications as a Water Supply in Libya

E. Wheida and R. Verhoeven

WASTEWATER TREATMENT TECHNOLOGIES AND EFFICIENCIES. EXPERIENCES FROM EUROPE

Prof. Dr.-Eng. K.-H. Rosenwinkel and Dr.-Eng. Walid Abdel-Halim

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ABSTRACT

Liquid and solid wastes produced by human settlements and industrial activities pollute most of the watercourses throughout the world. The increasing scarcity of water in the world along rapid population increase in urban areas gives reason for concern and the need for appropriate water management practices. In the past, very little investment has been made on sewage treatment facilities; water supply and treatment often received more priority than wastewater collection and treatment. Inadequate sanitation is one of the prime causes of disease. In the developing countries, for example, the provision of sanitation is not keeping up with population growth. Currently, about 300 million urban residents have no access to sanitation and they are mainly low-income urban dwellers that are affected by lake of sanitation infrastructure. Approximately two-thirds of the population in the developing world has no hygienic means of disposing excreta and an even greater number lack adequate means of disposing of total wastewater. However, currently there is a growing awareness of the impact of sewage contamination on rivers and lakes; wastewater treatment is now receiving greater attention from a lot of international organizations and government regulatory bodies. The greatest challenge in the water and sanitation sector over the next two decades will be the implementation of low cost sewage treatment that will at the same time permit selective reuse of treated effluents for agricultural and industrial purposes. Developers should base the selection of technology upon specific site conditions and financial resources of individual communities. Although site-specific properties must be taken into consideration, there are core parts of sustainable treatment that should be met in each case such as: No dilution of high strength wastes with clean water; Maximum of recovery and re-use of treated water and by-product obtained from the pollution substances (i.e. irrigation, fertilization); Application of efficient, robust and reliable treatment/conversion technologies, which are low cost (in construction, operation, and maintenance), which have a long lifetime and are plain in operation and maintenance; Applicable at any scale, very small and very big as well; Leading to a high self-sufficiency in all respects; Acceptable for the local population and comply with the regulations and standards. This paper presents the different operation procedures of some of municipal wastewater treatment plants in the North of Germany, the analyses of the available dimensioning, and operation results compared with the specific energy consumption. The paper also discusses and demonstrates the anaerobic technology as a cost-effective pre-treatment technology as a potential of comprehensive concept of wastewater treatment and re-use.

Keywords: Sustainable Treatment Systems; Municipal Wastewater; anaerobic; aerobic; European experiences.

1. Introduction

A supply of clean water is an essential requirement for the establishment and maintenance of diverse human activities. Water resources provide valuable food through aquatic life and irrigation for agriculture production. However, liquid and solid wastes produced by human settlements and industrial activities pollute most of the watercourses throughout the world.

The increasing scarcity of water in the world along rapid population increase in urban areas gives reason for concern and the need for appropriate water management practices. Very little investment has been made in the past on sewage treatment facilities; water supply and treatment often received more priority than wastewater collection and treatment. However, due to the trends in urban development, wastewater treatment deserves greater emphasis.

In developing world, around 300 million urban residents have no access to sanitation and they are mainly low-income urban dwellers that are affected by lake of sanitation infrastructure. Approximately two-thirds of the population in the developing world has no hygienic means of disposing excreta and an even greater number lack adequate means of disposing of total wastewater (Rose, 1999).

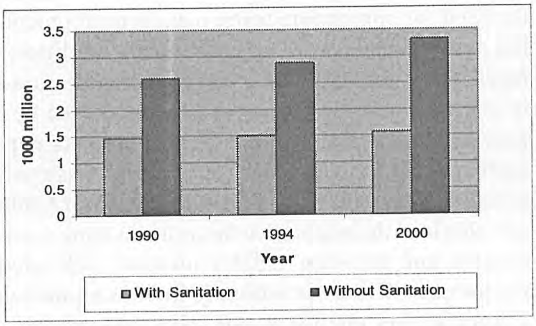


Figure 1: Population with and without sanitation, all developing countries. Source (WHO, 1997)

Inadequate sanitation is one of the prime causes of disease. In the developing countries for example, the provision of sanitation is not keeping up with population growth. The total numbers of population with and without sanitation in all developing countries are shown in figure 1 (WHO, 1997).

Currently there is a growing awareness of the impact of sewage contamination on rivers and lakes; therefore, wastewater treatment is now receiving greater attention from a lot of international organizations and government regulatory bodies.

The greatest challenge in the water and sanitation sector over the next two decades will be the implementation of low cost sewage treatment that will at the same time permit selective reuse of treated effluents for agricultural and industrial purposes. Developers should base the selection of technology upon specific site conditions and financial resources of individual communities. Although site-specific properties must be taken into consideration, there are core parts of sustainable treatment that should be met in each case such as: No dilution of high strength wastes with clean water; Maximum of recovery and re-use of treated water and by-product obtained from the pollution substances (i.e. irrigation, fertilization); Application of efficient, robust and reliable treatment/conversion technologies, which are low cost (in construction, operation, and maintenance), which have a long lifetime and are plain in operation and maintenance; Applicable at any scale, very small and very big as well; Leading to a high self-sufficiency in all respects; Acceptable for the local population and comply with the regulations and standards.

Because of recent regulations in the Federal Republic of Germany and in the countries belonging to the European Union, and also due to increasing eutrophy in both North Sea and Baltic Sea, the most imminent ecological problems are the elimination of oxygen consuming carbon compounds and the reduction of nutrient salt loads. Since the end of the 70s, several methods have been developed to biologically eliminate nitrogen and phosphorous from the wastewater. Parallel to an increase to the performance of wastewater treatment plants, the minimum limits for the effluent quality of wastewater treatment plants were raised.

Table 1: Minimum limits for the effluents from municipal wastewater treatment plants in the FRG, to be kept in qualified samples (valid since 27.08.1991), and demands of the EU, to be kept in the 24h mixed sample proportionate in regard to amount and time, (*) for „sensitive areas", (**) for „normal areas"

Minimum limits FRG					
Population equivalent	COD [mg/l]	BOD [mg/l]	NH_4-N [mg/l]	$N_{min.}$ [mg/l]	$P_{tot.}$ [mg/l]
5.000 - 10.000	90	20	10	-	-
10.000 - 100.000	90	20	10	18	2
> 100.000	75	15	10	13	1
Requirements EU-Standards					
Population equivalent	COD [mg/l] (**)	BOD [mg/l] (**)	NH_4-N [mg/l]	$N_{tot.}$ [mg/l] (*)	$P_{tot.}$ [mg/l] (*)
10.000 - 100.000	125 or 75 %	25 or 70 -90 %	-	15 or 70 - 80 %	2 or 80 %
> 100.000	125 or 75 %	25 or 70 - 90 %	-	10 or 70 - 80 %	1 or 80 %

2. Sustainable technologies for wastewater treatment and reuse

In order to achieve ecological wastewater treatment, a closed-loop treatment system is recommended. Many present day systems are a "disposal-based linear system". The traditional linear treatment systems must be transformed into the cyclical treatment to promote the conservation of water and nutrient resources. Using organic waste nutrient cycles, from point-of-generation to point-of-production, closes the resource loop and provides an approach for the management of valuable wastewater resources. Failing to recover organic wastewater from urban areas means a huge loss of life-supporting resources that instead of being used in agricultural for food production, fill rivers with polluted water. The development of ecological wastewater management strategies will contribute to the reduction of pathogens in surface and groundwater to improve public health. "The goal of ecological engineering is to attain high environmental quality, high yields in food and fiber, low consumption, good quality, high efficiency production and full utilization of wastes"(Rose, 1999).

There is currently a wide variety of systems, which can be successfully applied for wastewater treatment. They should however be selected on the basis of the specific local context. Generally, in industrialized countries the number of suitable alternatives may be more limited due to stricter regulations. In developing countries, however, the number of choices may be higher as a result of the more diverse discharge standards encountered. In this sense, effluent standards vary from the very conservative to the very relaxed. Likewise, the cost component and the operational requirements, while important in industrialized countries, play a much more decisive role in industrializing countries. Also the high contrast between urban and rural areas is an important feature.

Table 2: **Important factors in the selection of wastewater treatment system in developed and developing countries (von Sperling, 1995)**

Factor	Developed countries		Developing countries	
	Critical	Important	Critical	Important
Efficiency	x			x
Relaibility	x			x
Sludge disposal	x			x
Land requirements	x			x
Environmental impacts		x		x
Operational costs		x	x	
Construction costs		x	x	
Sustainability		x	x	
Simplicity		x	x	

The selection of wastewater treatment systems must based on important aspects such as efficiency, reliability, sludge disposal etc. A comparison of the most important aspects in the selection of wastewater treatment systems had analysed in the context of both developed and developing countries (table 1). It shows that in developed countries the critical items are: efficiency, reliability, sludge disposal and land requirements, whereas in developing countries the critical items are construction costs, sustainability, simplicity and operation costs. These factors, although important in developed countries cannot be considered critical. Therefore, each situation must be analysed individually and local conditions must be incorporated from the very beginning of the project cycle. The consideration of multiple alternatives is the best way to reach an efficient, economical and adequate solution not only at the design stage, but also throughout the operational life of the wastewater treatment plant.

Water reuse standards must be divided in standards for agriculture use, for industrial and municipal reuse, here should be given the standards for agriculture reuse. The quality assurance during re-utilisation and re-circulation must have priority. The quality of product water in industry has to met the standards of drinking water. Irrigation water needs to follow microbiological, chemical and physical requirements (Rosenwinkel and Cornel, 2005):

- microbiological (WHO):
    - Category A Treatment to Engelberg "unrestricted" guidelines essential
    - Category B Further measure may be needed

- Category C Protection needed only for field worker

- chemical and physical (FAO): classified irrigation water into three groups based on salinity, infiltration, toxicity and miscellaneous hazards

WHO guidelines

category	Reuse conditions	Intestinal nematode (arithmetic mean of no. eggs per litre)	Feacal coliforms (geometric mean of no. per 100 ml)
A	Irrigation of crops likely to be eaten uncooked, sport field, public park	≤1	≤1000
B	Irrigation of cereal crops, industrial crops, fodder crops, pasture and trees	≤1	no standard recommended
C	Localised irrigation of crops in category B if exposure to workers and the public does not occur	not applicable	not applicable

- Water should be free from filterable substances, mud and smell
- The load of depreciating substances (salt, metals) for soil, plants and groundwater should be so small, that these will not be harmed
- Water should be free from toxic substances
- Organic matter always has to be eliminated, because high loadings of organic matter can cause excessive build-up of soil microorganisms. This can cause a microbial slime layer to form on the soil surface, which can lead to soil surface clogging, and in turn, surface pounding, unpleasant odors, and vegetation decay. (EPA limits on BOD5 range from 10 mg/l to 30 mg/l)
- Substances in wastewater should not cause corrosion on the irrigation technique
- The treatment objective (nutrient content vs. nutrient removal) should be adjusted to the vegetation periods
- Water from wastewater treatment has to be disinfected
- Balance between nutrient demand and water demand of plants and soil

A critical element of water reuse systems is the effective treatment of wastewater to meet water quality objectives for water reuse applications and to protect public health. Typical wastewater treatment consists of a combination of physical, biological, and chemical processes to remove solids, organic matter, and if necessary pathogens, metals, and nutrients from wastewater. The goal in designing a wastewater reclamation and reuse system is to develop an integrated cost-effective treatment scheme that is capable of reliably meeting water quality objectives. The degree of treatment required in individual water treatment and wastewater reclamation facilities varies according to the specific reuse application and associated water quality requirements.

2. Demonstration of anaerobic technology for wastewater Pre-treatment

Answering to the high priority request concerning the sustainability criteria of the wastewater treatment technology, the anaerobic wastewater treatment should be regarded as the core method of a sustainable wastewater management strategy due to its benefits and enormous potentials such as: Little (if any) use of mineral resources and energy; Enabling production of resources / energy from wastes; Pairing high efficiency with long term of lives; Applicable at any place and at any scale; Plain in construction, operation and maintenance. Moreover, although conventional aerobic treatment systems generally provide excellent treatment efficiency, they do not fully meet the criteria needed for a sustainable wastewater management strategy (Lettinga, 1995,2001).

3.1. Benefits and drawbacks of anaerobic municipal wastewater treatment

Based on the past experiences and learned lessons in the municipal wastewater treatment, the anaerobic technology proved a very good performance and efficiencies due to its positive advantages against aerobic ones. The main advantages and drawbacks of the anaerobic municipal wastewater treatment systems are shown in table 3.

Table 3: Advantages and drawbacks of anaerobic municipal wastewater treatment systems (Lettinga, 2001; Foresti, 2001; Zeeman and Lettinga 1999; Jim Field, 2002).

Advantages	Drawbacks
1. **Economy of the process**, a substantial saving in operational costs as no energy is required for aeration as well as low investment costs of construction and maintenance.	*1.* **Need for post treatment**, in some cases to comply with the effluent standards, a simple/poor post treatment is necessary.
2. **Positive instead of negative energy balance**, on the contrary energy is produced in the form of methane gas, which can be utilized for heating and electricity production.	*2.* **Little available experiences**, some-times especially with the full-scale application at low and or moderate temperatures.
3. **High performance**, the process can handle high hydraulic and organic loading rates. Thus, the applied technologies are rather compact and reduce the volume of post treatment stages.	*3.* **Solubility of biogas**, significant amount of produced biogas dissolved in water and remains in the effluent especially for low strength wastewater.
4. **Simplicity**, the technologies are simple in construction, operation, monitoring, and maintenance, consequently they are cost-effective technologies.	*4.* Non utilized methane, produced methane during anaerobic municipal wastewater treatment is often not utilized for energy generation.
5. **Flexibility and sustainability**, the systems can be applied everywhere and at any scale and working with high treatment efficiencies.	
6. **Low generation of surplus sludge**, the excess sludge production is very low. Additionally, the sludge is well stabilized and easily dewatered due to high solids retention time. Hence, lower secondary costs for sludge handling	
7. **Possibility of nutrients recovery**, the valuable nutrients (N, P and K) is conserved which give high potential for crop irrigation and aquaculture when re-using the anaerobically treated wastewater in agriculture irrigation purposes.	
8. **Feasibility**, the anaerobic technology is feasible for a wide range of waste and wastewaters whether complex in composition, very low or very high strength as well as at low and high temperatures.	

3.1. Simplified concept of anaerobic wastewater treatment for re-use

3.1.1. Natural anaerobic UASB-pond

The hereafter Concept (figure 2) offers environmentally sound and economical attractive solutions for wastewater treatment and re-use. The wastewater is anaerobically treated using a natural UASB-Pond (figure 3) to win all of the anaerobic treatment benefits as winning of biogas as an alternative source of energy as well as less quantity of sludge with a very good stabilization status, and cost effective benefits as low capital, operation and maintenance cost. The treated wastewater will be naturally disinfected in polishing pond and re-used for agriculture purposes to recover the high valuable nutrients N, P and K.

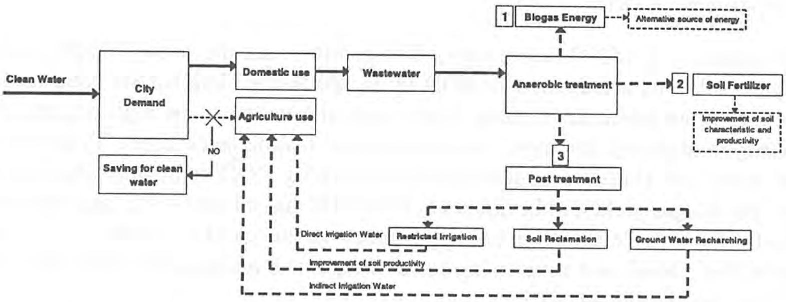


Figure 2: Integrated Concept for Wastewater Treatment and Re-use

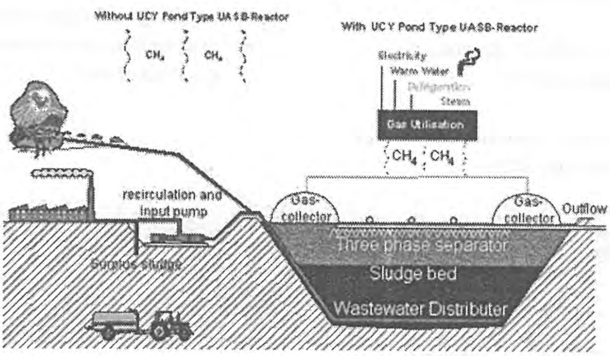


Figure 3: Natural UASB-pond reactor

The main aim of the Concept is the treatment of municipal wastewater and sustainable re-use for natural eco-systems maintaining by the development of a low cost technology in developing countries. The next illustrated below diagram figure 9 describes the main Concept that the clean water is entering to the city for the domestic use only but the agriculture use will be by using of the treated wastewater produced from this Concept.

Also Concept produces not only a water resource for irrigation but also produces an alternative sources of energy in form of biogas to be converted to heat or electricity to save the usual energy resources, also it produces a good stabilized sludge which can be used as a fertilizer to enrich and improve the soil characteristics. The treated wastewater will be used as a source of nutrients for soil, i.e. recovery of these nutrients will be done to use them in soil reclamation.

Because of the limited financial resources, there is a definite need for a cost-effective appropriate technology for sewage treatment system. The UASB reactor technology may be mostly attractive option for sewage treatment for developing countries, because it can be used at small or large scale, in technically simple, lower cost. In developing countries such as India and Colombia the UASB was executed in full scale to treat the municipal wastewater.

Performance of UASB-Pond reactor: The results from the used UASB pond in the tropical conditions a reduction in BOD up to (80-90) %. The UASB pond technology is feasible in an urban developing world context because of its high organic removal efficiency, simplicity, low-cost, low capital and maintenance costs. Typically UASB ponds have low sludge production (0.02-0.2 kg/kg COD removed) and low energy needs. The biogas yield will be about 60-75 % CH_4 and 20-30% CO_2, and then it will be a good, feasible renewable energy source to be used at a low cost concept. Construction, O&M, and monitoring costs of treatment and disposal with this Concept= (143-218) •/m3 or (29-44) •/P.E.

3.2.2. Components of large anaerobic treatment plant for Developing countries

Each anaerobic treatment plant consists of the following main elements, illustrated in Figure 4:

- overflow bypass and flow measurement structures
- preliminary treatment structure
- coarse and fine screens
- grit trap
- enhanced primary treatment structures (UASB reactor modules)
- by-product handling structures (biogas handling and using; sludge handling)
- post-treatment structures (simple, low cost technique)

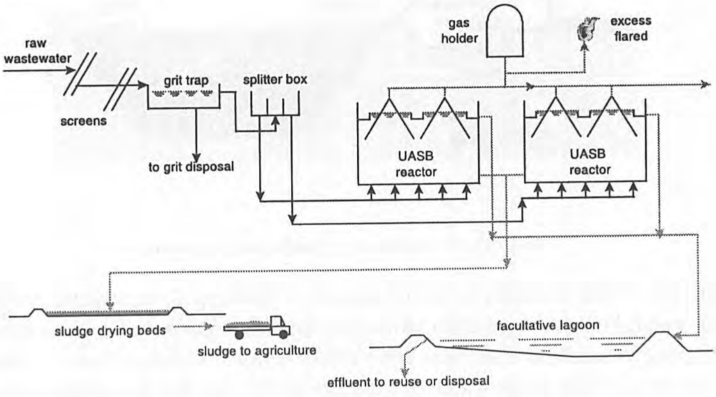


Figure 4: Flow Diagram of a UASB Treatment Plant

Table 4 gives the average removal efficiencies the UASB anaerobic treatment for oxygen consuming substances (BOD5 and COD) and total solids for the four treatment plants visited, including three treating municipal wastewater at Bucaramanga, Colombia; Mirzapur and Kanpur in India and one treating a mixture of tannery and municipal wastewater effluents, also in Kanpur.

Table 4: Comparison of average influent and reactor effluent quality and removal of four full scale UASB reactors

Parameter	Municipal Wastewater			Mixed wastewater
	Bucaramanga, Colombia	Mirzapur, India	Kanpur, India	Kanpur Indea
Design peak capacity (MLD)	42	14	5	36
Operating capacity (MLD)	36	10	4.8	21.8
Average organic loading				
COD (mg/l)	400	360	560	1183
BOD_5 (mg/l)	150	180	210	484
TSS (mg/l)	230	360	420	1000
Average removal efficiency				
COD (%)	65	61	74	57
BOD_5 (%)	75	66	75	63
TSS (%)	70	70	75	56
Average HRT (h)	5	8	6	5.2
Influent teperature range (oC)	23-25	21-30	20-30	22-30
Gas production (m^3/d)	3300	500	480	—

3.1.1. *Options for post treatment of anaerobically treated wastewater*

A normally functioning UASB reactor can remove an average of 65 percent of COD (range: 50-75 percent), 80 percent of BOD_5 (range 70-90 percent) and 75 percent of suspended solids (range 60-85 percent). Beginning with a typical municipal raw wastewater, this level of treatment will generally result in a treated effluent that corresponds to an "enhanced primary" quality, intermediate between primary and secondary (between 30-70 mg/l for BOD_5) (Alaerts et al., 1990). An effluent less than secondary quality will generally not meet environmentally sound effluent discharge standards and will definitely need further treatment to be safe for reuse in agriculture. The post-treatment should be designed to improve the effluent quality in the following parameters (APHA et al., 1981):

- pathogen contamination (measured by the index of E. coli);
- residual organic material (COD/BOD_5);
- oxygen demand from the reduced forms of N and S;
- residual suspended solids (TSS) and inorganic N and P (nutrients)

The treatment systems, which could be used for post treatment, can be summarized as follows:

Activated Sludge; Aerated bio filters; Expanded granular sludge bed reactor; Rotating biological contactor (RBC); Dissolved Air Flotation (DAF); Chemical post treatment; Trickling Filters; Waste stabilization ponds; Polishing ponds; Constructed wetlands; Aquating farming systems; Land treatment.

4. Wastewater treatment in European Union (German Experiences)

The most commonly used BNR processes in Germany are pre-denitrification and simultaneous nitrification-denitrification (SND). Plants practicing enhanced biological phosphorous removal (EBPR) typically employ Johannesburg, ISAH or Phoredox type configuration. Other plants use the cascade-denitrification and the alternating denitrification process.

In the following, we will present the operation experiences of six municipal wastewater treatment plants with different sizes and with different operation methods. Some plants are currently being initiated; some have already yielded long-term operation experiences. We will present the major dimensioning parameters, the achieved effluent data, and occurring operation problems with foam and bulking sludge.

Table 5: Survey of some plants in Germany

Plant	Operation Method	Aeration	Pop. equivalent
Hildesheim	simultaneous nitrification/denitrification, Bio-P in the main stream (ISAH-Method)	Surface aeration	160.000
Husum	simultaneous nitrification/denitrification, Bio-P in main stream and bypass (CISAH-Method)	Surface aeration	110.000
Wernigerode	Cascade plant with Bio-P in the main stream (ISAH-Method)	Pressure aeration	95.000
Rheda-Wiedenbrück	alternating cascade nitrification/denitrification	Pressure aeration	94.000
Bremen-Seehausen	combination: simultaneous nitrification/ denitrification and cascade plant with Bio-P in the main stream (ISAH-Method)	Pressure and surface aeration	695.000
Marne	Combination: anaerobic pre-treatment of the industrial bitstream and previous denitrification with nitrification in the rotary ditch	Surface aeration	32.000

3.1. Examples of Plants with Bio-P and simultaneous nitrification – denitrification

3.1.1. Wastewater Treatment Plant Hildesheim

The extension of the wastewater treatment plant at Hildesheim by a biological stage with nitrification, denitrification and biological phosphorous elimination was conceived in 1982/1983. At this time, practical experiences with industrial plants for biological phosphorous elimination were available only from plants in South Africa, where, as a rule, the wastewater is rather concentrated, that is the settled BOD ranges from 300 to 400 mg/l. In contrast, in Hildesheim one can expect concentrations below 100 mg BOD/l on rainy days or with high amounts of wastewater from external sources. The plant is run using the ISAH-Method, the main asset of which is that the return sludge is denitrified in a separate anoxic basin in order to prevent any possible impairing of the solution of phosphor through return nitrate. Because of the high concentration of biomass in the anoxic basin (proportionate to the concentration of solids in the return sludge), extensive denitrification via endogenous respiration is possible. If necessary, additional substrate can be fed from the anaerobic basin.

4.1.1.1 Dimensioning

Of the four lines planned for the activation plant, two were built and started in July 1987, in order to use operation experiences for the construction of the other two. The first of these was started at the end of 1996, the second is about to be initiated very soon.

Table 6: Dimensioning Data (2 activation basins)

Sludge Loads	0,12 kg BOD / kg DS * d
DS_{AB}	3,0 kg DS / m^3
T	15° C
activation basin	V_{AB} = 2 * 7.100 m^3 = 14.200 m^3 t_R = 11,3 h Aeration aggregates: 6 mammoth rotors per basin Aeration capacity. 288 kg O_2/h per basin V_{anox} = 2 * 625 = 1250 m^3 V_{an} = 2 * 875 = 1750 m^3
secondary clarification	V_{NK} = 2 * 3360 = 6720 m^3 A_{NK} = 2 * 1018 = 2036 m^2 q_A for Q_T = 0,61 m/h, q_A for Q_M = 1,22 m/h Effluent through submerged pipes

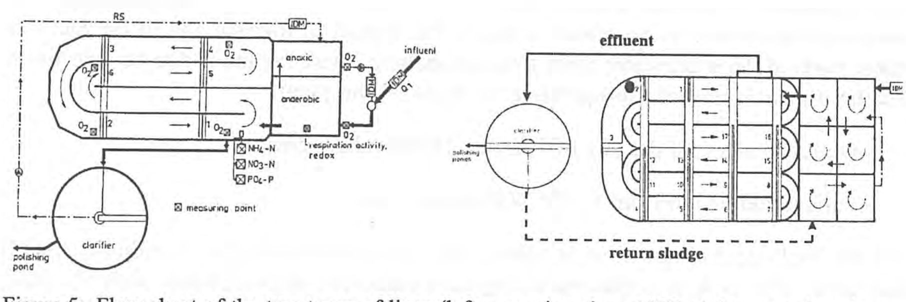


Figure 5: Flow sheet of the two types of lines (left = running since 1987, right = running since 1997) at the wastewater treatment plant at Hildesheim

Table 7: Load data of the wastewater treatment plant at Hildesheim for the operation with 4 lines in 1998 (average data)

	Operation 1998 (4 lines)	Dimensioning (4 lines)
Q	34.786 m^3/d	37.500 m^3/d
MLSS	2,5 g/l	3,0 g/l
COD	19.762 kg/d	-
BOD	10.011 kg/d	9.500 kg/d
$N_{tot.}$	1.647 kg/d	2.025 kg/d
$P_{tot.}$	283 kg/d	375 kg/d
Sludge Loading	0,07 kg BOD/(kg MLSS*d)	0,05 kg BOD/(kg MLSS*d)

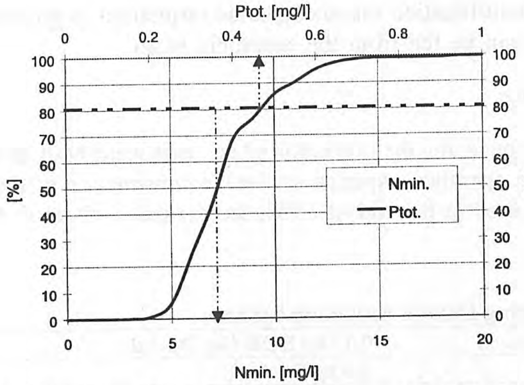


Figure 6: Cumulative frequency for the parameters $N_{mineral}$ (NH_4-N + NO_3-N) and P_{tot} in the effluent of the secondary clarifiers at the WWTP at Hildesheim as daily average 01-09/1998

For the analysis period (1998), excellent effluent data for the parameters Ptotal and Nmineral could be achieved. The low effluent values for Ptotal result almost entirely from the achieved biological phosphorus elimination. Because of the excellent operation results, the owners will for 1999 to the controlling authorities declare lower monitoring results compared with the demands presented in Table 1, which would allow for considerable financial savings in the area of wastewater levy.

3.1.1. Wastewater Treatment Plant at Husum

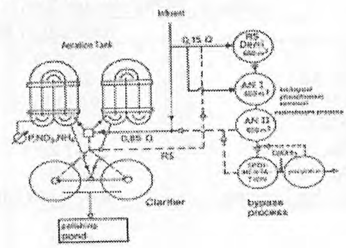
The wastewater treatment plant at Husum is conspicuous for the exceptionally wide seasonal variation of loads, which is due to the impact of tourism and to the fact that major parts of the wastewater come from an abattoir. These factors had to be considered for the dimensioning of the operation methods of the plant.

- municipal share of the city of Husum: 25.000 inhab. equiv.
- tourism and abattoir: up to 136.000 inhab. equiv.

For the biological elimination of phosphorus, the CISAH-process (Combined ISAH) was used. This process combines biological elimination of phosphorus with the main stream method (ISAH-process) with an optional precipitation of bitstreams from the anaerobic tank for the biological removal of phosphorus.

Table 8: Dimensioning Data and Flow sheet of the WWTP in Husum

aeration tank	$V = 2 * 5.500\ m = 11.000\ m^3$ 6 mammoth rotors per tank 360 kg O_2/h per tank $V_{anox} = 650\ m^3$ $V_{an} = 650\ m^3$
secondary clarifier	$V = 2 * 3.300 = 6.600 m^3$ $A = 2 * 700 = 1.400\ m^2$ Effluent through submerged pipes



Based on the good conditions, which are favourable for the elimination of both nitrogen and phosphorus (C_{BOD} = 1.318 mg/l, BOD/P = 84,9), it was possible to achieve excellent effluent data for the parameters nitrogen (NH_4-N + NO_3-N) and Ptot. It was possible to entirely dispense with any precipitation in the bypass flow. The carbon loads are considerably higher than those found in earlier data evaluations, which is due to the production extension and restructuring of the connected abattoir during the year 1996.

Table 9: Average day loads in the influent of the wastewater treatment plant at Husum in 1998 in comparison to the dimensioning data, (*) = no data available

		Operation data 1998	Dimensioning data
Q	m^3/d	6.794	13.000
BOD	kg/d	9.681	6.600
COD	kg/d	17.903	- (*)
NH_4-N	kg/d	275	- (*)
$N_{tot.}$	kg/d	592	1.320
$P_{tot.}$	kg/d	103	200
MLSS	g/l	5,6	5,0
Sludge Loading	kg BOD/(kgMLSS*d)	0,15	0,12

Figure 7: Cumulative frequency of the parameter $N_{mineral}$ (NH_4-N + NO_3-N) and P_{tot} in the effluent of the secondary clarification of the wastewater treatment plant at Husum from 1995 to 08/1998

3.1. New treatment technologies

Overall removal efficiency shows significant decline in effluent concentrations in Germany, mainly due to improved plant design, novel processes and better operation. Significant inroads in activated sludge treatment are being made in EU by the biofilm technologies. Some 40 plants with biological filtration operate in Germany today, treating effluent from 1.5 million PE. The plants use biofiltration for nitrification and also for denitrification. The reactors include aerated submerged-bed upflow filters, downflow filters, and continuously or discontinuously backwashed filtration. The effluent suspended solids concentration of filtration units, following conventional final clarifiers are below 5 mg/l, the nitrogen concentrations in the effluent are covering a wider scale, depending on variations of inlet flow and composition, mode of operation, e.g. dosage of external carbon (Rosenwinkel et al., 2005).

Increasingly stringent nitrogen standards call for separate treatment of sludge processing liquors using physical, chemical and biological methods. In a few cases, the nitrogen load is decreased by incineration of raw sludge, e.g. at Vienna or Berlin.

The membrane bioreactor (MBR) technology begins to be implemented more often and for larger facilities. In the MBR the secondary clarifier is replaced by a membrane module yielding high-clarity final effluent. Smaller reactor volumes are typically the result of higher contents of MLSS (up to 15 g/l and above) in comparison to standard activated sludge systems. Today 28 MBR plants operate all over Europe, with 15 of them in Germany. The capacities vary from 150 to 200,000 PE. The technology promises to reduce the suspended solids to zero and to minimise a lot of harmful substances in the effluent. Of special importance for the evaluation is the fact, that the membrane is only a physical selection-tool and the reaction is to be done by the biological system. The advantage of the effluent-quality with meeting the high standard of the EUbathwater-guideline stands in contrast to the higher energy demand and the sensitivity against peak-concentrations like ammonia (Rosenwinkel et al., 2005).

3.2. Phosphorous-recovery

Another trend is the development of techniques for phosphorous recovery, which can be done by recovery from the effluent, from the sludge or from the ash. In case of plants with enhanced biological P-removal (EBPR) the goal is to re-extract the phosphorous stored in the phosphorous accumulating organisms (PAOs). For this a reactor is placed on return activated sludge line in which an anaerobic phosphorous release is initiated by dosing readily biodegradable carbon. Through subsequent precipitation and separation the stored phosphorous can be recovered and re-used as e.g. fertilizer (Rosenwinkel et al., 2005).

4. Energy Aspects

One major part of the operation costs of municipal wastewater treatment plants are the energy costs, which are mainly accrued by the biological stage. According to Grünebaum et al. (1996), the ratio is 15 - 25 %, with approximately three quarters of the total amount of electrical energy of the WWTP is consumed by the biological stage (Bohn,1997). Parameters to describe the treatment efficiency of any biological wastewater treatment plant are the load or people specific energy consumption amounts, which in table 10 are shown for three of the six WWTP's. The parameters of the plants at Bremen and Hildesheim have comparable dimensions, which is mainly due to the fact that their influent situations are similar. Conspicuous is the considerably different dimension of the parameters of the WWTP at Husum, in particular in regard to the load specific and ensuing people specific values. The comparison with the water-amount specific value, which is relatively high, shows that this distortion is mainly due to the unusual influent conditions (BOD/N of 84,9) caused by the high ratio of industrial wastewater, which leads to a situation where the ratio of the aeration energy used up by the energy for nitrification is considerably lower than with the other two examined WWTP's.

Table 10: Energy consumption and production of three WWTP's, (#) = no energy production

Plant	Hildesheim (1998)	Husum (1998)	Bremen-Seehausen (1997)
External sources [Kwh/a]	2.710.000	1.879.420	10.537.212
production [Kwh/a]	2.880.945	- (#)	12.168.762
Total consumption [Kwh/a]	5.590.945	1.879.420	22.705.974
Q [m^3/a]	12.696.890	2.479.880	45.936.200
BOD-Load [kg BOD/a]	3.654.015	3.533.829	18.181.600
People equivalent	167.000	161.000	830.217
[Kwh/P.E. · a]	33	12	27
[Kwh/kg BOD]	1,5	0,5	1,3
[Kwh/m^3]	0,4	0,8	0,5

3. Conclusion

This paper presents the different operation procedures of some of municipal wastewater treatment plants in the North of Germany, the analyses of the available dimensioning and operation results compared with the specific energy consumption. The plant at Hildesheim has relatively low loads. It was possible to reach effluent values considerably below the required minimum demands. The wastewater treatment plant at Husum is conspicuous for the high degree of wastewater coming from food processing factories, which makes for high elimination degrees for the parameters nitrogen and phosphorus due to the high sludge production.

Also the paper discussed and demonstrated the anaerobic technology as a cost-effective pre-treatment technology as a potential of comprehensive concept of wastewater treatment and re-use.

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Domestic wastewater in Jordan

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DOMESTIC WASTEWATER IN JORDAN

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ABSTRACT

Jordan is an arid to semi-arid country with an area of approximately 89,342 km^2. More than 90% of Jordan receives less than 200 mm of rainfall per year and approximately 85% of the total average rainfall in Jordan is lost through evaporation. The remaining rainfall recharges groundwater and contributes to rivers, wadi flows, and reservoirs. In many Jordanian cities, residents receive water only sporadically, and domestic water consumption is among the lowest in the world, less than 100 liters/capita/ days. The most feasible options for reducing the gap between water demand and supply are the improved management of existing water resources, treating wastewater for reuse, and the rehabilitation of existing water sources. Moreover, optimal development and utilization of water resources in Jordan and the institution of an associated water policy requires the establishment and implementation of several integrated resources, information and management systems. The Ministry of Water and Irrigation / Water Authority of Jordan is responsible at the national level for administering water policy, pollution control and managing water resources. In fact, with the advent of industrialization and increasing populations, the range of requirements for water have increased together with greater demands for higher quantity and quality of water. Water issues are linked to scarcity of water, which leads to a shift in water planning in Jordan towards the use of non-conventional water resources mainly reclaimed domestic wastewater for the intended uses. Domestic wastewater is water that has been used in the home including toilets, clothes washers, showers and laundry and makes up 99.85% of sewage entering the treatment plant, it is collected and treated to standards to be used in unrestricted agriculture and other non-domestic purposes. This important resource, reclaimed water, has been considered from the highest level of the Jordanian government that it has a full value to the overall water resources of the country. This will help in meeting water supply needs, providing sanitation services that protect public health, preserving the source value of reclaimed water and ensuring environmental protection. In this paper, I will present a summary about Jordan's experience as a developing country in quality aspects of reclaimed wastewater. The following subjects will be discussed: wastewater definition, Reclaimed Wastewater Standard No.893/2002, means and ways to protect water resources from the effect of wastewater quality, the physical, chemical and biological properties of wastewater, monitoring activities, treatment plants efficiencies, cost of treatment, and a description of a reuse pilot project at Aqaba and Wadi Mousa. The paper will be finalized by conclusions and recommendations.

Keywords: Reclaimed water, Reuse, Quality, Monitoring, and Efficiency.

Introduction

The directorate of Laboratories & Quality through the Division of Environmental Monitoring and Assessment monitors wastewater all over the country. Reclaimed wastewater discharged from domestic wastewater treatment plants is an important component of Jordan's water budget. About 94-98 MCM in the year 2003/2004 were treated and discharged into various water courses or used directly for irrigation and other intended uses. Effective protection of the environment requires accurate and detailed knowledge of existing environment conditions and the ability to detect and measure the water quality trends.

The monitoring of reclaimed wastewater quality involves many distinct activities to give reliable and usable data. A monitoring program for domestic wastewater is designed according to standard no. 893/2002 to collect representative samples through QA and laboratories accreditation process. In fact, sewage treatment is a multi-stage process to renovate wastewater before it reenters a body of water or is reused. The goal is to reduce or remove organic matter, solids, nutrients, disease causing organisms and other pollutants from wastewater. Treatment plants should reduce pollutants in wastewater to a level that nature can handle.

What is wastewater and why treat it?

Wastewater is not just sewage. All the water used in the home that goes down the drains or into the sewage collection systems is wastewater. This includes water from baths, showers, sinks, dishwashers, washing machines, and toilets that is called black water. In combined municipal sewage systems, water from storm drains is also added to the municipal wastewater sewer system.

The average Jordanian contributes < 100 liters of wastewater everyday. Wastewater is about 99.85% water by weight and is generally referred to as influent as it enters the treatment plant. Domestic wastewater is wastewater that comes primarily from individuals, and doesn't generally include industrial wastewater. Moreover, domestic wastewater in Jordan includes industrial wastewater from industries connected to the public sewer system according to WAJ regulations issued in 1998.

Wastewater Treatment

Most of the cities of Jordan are equipped with wastewater treatment plants since it was decided to treat wastewater up to the secondary level and meet the current standards and WHO guidelines. The existing public – sector wastewater treatment plants in Jordan are 20 using different types of treatment systems mainly divided into trickling filters, activated sludge and waste stabilization ponds as shown in Table 1.The aim of WAJ is to increase the volume of treated wastewater through improvements in the existing treatment infrastructure and the construction of new treatment systems ensuring compliance with current standards. It is also planned to shift most of the treatment trains to activated sludge processes.

Table 1: Wastewater treatment plants in Jordan

Treatment System	Treatment Plant
Activated Sludge	Irbid,W.Arab,W.Hasan,Salt,Madaba,A,Nusier,Tel Mantah,Fuhais,W,Musa,Ramtha
Trickling Filter	Kufranja,Tailah,Baqah,Karak
Stabilization Ponds	Samra,Maan,Mafraq,Aqaba,W.esseir

A common set of processes that might be found at a municipal treatment plant would be:

- Preliminary treatment to remove large or hard solids that might clog or damage other equipment.
- Primary settling basins, where the water flows for up to a few hours, to allow organic suspended matter to settle out or float to the surface, these settling tanks can be rectangular or circular.
- Secondary treatment, a type of wastewater treatment used to convert dissolved and suspended pollutants into a form that can be removed, producing a relatively highly treated effluent. It utilizes biological treatment processes followed by settling tanks and removes approximately 85% of the BOD and TSS in wastewater. Secondary treatment for municipal wastewater is the minimum level of treatment required.
- Tertiary treatment: any level of treatment beyond secondary treatment, which could include filtration, nutrient removal (removal of nitrogen and phosphorus) and removal of toxic chemicals or metals. This type of treatment will be used in the new Aqaba treatment plant under construction.

Jordanian Wastewater Quality Standards

WAJ follows national legislation that has been issued by the Jordanian Institute of Standards and Metrology (JSIM) and regulations issued by the Minister of Water and Irrigation.

The most important standards which wastewater quality is governed by can be summarized into:

1. JS 893/2002: This Jordanian standard addresses the standard requirements and quality control for reclaimed water. It deals with requirements and properties that domestic wastewater must meet before being discharged to any receiving body or reused for agriculture or other intended uses.
2. JS 202/2004: This standard deals with industrial wastewater, which is produced after being used for industrial purposes. The aim of implementing industrial wastewater monitoring program is to protect the environment, water resources, safeguard health and human safety. In case of discharging the industrial wastewater or reuse, it should meet and comply with the above standard that has been renewed.
3. Regulations issued by the Ministry of Water and Irrigation according to WAJ Law No. 18/1988. These regulations deal with industries to be connected to the public sewer systems in order to control the release of wastewater to sewer pipelines and treatment systems.

Monitoring Activities

The Wastewater Monitoring Programs at labs & Water Quality Control department can be summarized as follows:

A) **A Domestic Wastewater Quality Monitoring Program**

1) This program focuses on monitoring the effluents & influents from the public treatment plants, which are operated by WAJ. These treatment plants are all mainstream technologies that are in common use throughout the world and would be classified as in Table 1.

2) This program focuses on monitoring the effluents of 20 Treatment plants that are operated by private sector such as Mut'a T.P effluent and others. The basic objective of the implementation of this program is to control the pollution loads and minimize their effects on groundwater and surface water, which can be achieved by having a well designed operational system so that its effluent complies with the Jordanian standards to be used for the recommended application since the water strategy gives a high priority and a full value for the reclaimed water in the water budget.

B) **Streams, Wadis, Dams and Reservoirs Monitoring Program**

The number of sites to be monitored is about (60), this program is designed to monitor selected sites such as King Talal Dam, Wadi Arab Dam, and that receive direct treated flow from domestic wastewater treatment plants.

C) **Industrial Wastewater Monitoring Program**

This program focuses on monitoring the effluents of more than 175 industrial establishments. These factories are classified as follows:

1) Connected industries to the sewer system: the evaluation is based on WAJ regulations in order to protect the sewer pipelines and the treatment plant system.

2) Non-connected industries to the sewer system: the evaluation of the water quality is based on the Jordanian standard 202 which is specified for factories dumping their waste in to the environment.

The over all value of implementing this monitoring program is to protect water resources from the toxic materials and pollution loads resulting from industrial emissions and protect treatment systems to keep good reclaimed water.

New approach for monitoring mechanism

The role of the government in monitoring is being reevaluated in Jordan. The old model–government which does everything and pays for everything is being replaced by private sector participation. The countries of USA, Canada and Europe are using this mechanism. In fact, this means that the government of Jordan will focus on setting and enforcing rules and standards for this sector .Therefore, adequate treatment has to be provided to improve water quality and prevent health hazards and environmental problems.

Wastewater Analysis

Various types of pollutants are present in domestic wastewater that can be measured by many different parameters such as H, M, NO_3, e-coli, CL, PO_4, TSS, TDS…etc. The most important class of wastewater contaminants are compounds that react with oxygen which are characterized by COD, BOD, and the second class is suspended solids.

Wastewater Evaluation

The generated water quality data Tables 2 and 3 will be evaluated according to the reclaimed wastewater standard no 893/2002. After the evaluation process the directorate issues monthly, quarterly, biannual and annual reports that show treatment plants violating the standard. The objective of issuing these reports is to address the problems and ask for improvements to protect and minimize their effects on water resources and the environment. Moreover, the water quality differs from one treatment method to another depending on the operation conditions and the type of treatment system. Untreated sewage in Jordan, for example, has a BOD_5 (BOD_5 measures the amount of oxygen microorganisms required in five days to break down sewage) ranging from 475mg/l in Wadi Arab Treatment Plant to 1137 mg/l in Madaba Treatment Plant that means that it is a strong sewage comparing it with raw sewage in the USA which ranges from (100-300) mg/l. Table 4 shows the difference of untreated wastewater concentration from one city to another.

Treatment plants efficiency

The efficiency of 20 treatment plants as shown in Table 5, measured by BOD_5 as an indicator of removing dissolved organic matter from treated sewage, ranges from 71% for Maan Treatment Plant to 99% for Wadi Arab Treatment Plant. The efficiency for the wastewater treatment plants and the operation systems used in Jordan is shown in Figures 1 and 2 for the year 2002/2003. These figures clarify that the activated sludge is very effective in removing dissolved organic matter and WAJ can rely on it as a first choice followed by the trickling filter. The historical data shows that the wastewater stabilization ponds have a low efficiency in removing dissolved organic matter.

How well are we doing

In Jordan, the government's policy is to achieve and improve wastewater collection, conveyance, treatment, and disposal and reuse systems. WAJ so far has provided the service on sewer and treatment systems, 20 treatment plants exist all over the country working 24 hours a day and the number of carried out connections is (172133) at the end of the year 2003, 67% of these connections flow to Samra Treatment Plant. Therefore, urban sanitation coverage is more than 66% of the urban areas and about 50% of the total population.

Treated wastewater quantity

The wastewater quantity that was treated in plants is about 94.1 MCM for the year 2003 and it increased by 6.27% from the previous year as shown in Table 7. Moreover, 72.5% of wastewater quantity was treated at the Samra Treatment Plant. The quantity of reclaimed water was about 74 MCM for the year 2003.In fact, reclaimed water has long been recognized as a valuable resource for use in irrigation and other intended uses and considered as an important water resource according to the Jordan Water Strategy. WAJ has a goal of attaining total water reuse by having highly treated effluent to be used where required.

Wastewater reuse

The water reuse policy was launched at the beginning of 1997and the water reuse has been made an integral part of overall environmental pollution control and water management strategies. Water reuse is now a part of Jordan's overall water resources balance and also a way of protecting water resources, coastal areas and receiving bodies from pollution effects. Planned reclaimed water reuse has been practiced in Jordan and some pilot projects have been launched or are under study for irrigation and other intended use. Water reuse was implemented after the construction of existing treatment plants. However, for new plants, treatment and reuse are combined from the planning stage up to the implementation studies. A water reuse program has been set up and experimental research has been conducted on this subject .The main problem with the use of reclaimed water is the threat to public health, the soil and water resources if reuse is not done carefully. Given the emphasis that Islam places on cleanliness there is also a persistent notion within the region that wastewater reuse is against Islam. However, as noted in Water Management in Islam, published jointly by IDRC-UNU Press (2001), wastewater reuse is permissible for all purposes, including Wudu, provided that it is treated to the required level of purity for it is intended use and does not result in any adverse public health effects. Wadi Musa is a project funded by USAID in collaboration with the WAJ and Badi Program to implement direct reclaimed water reuse which covers 1069 dunums of land available for irrigation, that is reliable, commercially viable, socially acceptable, environmentally sustainable and safe. In doing so, it will demonstrate to decision makers and the public that water reuse is an effective, viable and safe component for managing water resources The program will work towards a practical approach to the reuse of reclaimed water ,while incorporating poverty alleviation, economic improvement and long–term project sustainability.

Cost of Treatment

The treatment cost differs from one treatment plant to another; the minimum cost was 16.2 fils/m^3 at the Aqaba Treatment Plant while the maximum cost was 798.4 fils/m^3 for Wadi Musa. Table 6 shows the cost of treatment for 19 treatment plant in 2003.

Conclusions:

1. Water reclamation and reuse have rapidly expanded in recent years. It is a clear indication that the highest level of government in Jordan recognizes the full value of reclaimed water to the over all water resources of the country.

2. Current reclaimed wastewater standards regulate water reuse and environmental discharge to ensure optimal performance of the wastewater treatment plants.

3. Reclaimed water used for irrigation shall be used carefully to control and protect the heath and safety of worker and the general public who may be exposed to the water.

4. Planned reuse programs should be created to stop discharging wastewater effluent to streams and catchments areas.

5. There is a need for active and collaborative involvement of other ministries and agencies and public participation to make use of reclaimed wastewater.

6. Reclaimed wastewater monitoring program should be implemented according to standard 893/2002 from both the regulatory body and the operational agency.

7. There is a need to conduct research projects based on actual uses of reclaimed water.

8. Public awareness programs should be organized all over the country to let the people accept the reuse of reclaimed wastewater in irrigation and other uses.

Technical References

{1} Jordanian Reclaimed Wastewater Standard 893/2002.

{2} FAO Guidelines for Agriculture, 1991.

{3} Reuse of Effluents, Methods of Wastewater Treatment &Health Safeguards WHO,1989.

{4} Standard, Regulations & Legislation for Water Reuse in Jordan/USAID/Ard-January 2002.

{5} Wastewater Annual Report 2003/Dept of Wastewater Systems-WAJ.

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{7} Wastewater Treatment Principles by Karen Mancl-www.ohioline.osu.edu.

{8} Labs Annual report for the year 2002 & 2003.

{9} Operation of Wastewater Treatment Plants, Volume 1, EPA, Kenneth D.Kerri, 1998.

Appendix A
Tables from (2-7)
Tables 2 and 3: Domestic Wastewater Samples Results-Effluents for 2003

		(BOD) mg/L	(COD) mg/L	Phosphate mg/l as P	T.D.S mg/L	pH unit
Abu Nusseir T.P Effluent	02/01/2003 00:00	60				3.0
Abu Nusseir T.P Effluent	21/01/2003 00:00	29				7.2
Abu Nusseir T.P Effluent	03/02/2003 00:00	10				7.0
Abu Nusseir T.P Effluent	11/03/2003 00:00	12				2.5
Abu Nusseir T.P Effluent	20/03/2003 00:00	14				4.1
Abu Nusseir T.P Effluent	28/03/2003 00:00	20				3.2
Abu Nusseir T.P Effluent	01/04/2003 00:00	11				3.6
Abu Nusseir T.P Effluent	03/05/2003 00:00	7.0				6.8
Abu Nusseir T.P Effluent	15/05/2003 00:00	6.0				6.89
Abu Nusseir T.P Effluent	11/06/2003 00:00	5.00				6.80
Abu Nusseir T.P Effluent	25/06/2003 00:00	13.0				7.30
Abu Nusseir T.P Effluent	01/07/2003 00:00	13				6.80
Abu Nusseir T.P Effluent	19/07/2003 00:00	13.0			1172	7.30
Abu Nusseir T.P Effluent	05/08/2003 00:00	7			1136	6.2
Abu Nusseir T.P Effluent	21/08/2003 00:00	4	60		956	8.0
Abu Nusseir T.P Effluent	06/09/2003 00:00	38.0	92		1128	7.5
Abu Nusseir T.P Effluent	16/09/2003 00:00	22.0	94		1077	6.90
Abu Nusseir T.P Effluent	01/10/2003 00:00	<5.0			1262	7.80
Abu Nusseir T.P Effluent	11/10/2003 00:00	<5	96		1298	7.00
Abu Nusseir T.P Effluent	01/11/2003 00:00	11.0	55.0	14.6	1001	6.70
Abu Nusseir T.P Effluent	15/11/2003 00:00		84	14.5	976	7.50
Abu Nusseir T.P Effluent	04/12/2003 00:00	<5.0	57	15.93	1371	8.00
Abu Nusseir T.P Effluent	16/12/2003 00:00	14			1596	6.00
Abu Nusseir T.P Effluent /After Chloraniation	19/04/2003 00:00	19.0				6.5
Abu Nusseir T.P Effluent /Before Chlorination	19/04/2003 00:00	18.0				6.8
		(BOD) mg/L	(COD) mg/L	Phosphate mg/l as P	T.D.S mg/L	pH unit
Alkherbeh Alsamra (site 4.0)T.P Effluent	07/01/2003 00:00	186				8.1
Alkherbeh Alsamra (site 4.0)T.P Effluent	15/01/2003 00:00	177				8.6
Alkherbeh Alsamra (site 4.0)T.P Effluent	06/03/2003 00:00	132				8.2
Alkherbeh Alsamra (site 4.0)T.P Effluent	11/03/2003 00:00	90				8.0
Alkherbeh Alsamra (site 4.0)T.P Effluent	31/03/2003 00:00	146				8.1
Alkherbeh Alsamra (site 4.0)T.P Effluent	08/04/2003 00:00	108				8.2
Alkherbeh Alsamra (site 4.0)T.P Effluent	08/05/2003 00:00	102.0				8.00
Alkherbeh Alsamra (site 4.0)T.P Effluent	02/06/2003 00:00	118				8.10
Alkherbeh Alsamra (site 4.0)T.P Effluent	11/06/2003 00:00					8.20

Alkherbeh Alsamra (site 4.0)T.P Effluent	05/07/2003 00:00					7.90
Alkherbeh Alsamra (site 4.0)T.P Effluent	29/07/2003 00:00				1220	8.03
Alkherbeh Alsamra (site 4.0)T.P Effluent	11/08/2003 00:00	92			1282	7.5
Alkherbeh Alsamra (site 4.0)T.P Effluent	17/08/2003 00:00		240	6.75	1284	8.0
Alkherbeh Alsamra (site 4.0)T.P Effluent	04/09/2003 00:00	48.0	297			8.3
Alkherbeh Alsamra (site 4.0)T.P Effluent	29/09/2003 00:00	69.0		18.6	1290	7.80
Alkherbeh Alsamra (site 4.0)T.P Effluent	06/10/2003 00:00	143			1124	7.60
Alkherbeh Alsamra (site 4.0)T.P Effluent	14/10/2003 00:00		236	11.36	1361	7.90
Alkherbeh Alsamra (site 4.0)T.P Effluent	04/11/2003 00:00	94.0	324.0		1428	7.80
Alkherbeh Alsamra (site 4.0)T.P Effluent	16/11/2003 00:00		336	17.92	1188	7.60
Alkherbeh Alsamra (site 4.0)T.P Effluent	02/12/2003 00:00	84.0	321	19.91	1324	7.60
Alkherbeh Alsamra (site 4.0)T.P Effluent	28/12/2003 00:00				1299	7.52
Alkherbeh Alsamra (site 4.0)T.P Effluent/s.1	29/04/2003 00:00	76.0				7.80
Baqaa T.P Effluent	02/01/2003 00:00	74				8.3
Baqaa T.P Effluent	21/01/2003 00:00	36				8.0
Baqaa T.P Effluent	03/02/2003 00:00	21				8.0
Baqaa T.P Effluent	11/03/2003 00:00	42				7.8
Baqaa T.P Effluent	20/03/2003 00:00	27				8.2
Baqaa T.P Effluent	28/03/2003 00:00	9				7.0
Baqaa T.P Effluent	01/04/2003 00:00	47				8.3
Baqaa T.P Effluent	19/04/2003 00:00	62.0				8.1
Baqaa T.P Effluent	03/05/2003 00:00	21.0				7.90
Baqaa T.P Effluent	15/05/2003 00:00	9.00				7.48
Baqaa T.P Effluent	11/06/2003 00:00	42.0				8.10
Baqaa T.P Effluent	25/06/2003 00:00	27.0				8.0
		(BOD) mg/L	**(COD) mg/L**	**Phosphate mg/l as P**	**T.D.S mg/L**	**pH unit**
Baqaa T.P Effluent	01/07/2003 00:00	14				7.90
Baqaa T.P Effluent	19/07/2003 00:00	25.0			1232	7.70
Baqaa T.P Effluent	05/08/2003 00:00	26			1192	7.5
Baqaa T.P Effluent	21/08/2003 00:00	21	99		1336	8.0
Baqaa T.P Effluent	25/08/2003 12:00	16	84		1268	7.5
Baqaa T.P Effluent	26/08/2003 00:00					
Baqaa T.P Effluent	26/08/2003 10:30	7	24		1382	7.7
Baqaa T.P Effluent	27/08/2003 00:00	39.0	95.0			
Baqaa T.P Effluent	27/08/2003 12:00	13.0	152.0		1480.0	7.60
Baqaa T.P Effluent	06/09/2003 00:00	35.0	75		1280	8.2
Baqaa T.P Effluent	16/09/2003 00:00	41.0	92		1240	7.40
Baqaa T.P Effluent	01/10/2003 00:00	36.0			1364	7.80
Baqaa T.P Effluent	06/10/2003 00:00	-				

Table 4: Raw wastewater in Jordan

T.P	BOD(mg/l)	T.P	BOD(mg/l)
Madaba	1137	As-samra	693
Jerash	1114	Tafila	691
Kufranja	1076	Karak	654
Irbid	1066	Maan	607
Baqaa	986	Fuhis	604
Salt	848	Wadi Alseer	538
Wadi Hassan	802	Wadi Mousa	527
Mafraq	728	Abu-Nusir	525
Wadi Arab	709	Aqaba	475
Ramtha	696	As-samra	693

Table 5: Treatment plant efficiency

T.P	Efficiency	T.P	Efficiency
Madaba		As-samra	
Jerash		Tafila	
Kufranja		Karak	
Irbid		Maan	
Baqaa		Fuhis	
Salt		Wadi Alseer	
Wadi Hassan		Wadi Mousa	
Mafraq		Abu-Nusir	
Wadi Arab		Aqaba	
Ramtha		As-samra	

Table 6: Cost of treatment.

WWTP	Cost Fils/m3	WWTP	Cost Fils/m3
Aqaba	16.2	Salt	107.0
As-samra wsp	18.6	Abu-nuseir	119.6
wadiessir	39.6	Karak	142.0
Ramtha	46.7	Madaba wsp	142.7
Mafraq	57.5	Wadi arab	145.4
Baqaa	65.3	Fuheis	164.5
Jerash	69.3	Tafila	232.7
Maan	69.5	Wadi hassan	484.1
Irbid	75.4	Wadi mousa	798.4
Kufranja	100.0	Tel.Mantah	-

* source: Wastewater Department

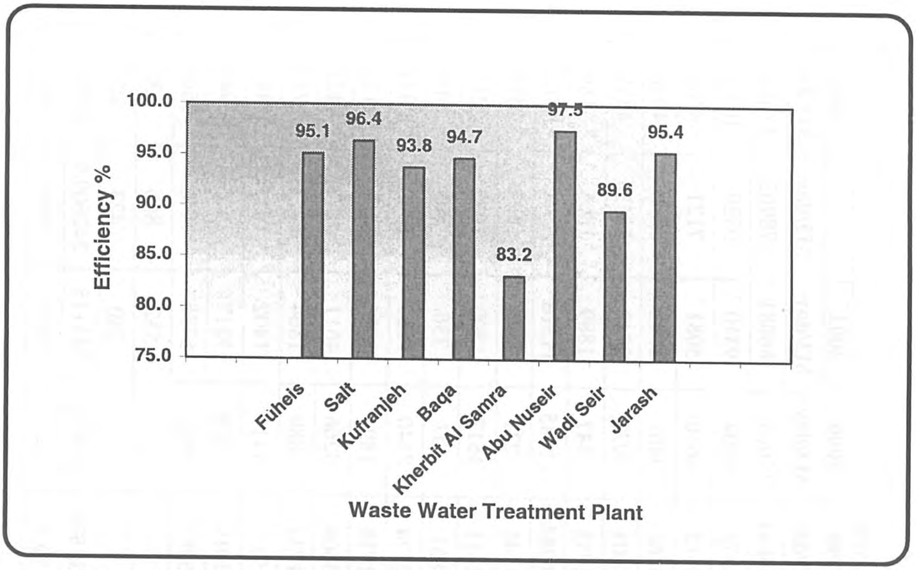

Figure 1: Domestic Waste Water Treatment Plant Efficiencies in 2003

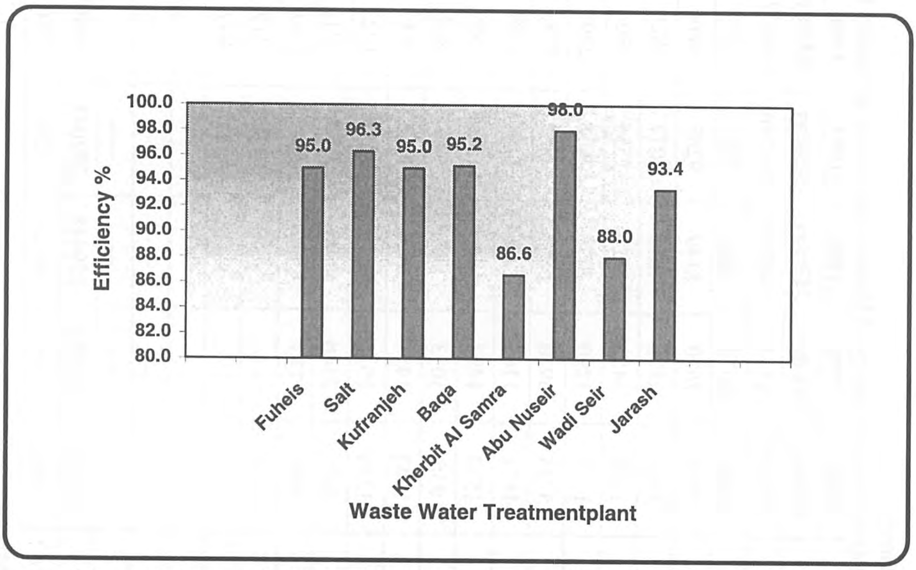

Figure 2: Domestic Waste Water Treatment Plant Efficiencies in 2002

Table 7: Treated water quantity.
Ministry of Water & Irrigation, Water Authority of Jordan, Laboratories & Quality Department

Year*	1994	1995	1996	1997	1998	1999	2000	2001		2003
	M3/day	M3/day	M3/day	M3/day	M3/day	M3/day	M3/day	M3/day	M3/day	M3/day
Samra	129177	143441	148795	156746	168857	166844	170752	186081	178902	186823
Aqaba	5488	6014	6666	7341	8219	8774	8804	9310	9329	10332
Irbid	7238	7620	8149	9287	8474	4612	4610	5081	7121	8103
Salt	3761	3870	4053	4077	3825	3166	3403	3598	3898	4248
Jerash	1354	1450	1524	1555	1804	1603	2072	2743	2913	4359
Mafraq	1317	1290	2379	2638	2297	1933	1847	1889	1805	2189
Baqa a	5214	6920	6891	7301	8783	10284	11185	11516	11768	12052
Karak	1071	1165	1266	1164	1122	1146	1231	1275	1508	1574
Abo Nuseir	1532	1497	1463	1486	1499	1411	1617	1800	1977	2215
Tafila	936	1013	966	747	862	851	707	736	740	844
Ramtha	1247	1431	1414	1675	1617	2174	2340	1889	2300	3071
Ma an	1350	1530	1672	1802	1923	1738	1892	1556	2155	2119
Madaba	2077	2440	2693	3309	3219	3609	4266	4611	4178	4422
Kufranja	690	730	1517	1649	2240	1734	1889	1864	2223	2787
Wadi Alseer	-----	----	----	856	819	914	1113	1402	1917	2445
Fuhis	-----	----	----	410	847	1019	1218	1217	1523	1944
Wadi Arab	-----	----	----	----	----	5993	5985	5735	7063	6667
Wadi Mousa	-----	----	----	----	----	----	----	532	866	900
Wadi Hassan	-----	----	----	-----	----	-----	----	280	423	725
Total m3 \ d	162452	180411	189448	202043	216407	217805	224931	243115	242609.9	257819
Total(MCM) M3 \yrl	59.3	65.9	69.3	73.7	79.0	79.5	82.3	88.7	88.6	94.1

* Source: Wastewater Department

Centralized Management of Treated Wastewater Reuse in Kuwait: Collection, Storage and Distribution

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CENTRALIZED MANAGEMENT OF TREATED WASTEWATER REUSE IN KUWAIT: COLLECTION, STORAGE AND DISTRIBUTION

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ABSTRACT

The State of Kuwait has a severe arid environment with high evaporation and scanty rainfall. Most of the water supply for the country comes from desalinated seawater the recent industrial and agricultural renaissance in the country has created a condition for the country's need for exploring other possible sources of useable water. Kuwait took several planned steps towards augmenting the water sources. The plan of treating wastewater to levels of tertiary and beyond, and utilizing the effluent for restricted use in agriculture is one of them. The country presently generates in average of nearly 350,000 m^3 each day of treated effluent from three main wastewater treatment plants. To achieve the goal, recently an advanced treatment process including the provision of applying a reverse osmosis (RO) system was built and commissioned for operation. Nearly 20-25% of potential wastewater is utilized for direct irrigation of agricultural lands producing grass (animal feed) and for the irrigation of the trees and shrubs of greeneries and landscaping in the country. Treated effluents from various plants are collected by a system of pipes, pumps and valves to a central storage facility. The Ministry of Public Works (MPW) maintains a special section for the management of effluent storage and distribution systems. Management activities include regulation of flows, monitoring water quality of in and out flows, maintaining the storage places, balancing the flows, adding appropriate chemicals for quality assurance and supplying water to the point of use. In providing these services, additional costs are incurred beyond the cost of normal wastewater treatment. The paper includes the experiences of operation, maintenance and decision-making processes involved in the centralized collection, storage and distribution of the treated effluent in Kuwait. The paper also contains approximate cost estimates for maintaining the service facility.

Keywords: Wastewater-effluent, utilization, facility-management, database, cost

Introduction

The State of Kuwait has a total land area of 17,818 km^2 and is located at the north-east corner of the Arabian Peninsula. The country has an extreme arid climate with a high evaporation rate and scanty rainfall. The mean evaporation rate is about 4000 mm per year while yearly rainfall is about 140 mm (Ministry of Planning (MP), 2002). Temperature varies widely from near zero in winter to 46°C in summer. Estimated mid-year population in 2002 was nearly 2.3 million. Per capita water-use is recorded as 494 l /p/d in year 2001 (Ministry of Electricity and Water (MEW) 2002). Main source of potable water supply is desalinated seawater.

Kuwait has a potential of generating about 500,000 m^3 of wastewater everyday. The figure results from the assumption of mean water use of 360 l /p/d, 2.3 million people and 60% of water-use turning to wastewater. Treated wastewater has great potential for reuse for non-potable purposes particularly considering its total dissolved-solids are significantly less than that of seawater or brackish groundwater. In addition, wastewater effluent water contains valuable nutrients that are sources of valuable fertilizer for plants.

Significant advancements have been made in Kuwait in wastewater treatment and reuse. Most of municipal wastewater in Kuwait is treated to an effluent quality suitable for irrigation purposes. Five existing treatment plants provide treatment up to tertiary level. A centralized new treatment plant has been constructed which provides treatment beyond tertiary level by reverse osmosis and carbon media and renders an effluent quality suitable for unrestricted use for irrigation purposes.

The country undertook an ambitious project of collecting all treated effluent to a centralized reservoir system for effluent storage, and subsequent distribution to points of use. This paper includes the aspects of centralized treated effluent management in the state of Kuwait.

Treated Effluent Quantity

Two main wastewater treatment plants at Reqqa (106,000 m^3/d) and Jahra (54,000 m^3/d) provide up to a tertiary level of treatment. Following pretreatment and conventional secondary level treatment by the activated sludge process, the effluent receives tertiary level treatment through sand filtration and chlorination. In addition, two smaller treatment plants at Umm Al-Himman (approximate capacity 27,000 m^3/d) and Wafra settlements (6,000 m^3/d) exist.

Under the effluent reuse program, presently a modern wastewater treatment plant at Sulaibiya (375,000 m^3/d, peak capacity 525,000 m^3/d) was installed and it started operation in August 2004. It provides ultra-filtration, RO and ultraviolet ray treatment to tertiary effluent (MPW, 1995). It processes presently about 350,000 m^3/day. The plant was built on a built-operate-transfer (BOT) plan arranged with a joint-venture organization. The BOT project is a special project managed by a private organization with a 30 year contract on a built, operate and transfer arrangement with the MPW, Kuwait. Refined effluent from the BOT project is bought at a cost of about KD 0.181/ m^3.

Most of Reqqa treatment plant effluent is used for landscaping in near-by coastal villages. Its annual amount averages to nearly 6.67x10^6 m^3/yr. The BOT and Jahra plants divert effluents to a central effluent reservoir. At 500,000 m^3/d wastewater generation, potential wastewater resource available annually is about 182.5x10^6 m^3/yr. Treated effluent reuse during June 2003-May 2004 was about 23% of the total potential effluent in the country (MPW, DMC, 2004). When compared with annual estimated water-use of 302 x10^{6}m^3/yr (at mean 360 l/p/d), the reused volume is about 13.7% of water supply. The percentage may be slightly higher when wastewater effluents used at treatment plant premises are included.

Treated Effluent Quality

The older operating processes provide treatment to municipal wastewater up to a tertiary level. Secondary processes are conventional biological units of activated sludge while the tertiary level treatment involves sand filtration and chlorination. Sample effluent quality of the Jahra and Reqqa plants appears in Table 1. Table 2 contains sample effluent quality of the new BOT plant at Sulaibiya.

Table 1: Treated Effluent Characteristics of Two (Jahra and Reqqa) Conventional Wastewater Plants in Kuwait.

Parameter	Jahra average, 2000-2004			Reqqa average, 14/2-2/3, 2005)		
	Average	Max.	Min.	Average	Max.	Min.
COD, mg/l	20.5	103	2	22.9	97	0
BOD_5, mg/l	3.8	15.8	0	6.7	13.1	4.3
TSS, mg/l	2.9	51	0.1	5.5	30.5	1.5
VSS, mg/l	2.3	40	0	4.4	24.4	1.2
TDS, mg/l	1142	2000	101	731	835	127
Temp. ^{0}C	25.6	31.7	17.2	22.4	24	20.4
pH	7.2	9	5.4	6.6	7.2	6.3
Oil and Grease, mg/l	0	0	0	0	0	0
Conductivity, us/cm	1754	2310	16	1239	2350	1000
Organic –N, mg/l	1.2	13.4	0	0.8	1.4	0.8
Total Nitrogen, mg/l	6.2	30.7.	0	1.8	2.5	0.9
Ammonia, mg N/l	3.5	30.2	0	0.7	1.1	0
Nitrite, mg N/l	-	-	-	-	-	-
Nitrate, mg N/l	1.5	17.5	0	0.2	0.3	0
Total Phosphate, mg/l	6.5	46.4	0.4	6.2	6.6	1.7
Alkalinity ($CaCO_3$), mg/l	110	215	12	27	58	23
Sulfate, mg/l	325	485	11	148	370	162
Sulfide, mg/l	-	-	-	-	-	-
Chlorides, mg/l	272	370	100	205	103	82
Coliform, Colony/100 ml	3.71E+03	2.5E+02	20	1.5E+02	4E+02	4

Source: DMC Data-base (MPW, 2004), and Records (MPW, 2004)

VSS=Volatile suspended solids, COD=Chemical Oxygen Demand, BOD=Biochemical Oxygen Demand, TDS=Total Dissolved Solids, TSS=Total suspended solids

Central Effluent Collection and Distribution Center

The state of Kuwait maintains a central facility for the collection and distribution of treated effluent (Figures 1 and 2). The facility includes effluent storage tanks, pump houses, chlorination units and a laboratory for water analysis. In addition to regular staff engaged by the Ministry of Public works for the normal functioning of the facility, the Ministry engages a contracting company to maintain the facility structures and equipment. The ministry staff members headed by an engineer oversee the normal functioning of the facility. The center maintains a database for recording flows and the quality of inflows and outflows of the storage tanks.

The central effluent collection and distribution center houses facilities such as reservoirs, a pump house, workshop and maintenance building, an analytical laboratory, a data management facility and office buildings. There are four rectangular reservoir tanks each having an effective storage of 85000 m^3. In addition, there are two circular tanks with storage capacity of 20,000 m^3 each. Total storage capacity at the facility is nearly 380,000 m^3. The effluent users maintain separate reservoirs (Z1 farm 340,000 m^3, Z3 farm 80,000 m^3 and SS farm 1,000 m^3). In total, these facilities add additional temporary storage of 421,000 m^3. The tank flows are independently controlled for sediment cleaning and structural maintenance. Considering the treated wastewater transport through long trunk-lines,the influent to storage tanks is chlorinated at about 2.4 mg/l.

Table 2: Treated Effluent Characteristics of BOT Project with RO Refinement of Conventional Treatment Effluent. Source: DMC Data-base (2005)

Parameter	BOT Plant Consecutive 30 days records, 1st Quarter, 2005		
	Average	Max.	Min.
COD, mg/l	7.8	8	0
BOD_5, mg/l	3.5	10	0
TSS, mg/l	1.2	8	0
VSS, mg/l	1	14	0
TDS, mg/l	40	326	0
Temp. ^0C	22.7	27.7	19.5
pH	7.2	7.6	6.9
Oil and Grease, mg/l	0	0	0
Conductivity, us/cm	64	549	6
Organic -N, mg/l	2.2	5.6	0
Total Nitrogen, mg/l	3.1	15	0.3
Ammonia, mg N/l	0.4	2.2	0
Nitrite, mg N/l	-	-	-
Nitrate, mg N/l	0	0.1	0
Total Phosphate, mg/l	1.9	11.2	0.1
Alkalinity($CaCO_3$), mg/l	17	24	8
Sulfate, mg/l	28	-	1
Sulfide, mg/l	-	-	-
Chlorides, mg/l	9	19	1
Coliform,Colony/100 ml	2.6E+02	1.4E+04	0

VSS = Volatile suspended solids, COD = Chemical Oxygen Demand, BOD = Biochemical Oxygen Demand, TDS =Total Dissolved Solids, TSS = Total suspended solids

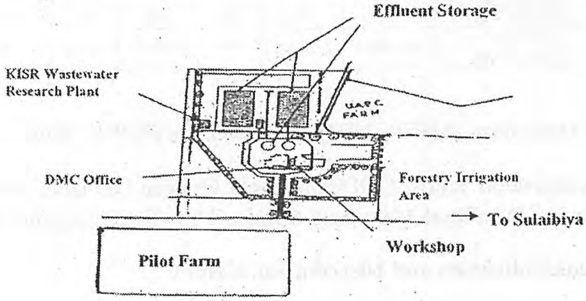


Figure 2. Facilities at Data Monitoring Center

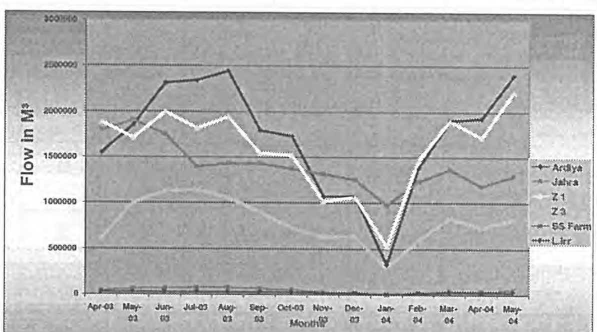
The center is on about 50 hectares of land with boundary fences and a single entry-exit gate guarded by 24 hours surveillance. The site is in a remote desert area about 30 kilometers away from the center of Kuwait city. The nearest residential area is about 10 km away. The wastewater research facility of Kuwait Institute for Scientific Research is located within the DMC (Data Management Center of MPW) premise. The analytical laboratory is well equipped for normal physical, chemical and biological tests of wastewater. The data management section maintains a computerized database of all data collected at the center. A computerized system for remote control of pumps and flows ensures proper in-flow to reservoirs and the distribution of outflows to points of use.

The MPW staff includes an engineer-in charge, operational engineers, technicians and administrative assistants. A contracting company performs all maintenance and day-to-day operation works.

Data Monitoring

The center maintains daily records of reservoir inflows and outflows. Figure 3 shows the typical flow data for average-month for a period of one year (MPW, DMC 2004). Typical cumulative inflows and outflows of the facility for one operating year are shown in Figure 4 (MPW, DMC 2004).

The DMC laboratory regularly analyzes inflow and outflow samples for water quality. Important quality parameters are determined on a daily basis. A summary of daily sample-parameters for the month of February 2003 appears in Table 3. Table 4 and 5 contain the summaries of sample records of chemical and biological parameters (MPW, DMC, 2004) respectively. Table 6 shows the example of sample results of metal analysis for the month of February 2003 (MPW, DMC, 2004). Biological parameters are determined periodically. All data are collected year-round and are recorded and maintained in the DMC database (MPW, DMC, 2004).



Ardiya= Ardiya to DMC, Jahra= Jahra to DMC, Z1 = DMC to Z1 farm, Z3 = DMC to Z3 farm
SS farm= DMC to SS farm, and L.Irr = Flow for local DMC irrigation

Figure 3. DMC Inflow/Outflow from Apr-03 to May-04.

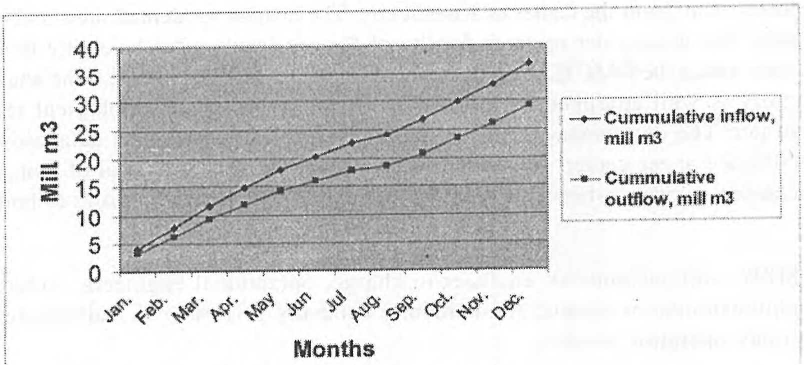


Figure 4. Cummulative in and out flows in the DMC facility (June 2003 - May 2004)

Table 3: Example of Daily Laboratory Analysis (July 2000-December 2004).

NO	TEST	UNIT	JAHRA SUPPLY			DMC FLOW OUT		
			MIN	MAX	AVE	MIN	MAX	AVE
1	Temperature	°C	17.2	31.7	25.6	3	33.1	27
2	pH Value	pH	5.4	9	7.2	3.7	8.7	7
3	Conductivity	mS/cm	16	2310	1754	67	12509	1600
4	Total Suspended Solids (TSS)	mg/1	0.1	56.7	2.9	0.1	25.5	3
5	Volatile Suspended Solids (VSS)	mg/1	0	45.4	2.3	0.1	24	2.4
6	Residue on Evaporation (TS)	mg/1	255	1656	1156	1	1800	1025
7	Chemical Oxygen Demand (COD)	mg/1	2	118	20.5	0	384	23.3
8	Biochemical Oxygen Demand 27/1/2003 to 22/2/2003)	(mg/1	0	16.1	3.8	0	25	4.5
9	Chlorine FreeAvailable	mg/1	0	5	0.17	0	1.5	0.3
10	Chlorine Total Available	mg/l	0.1	8	1.67	0.1	8	0.4

Source: Adopted from DMC Data-base (MPW, 2004), Records (MPW, 2004)

Table 4: Example of Periodic Sample Analysis for Chemical Parameters (July 2000 – Decmber 2004).

NO	TEST	UNIT	JAHRA SUPPLY			DMC FLOW OUT		
			MIN	MAX	AVE	MIN	MAX	AVE
1	Ammonia	mg/l	0	30.2	3.5	0	21.8	5.5
2	Organic Nitrogen	mg/l	0	13.4	1.2	0	16.5	1.9
3	Nitrates	mg/l	0	17.5	1.5	0	12.9	1.6
4	Total Nitrogen	mg/l	0	30.7	6.2	0.1	103	9.2
5	Chlorides	mg/l	100	370	272	20	403	267
6	Sulphides	mg/l	0	0	0	0	0	0
7	Phosphates	mg/l	0.4	46.4	6.5	0.1	227.6	9.4
8	Turbidity	ntu	0	38	4.7	0	29	3.6
9	Chlorine Demand	mg/l				0	11	5
10	Sulphates	mg/l	11	485	325	3	418	262
11	TDS	mg/l	101	2000	1142	1	1729	1024
12	Alkalinity as $CaCO_3$	mg/l	12	215	110	18	863	95
13	Hardness as $CaCO_3$	mg/l	64	723	264	0	429	216
14	Grease and Oil	mg/l	0	0	0	0	0	0

Source: Adopted from DMC Data-base (MPW, 2004)

Table 5: Example of Sample Analysis for Microbiological Parameters (July 2000 - December 2004).

NO	TEST	UNIT	JAHRA SUPPLY			DMC FLOW OUT		
			MIN	MAX	MIN	MAX	MIN	MAX
1	**Total Count**	Number of Colonies in 100 ml	2E+01	2.7E+04	3.7E+03	0	3.38E+04	6.6E+03
2	**Coliform**		0	2.5E+02	3.1E+01	0	1.6E+01	2.32
3	**Fecal Coliform**		0	10	0.9	0	1	0.1
4	**Salmonela**		0	16	1.1	0	0	0
5	**Fecal Streptococi**		0	0	0	0	0	0
6	**Fungi**		0	5	1.8	0	1	0.8
7	**Observation**	**No presence or activity of Nematodes or Parasites**						

Source: Adopted from DMC Data-base (MPW, 2004) and Records (MPW, 2004).

Table 6: Example of Periodic Sample Analysis for Metals, February 2003.

TEST	PLACE OF SAMPLING								
	ARDIYA SUPPLY			JAHRA SUPPLY			DMC FLOW OUT		
	MIN	MAX	AVE	MIN	MAX	AVE	MIN	MAX	AVE
Aluminium	0.0016	0.4992	0.1991	0.0101	0.6211	0.3527	0.0089	0.8473	0.3343
Arsenic	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Boron	0.0	6.3	1.77	0.01	9.6	2.712	0.004	5	1.7577
Cadmium	0.0001	0.0021	0.0007	0.0	0.0006	0.0002	0.00	0.0006	0.0003
Calcium	44.9	66.1	56.06	65.6	88.4	76.2	44.6	80.0	61.68
Chromium	0.0	0.0064	0.002	0.0	0.0002	0.0001	0.0	0.0133	0.0026
Cobalt	0.0	0.0088	0.0026	0.0	0.0006	0.0002	0.0	0.0073	0.0014
Copper	0.0016	0.0118	0.0053	0.0016	0.0042	0.0028	0.0008	0.0041	0.0021
Iron	0.0001	0.0014	0.0009	0.0001	0.0018	0.0009	0.0003	0.0016	0.0009
Lead	0.0005	0.0791	0.0162	0.0008	0.0847	0.0316	0.0001	0.0801	0.0212
Magnesium	12.95	18.35	15.23	23.88	30.53	27.38	14.83	24.74	19.89
Manganese	0.0061	0.0217	0.0132	0.0003	0.0227	0.0107	0.0015	0.0193	0.0101
Mercury	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Nickel	0.0	0.0113	0.0027	0.0	0.0025	0.0012	0.0	0.0045	0.0013
Sodium	46.8	97.5	69.35	69.44	132.05	97.79	55.3	115.0	81.2
Zinc	0.0187	0.0599	0.0350	0.0057	0.0195	0.0131	0.0063	0.0312	0.0174

Source: Adopted from DMC Data-base (MPW, 2004)
All units are in mg/l.

Maintenance works involve routine and preventive maintenance of the DMC premise, civil works, storage facility, and electrical and mechanical systems at the facility. They specifically include laboratory equipment, pumps, motors, valves, exhaust fans, workshop equipment, fire fighting system, chlorine equipment, standby generators, overhead cranes, control panels in the valve house, and routine cleaning of yards and buildings. They also comprise utility service systems such as a telephone system, building, gate and street lighting, gardens, lawns, lawn watering system, telemetry and radio communication systems in the center, clean water supply system, and sewage and sanitary systems. The maintenance of the facility is contracted to an agency on an annual basis.

Costs

It is difficult to calculate absolute cost figures for sewage service, treatment, effluent transfer to central storage, and distribution of treated effluent to sites of reuse due to several factors such as variable service areas of treatment plants, ages of treatment plants, level of treatment provided, capacity of treatment plant, distances of treatment plants from DMC (centralized effluent storage facility), and variable quality of effluent water from different plants.

The Reqqa and Jahra plants are the parts of the old system of wastewater treatment facilities in Kuwait. These systems are completely separated and function as independent projects. The treatment of sewage up to secondary level in these plants is similar and compatible. An estimate based on 1982 figures shows an annual cost of KD 4,401,446 for a combined average flow of 160,000 m^3. The estimate considered straight 20 years' depreciation of mechanical equipment, 40 years' depreciation of civil works, 30 years' depreciation of sewage lines, 5% annual increase in operation and maintenance costs, 400 KD cost for each house connection and 6 persons per house. The population of the State of Kuwait in 1982 was about 1.7 million. The derived cost is equivalent to KD 0.342 per 1000 imgal (4.54m^3) for sewage collection and treatment up to secondary level. The cost for treatment only to secondary level is about KD 0.101/ 1000 imgal (4.54m^3). The remainder of KD 0.241 per 1000 imgal is attributable to sewer network, pump stations and other items related to sewage collection.

The tertiary treatment at Reqqa and Jahra is provided to render the effluent suitable for reuse as an irrigant. The advanced treatment (beyond tertiary) at BOT is directed to improve the effluent further so that the treated effluent can be reused unrestrictedly for irrigation and other purposes. The added costs for rendering effluent for reuse through tertiary treatment (beyond secondary), advanced treatment (beyond tertiary) and transfer of treated effluent to a central storage place are important elements in the State's program of augmenting national water resources through reuse of treated effluent. The scheme of treated-effluent transfer to the Data Monitoring Center (DMC, Centralized storage location) appears in Figure 5.

Figure 5 shows the BOT project in place of the Ardiya plant, which has been abandoned since the new plant, started operation in August 2004. Flow-weighted average costs (1982) for tertiary treatment of secondary effluent and the cost of effluent transfer from plants to central storage were estimated to be 0.0748/1000 imgal (4.54m^3) and KD. 0.03667/1000 imgal (4.54m^3) respectively. It is noted that the estimates for tertiary treatment and effluent transfer to DMC were based on the operating stage of the Jahra and Reqqa plants with 30 years depreciation of equipment.

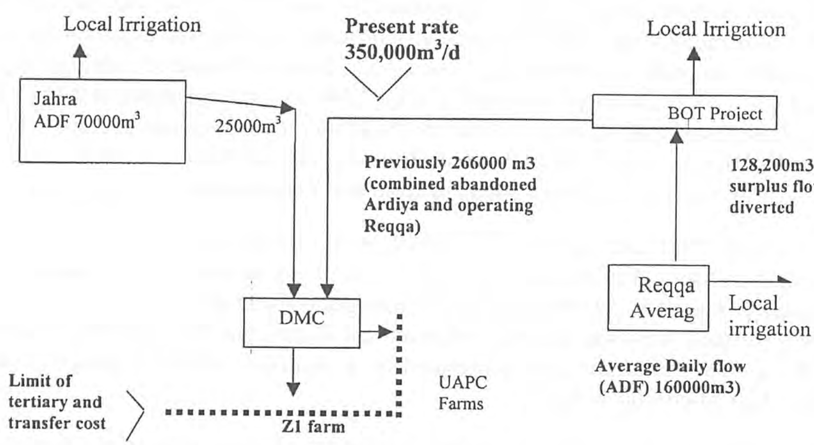


Figure 5: Scheme of Treated Effluent Transfer to DMC (Main Storage Center)

Table 7: Estimated Cost of Wastewater Treatment and Water Supply in Kuwait.

Item	Approximate cost KD per 1000 imgal (4.54m³)	Base year of estimate
Sewage network + treatment up to secondary level	0.342	1982
Treatment only up to secondary level	0.101	1982
Sewage collection, house connection and pumping stations only	0.241	1982
Tertiary treatment only	0.075	1982
Transfer and storage cost to DMC only	0.037	1982
Approximate BOT treatment cost	0.822	2003
Charge for BOT effluent use (Approximate)	0.822	2003
Advanced treatment (ultra-filtration + RO)	0.561	2003
Desalinated pure water (Approximate)	2.8	2003
Charge to public for desalinated water	0.880	2003
Tanker water supply	0.300	2003

Source: MEW, 2002, 2003; MP, 2002, 2003; MPW, 1995; DMC Database (MPW)
1 KD = About 3.3 US$

Discussion

The facility of centralized effluent management for wastewater reuse and data collection has been operating since 1998 without any significant problem. The approximate yearly treated-effluent reuse (last three years average) in three well planned firms namely Z1, Z2 and SS, is 28.7×10^6 m^3/yr. Including average DMC use in its premises of 0.35×10^6 m^3/yr, total use from DMC storage is about 29.05×10^6 m^3/yr while annual average inflow in DMC is about 36.7×10^6. This does not, however, include treatment plant and DMC uses or uses at coastal villages supplied from the Reqqa plant.

In years 2000-2003, total amount of 41.56×10^6 m^3/yr of effluent was supplied from DMC for irrigation uses. The amount was about 23% of estimated potential wastewater in the country. Out of 41.52×10^6 m^3/yr of the managed effluent reuse, about 36.7×10^6 m^3/yr (87% of total use) was directly collected and distributed through DMC facility at Sulaibiya. The remainder was distributed by a separate system of pumps from the Reqqa plant to coastal villages.

Figure 4 showed an example of annual cumulative inflows and outflows from DMC facility for one year starting from June 2003. Treated wastewater reuse during these 12 months was nearly 23% of potential wastewater generation. Yearly records show that the increase in reuse was of a substantial amount during year 2003-2004. It is evident that the supply is presently controlled to keep pace with the inflows as total outflow is always below total inflow. The difference in annual cumulative inflow and outflow was nearly 7.5 million cubic meter. Total storage capacity is nearly one million cubic meters. The lost amount of water is about six million cubic meters. The loss is incurred through the process of transportation, storage, distribution and use. There are several possible reasons for such losses. One of the important reasons is the high rate of evaporation.

Average effluent quality from conventional systems (Jahra and Riqqa plants) appears to be adequate for restricted irrigation use according to WHO and USEPA guidelines (WHO, 1989; USEPA, 1992). The effluent quality from the new treatment plant using RO processes (Table 2) appears to be excellent.

The cost estimates shown in Table 7 are approximate. However, they reflect the relative costs of treating wastewater to various levels. Particularly the use of an RO system for the refinement of treated wastewater effluent is new in its application. The cost of its use for wastewater is deemed by many experts as a discouraging factor. However, its use for rendering the effluent for unrestricted use of the water for irrigation definitely is an added attribute to the effluent. The estimated cost of KD 0.124/ m^3 appears to be reasonable considering the excellent quality of effluent water (Table 2).

Conclusion

The centralized treated-effluent management for wastewater reuse in Kuwait is a unique example in the region. The facility performance is normal without any major problems of management, receive, distribution and quality control. The cost figures appear reasonable. With the newly constructed BOT plant in full operation, the total quantity of wastewater available for reuse is expected to increase significantly.

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Field scale application of a new management technique

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FIELD SCALE APPLICATION OF A NEW MANAGEMENT TECHNIQUE

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ABSTRACT

A major concern in New Zealand is the potential impact on groundwater quality following the application of wastewater to land. The concern is that land application of wastes at excessive rates may cause nitrate leaching and groundwater contamination. It was learned from a field experiment (Mahmood and Wall, 2001) that land application of effluent on a regular basis, even when the nitrogen-loading rate was within fertiliser application guidelines, could still pollute the groundwater. Therefore, there is a need to develop better management options for land treatment systems which can reduce the risk of groundwater contamination. Based on the findings of that field scale experiment, a management technique was developed to make the long-term (seasonal or yearly) management decisions (i.e. to check the long-term effects of land application of wastewater on groundwater quality) at a land treatment system (LTS). This technique forecasts the amount and timing of effluent irrigation by predicting the quantity and timing of future outflows (i.e. leachate volume and leachate nitrate concentration). In this new management technique, LEACHN, a sub-module of LEACHM (Leaching Estimation and Chemistry Model) acts as a decision support system (DSS) that deals with both the hydrological and biological aspects of wastewater-irrigated soils. This paper outlines the concept of the new management technique to make management decisions of "how much effluent to apply" at a LTS, the calibration and sensitivity analysis of the LEACHN model and then the application of the new management technique (using the calibrated LEACHN model) at field scale level. The LEACHN model was calibrated with the field data collected for a land treatment site of Carterton District in New Zealand and then subjected to a sensitivity analysis against several parameters. The new management technique was applied to the Carterton data set (using the calibrated LEACHN model) to test the performance of the technique. It was difficult to calibrate the model because of a lack of measured nitrate data in the root zone, nitrate movement below the root zone to the water table was not evaluated, and the need to relate the predicted concentration in the root zone to measured results in the top few meters of the groundwater table. The calibration results showed that the predicted nitrate-N concentration followed the trend of the measured nitrate-N concentration, and the leaching was well predicted by the model and indicates some dilution occurs in the groundwater. A correlation of 73% was found. Nitrate-N values predicted with LEACHN were relatively sensitive to certain soil physical and chemical properties, such as bulk density, air entry value (a), exponent for Campbell's equation

i.e. BCAM (b), mineralisation rate, base temperature, and Q_{10} which is the factor change in rate with a 10-degree change in temperature. Models are available to predict nitrate-N on an annual basis. This management technique/model predicts nitrate-N leached over a much shorter stage (daily, weekly, monthly). The field scale application of the management technique showed that looking at the mass of nitrate leached (leachate volume x leachate nitrate concentrations) could aid the making of management decisions. This management technique takes account of the soil and plant ability to absorb the applied amount of water and nitrogen.

Keywords: Land treatment system, groundwater contamination, nitrate leaching, LEACHN, Management technique.

Background

A major concern in New Zealand is the potential impact of wastewater application to land on groundwater quality (Di et al., 1998). The concern is that land application of wastes at excessive rates may cause nitrate leaching and groundwater contamination. One management tool for the prevention of nitrate contamination of groundwater at a LTS is the use of solute transport models to assess the possibility of nitrate leaching at the designed effluent irrigation volume. Computer simulation models have become popular tools for studying the transport mechanism of agricultural chemicals through subsurface zones. Simulation studies can be used as an inexpensive, time saving, and environmentally safe technique to evaluate the effects of various agricultural management practices on the subsurface movement of NO_3-N (Singh and Kanwar, 1995a). Over the past several years, a large number of computer simulation models have been developed. However, few data sets are available for testing a range of models, few models have been tested on a range of soils, and very few models have much demonstrable ability to simulate transient field leaching conditions (Addiscott and Wagenet, 1985). Few of the models developed so far are suitable for solving management related problems because of the lack of thorough validation (de Willigen et al., 1990).

The behavior of nitrogen (N) in the soil-plant-water system is very complex. It is dynamic and involves numerous interactions and transformations. Models are useful tools for integrating different processes involved in N transport in soil and can be used in forecasting how a system will behave without actually making measurements in the physical system. In recent years, development and application of models to predict soil water transport, N transformation, and N transport have increased tremendously (Ahmed, et al., 1994). LEACHN is one such model which can be used to simulate field-scale N transformations and movement in the unsaturated zone of the soil profile. LEACHN is one of the five sub-models of LEACHM (Leaching Estimation and Chemistry Model) developed by the Department of Agronomy at Cornell University, USA. Hutson and Wagenet (1992) have given a detailed description of the model. One particular application of LEACHN is the simulation of the leaching of nitrate-N (NO_3-N) and ammonium-N (NH_4-N) from the plant root zone, which is of concern because once beyond the root zone, N is no longer available to plants and thereby has the potential to pollute the groundwater.

Irrigation with effluent is an increasingly popular treatment option due to concern about nutrient addition to rivers and coastal waters. Studies have shown that irrigation with wastewater can lead to contamination of groundwater resources. Therefore, there is a need for a management technique, which predicts the leaching of nitrate nitrogen (NO_3-N) for supporting operations and making management decisions about effluent application (when and how much to apply?) at a LTS. There could be significant economic and environmental effects from such decisions, and it is important that they are based on scientific data. Land treatment of wastewater is expected to produce effluent in the form of leachate from the near surface soil-plant ecosystem which has a low nutrient-concentration and does not exceed quality parameters set by local authorities. Achievement of this performance is usually attained by controlling the input loading to the LTS. The current resource consent procedures in New Zealand recognise this level of technology by prescribing maximum allowable annual inputs of potential contaminants such as total nitrogen. There may be a requirement to monitor the quality of groundwater likely to be affected by the land treatment of effluent, but any detected contamination indicates damage already done and then it is rather too late for effective management of the LTS. This paper outlines the concept of the new management technique to make management decisions of "how much effluent to apply" at a LTS, the calibration and sensitivity analysis of the LEACHN model, and then the application of the new management technique (using the calibrated LEACHN model) at field scale level.

A New Management Technique

The main focus of the new management technique is to forecast the amount and timing of effluent irrigation by predicting the quantity and timing of future outflows (i.e. leachate volume and leachate NO_3-N concentration) from a LTS (Figure. 1).

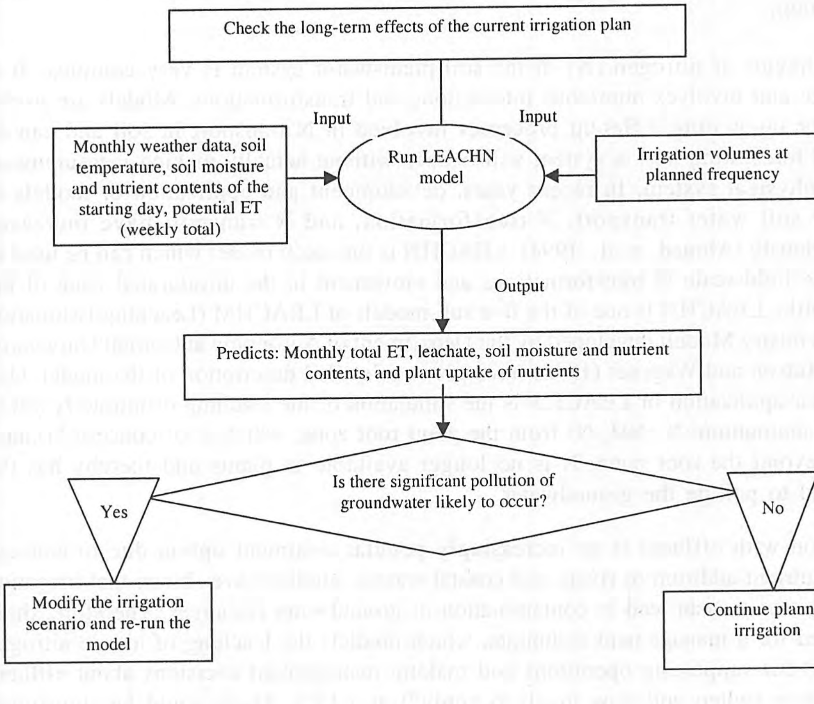

Figure 1: A new management technique to reduce the risk of groundwater contamination at a land treatment system.

These forecasts are critical in planning and controlling the application of effluent onto land. The approach to this research was that monitoring of site-specific factors (SSF, i.e., relevant to rainfall, effluent, soil, plant, and water) is necessary for the effective control over the influence of NO_3-N on the local groundwater quality. The ultimate objective of the new management technique was to apply effluent on land up to the optimum quantities (mm/week) so that the groundwater would not be contaminated. This management technique takes account of the soil and plant ability to take up the applied water and nitrogen. Using this technique, management decisions can be made by looking at the soil's ability to absorb the designed irrigation volume, the NO_3-N concentration in the leachate, and the plant's ability to take up the applied amount of water and nitrogen. In this technique, the LEACHN model predicts the information on which to base a management decision. Monthly weather data, initial water and nutrient contents (SMC and SNC) of the soil profile, and the design effluent irrigation volume are the input parameters to the model. It predicts SMC and SNC, actual evapotranspiration (ET), plant uptake of nutrients, leachate volume and nutrient concentration in ppm in the leachate.

To check the long term effects of the current irrigation plan (seasonal or yearly), the SMC and SNC on the starting day along with the weather scenario and current irrigation plan (designed hydraulic loading rate) are given to the LEACHN model (as the input parameters). The leachate's volume and NO_3-N concentration, as part of the model predicted information, are used to predict the groundwater contamination. If the NO_3-N concentration in leachate is below the World Health Organisation (WHO) maximum permissible limit (MPL) of 11.3 mg/l in drinking water (i.e. no risk of groundwater contamination), then the planned irrigation can be applied. If the leachate volume and the NO_3-N concentration in the leachate exceed the MPL (i.e. there is a risk of groundwater contamination), then the planned irrigation scenario is modified and the model is re-run.

Methodology

Experimental site

A field scale experiment was conducted at a land treatment site of Carterton District in New Zealand. The purpose of this field experiment was to examine the effect on groundwater quality of the irrigation of sewage effluent onto trees and pasture at different application rates. The experiment began in December 1997 and continued through to August 1998. The study area consisted of 0.162 ha, planted with two-tree species (two years old Eucalyptus-nitens (E-N) & Eucalyptus-ovata (E-O) and pasture. There were three blocks, each of 540 m^2 (Figure 2). Each block consisted of 3 plots (2 plots of trees randomly planted and one control plot of pasture). The area of each plot was 180 m^2. A trickle irrigation system was designed to apply effluent at three designed hydraulic loading rates of 30, 45, and 100 mm per week, i.e. low (L), medium (M), and high (H) treatments, respectively. Five monitoring wells were installed at the site. Three were in each effluent irrigation treatment, i.e. wells, 1, 2 and 3. The fourth one (well 4) was in the high treatment pasture plot, and the fifth (well 5) on the upstream side of the experimental site between an oxidation pond and plantation area. All monitoring wells were extended 1 m above the ground level, with the screened section starting 1 m below ground level. Monitoring wells 3 and 4 on the downstream were extended only 3 m below ground level due to caving in of material during excavation, but the rest extended to the depth of 5 m. The location of these wells was determined on the basis of the piezometric survey conducted by Good Earth Matters Consultants. Monitoring of land treatment site-specific factors (SSF) relevant to the climate, effluent, soil, plant, and groundwater was undertaken. The climate data (rainfall, pan evaporation, and temperature) was collected on a daily basis from a nearby weather station. The soil moisture content (S.M.C) measurements were made at three depths ranges i.e. 0-150, 0-300, and 0-500 mm prior to irrigation and then fortnightly in all treatments of tree and pasture plots. A Kent flow meter was used to measure the amount of effluent added to the system during each irrigation cycle. Flow meter readings were taken to check that the system was running at the designed application rates. Soil-water samples were collected from the suction cups installed at the same depths prior to irrigation and then fortnightly. The effluent, soil-water and groundwater samples were collected prior to irrigation and then fortnightly. The samples of effluent, soil, soil-pore water, plant and groundwater were monitored in terms of nitrogen. The samples were stored at 4°C and then analyzed for nitrate-nitrogen (NO_3-N) and ammonium-nitrogen (NH_4-N) using the standard methods of water analysis (Gillian, 1984). Soil available nitrogen, i.e., NO_3-N and NH_4-N, soil bulk density, soil pH, and organic soil carbon was determined at the same depths in all treatments of the tree and pasture plots before and at the conclusion of the experiment. The soil at the site was a stony silt loam from the recent fluvial deposits associated with the Mangaterere River and tributary streams. The land treatment area was located immediately adjacent to the Carterton sewage treatment oxidation ponds. The details of the field experiment can be found in Mahmood and Wall (2001).

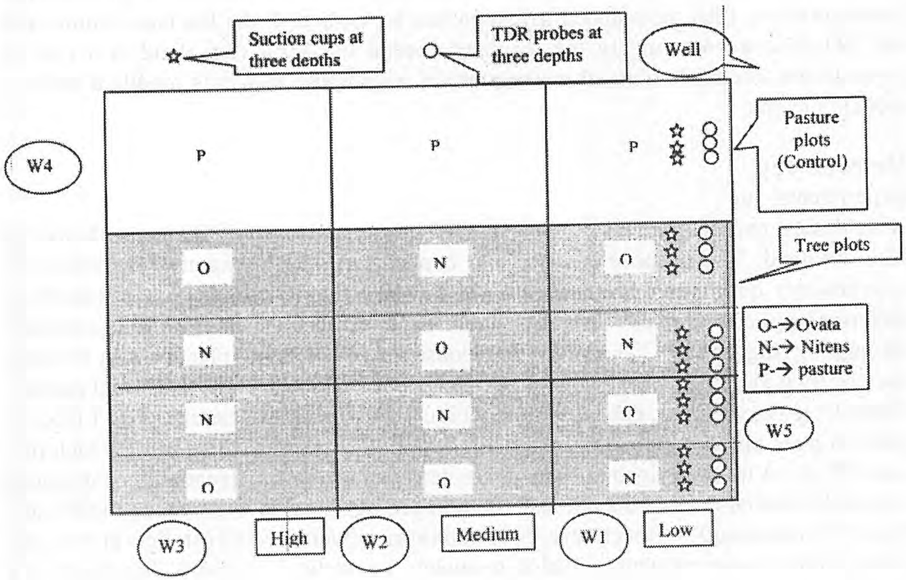

Figure 2. The land treatment site of Carterton District in New Zealand.

LEACHN Calibration Process

The calibration process focused on input parameters controlling leachate nitrate concentration in the root zone (0 - 500-mm soil depth). The 500 mm depth was chosen because of data availability for the same depth, plus monitoring of moisture content just below the root zone may alert producers or land managers to potential nitrate leaching. It was difficult to calibrate the model at field scale level because the actual drainage volume could not be measured and the groundwater composition reflected effects from all of the treatment options at the site. Moreover, we experienced some difficulty in getting the soil solution samples from the suction cups (lack of measured nitrate data in the root zone) and only a few soil solution samples could be obtained from suction cups installed at different depths at the Carterton experimental site, details can be found in Mahmood and Wall (2001). Therefore, the measured groundwater nitrate concentration data collected from the medium treatment of the tree plots was used to calibrate the LEACHN model. The predicted nitrate level at the 500-mm depth was compared to the nitrate level observed in the groundwater (groundwater level varied between 1 and 5 m from ground surface). If the nitrate level at 500 mm was consistently above the observed nitrate level in the groundwater, then this would indicate additional dilution in the groundwater zone. The depth-weighted nitrate concentration from ground surface to groundwater level, for two dates 1 June and 14 June 1998, were also estimated by multiplying the average predicted leachate nitrate concentration at different depths and thickness of each soil layer. The observed groundwater nitrate level and predicted leachate nitrate concentrations at 500 mm depth were also graphed for the whole irrigation period. The medium treatment of the tree plots was chosen because it was in the middle of the tree plantation area, and also for the reason that there would be less effect of seepage from the oxidation pond and the stream on this plot. The data from 2nd June-1998 to 18th August-1998 was chosen for the calibration process. The calibration period consisted of 77 days. The input parameters used to calibrate the model are given in Table 1. The input parameter values required by the model were obtained from direct field and laboratory measurements and literature sources where selected model parameters were adjusted within an expected range and the discrepancies between the measured data and model predictions were minimized. The air entry value, BCAM, nitrification rate, denitrification rate, and humus mineralisation rate constants were adjusted one by one within the expected range to minimise the discrepancies between the measured and predicted values.

Table 1: A list of parameters utilised in the calibration process.

Parameters	Units	Values/Range
*AEV (a)	kPa	-1.0
**BCAM (b)	dimensionless	3 - 5 (different for each soil segment)
Clay particles	(%)	20
Silt particles	(%)	12
Organic Carbon	(%)	2.50 - 4.50 (different for each soil segment)
Initial SMC	(%v/v)	26 - 29 (different for each soil segment)
Bulk density	(kg/dm^3)	1.00 - 1.44 (different for each soil segment)
C/N	dimensionless	10:1
NH_4-N	(mg N/kg)	2.63 - 4.00 (different for each soil segment)
NO_3-N	(mg N/kg)	3.85 - 6.38 (different for each soil segment)
Crop cover	0.8	
K_d for NO_3-N	l/kg	0.00
Nitrification rate	day^{-1}	0.02 - 0.09 (different for each soil segment)
Denitrification rate	day^{-1}	0.02
Humus mineralisation rate	day^{-1}	0.00007

**AEV = Air entry value (a)*
***BCAM = Exponent for Campbell's equation (b) (Campbell, 1974)*

The measured rainfall, amount of effluent irrigation, and bulk density values were inputted to the model. The measured nitrogen compositions of the effluent applied (i.e. NO_3-N, and NH_4-N concentrations in mg/l) were given to the hydrology portion of the model along with the measured rainfall and amount of irrigation. The nitrogen concentration in the rainwater was considered to be zero (rainwater is pure water and nitrogen is almost zero in rainwater). The estimated weekly potential evapotranspiration (PET) values were also inputted to the hydrology portion of the model along with the mean weekly measured maximum/minimum air temperature values. The soil layers were assigned the clay, silt, inorganic nitrogen and carbon pools (mg/kg of dry soil), organic carbon content (%), and SMC (%) measured at the site. The air entry value (AEV - a) and b (BCAM) exponent for Campbell's equation (Campbell, 1974) were taken from the literature (Lesikar et al., 1997). No data was collected for humus-N, an input to the model. Initial values of humus-N were estimated from the measured initial carbon pools using the C: N default ratio of 10:1 in the model. Most of the N transformation, dispersivity, and hydraulic conductivity data used by the model developers were retained as no data could be obtained. Minor adjustments (from the default values) were made for parameters used in the transport and transformation of nitrogen, e.g. nitrification, denitrification and mineralisation rate constants. Some of these parameters had different values at different depths for the same soil profile and the range of values is shown in Table 1. As mentioned above, simulation accuracy of the LEACHN model was evaluated on the basis of its ability to predict NO_3-N concentration in the leachate at 500-mm depth. The data was summarised both graphically and numerically (statistical methods). The statistical methodologies suggested by Loague and Green (1991) were used to evaluate the prediction capabilities of the model. These statistical measures included root mean square error (RMSE) and correlation co-efficient (R) between the measured and predicted nitrate concentrations. The predicted and measured NO_3-N concentrations were graphed.

$$RMSE = \left[\frac{\Sigma(P-M)^2}{n}\right]^{0.5}\left[\frac{100}{M_m}\right]$$

where

P = predicted value
M = measured value
M_m = measured mean
n = number of measurements

RMSE is a measure of the deviation of the predicted from the measured value and ideally it should be 0. The correlation coefficient is a measure of the degree of association between the predicted and measured values. A lower RMSE and higher (R) value indicate a better agreement between measured and predicted values, but it is necessary to verify this with graphical results.

Sensitivity Analysis - A sensitivity analysis was performed on the calibrated LEACHN model by changing the value of input parameters including soil moisture content (% v/v), air entry value (a), exponent for Campbell's equation (BCAM, b), soil organic carbon level (%), bulk density (kg/dm^3), nitrification, denitrification, and humus mineralisation rates, base temperature at which rate constants apply, and Q_{10} factor by which rate constant changes per 10°C increase. The value of each input parameter was increased by 30%. Model sensitivity to changes in these parameters was evaluated on the basis of their impact on the amount of nitrate-N in the leachate (mg/l) at 500-mm depth on the 70th day. Model sensitivity to changes in parameters was evaluated only for the medium effluent treatment of tree plots (i.e., 45 mm/week).

Application of the Management Technique at Carterton Site - The management technique was applied to the medium treatment of the tree plots. The designed irrigation volume (i.e. 45 mm/week) for the medium treatment of tree plots and the weather data (rainfall and potential evapotranspiration - PET) measured at the site was used as input to the model. Different irrigation and weather (rainfall) scenarios were given to the model. The rainfall data used for all scenarios was the same. The management technique was run for 260 days to predict the mass of nitrate-N leached (M) in (kg/ha), i.e., leachate volume (mm) x nitrate concentration in the leachate (mg/l), on which to base management decisions. A seven-day period was chosen to add a moving average trend line on the graph. The designed irrigation volume was modified by looking at the mass of nitrate leached (kg/ha).

Results and Discussions

Calibration Process - Minor changes to the b (BCAM) exponent were made to adjust the drainage time and these changes were within the range (i.e. 0.14 - 13.3) as given by Lesikar et al. (1997). All the calibrated rate constants were within the acceptable range as reported by many researchers (e.g. Hagin and Welte, 1984; Johnsson et al., 1987; Misra, et al., 1974; Myrold and Tiedje, 1986; Wagenet, et al., 1977, and pers. comm. John Hutson - developer of LEACHM). The calibration results showed that the predicted nitrate concentration followed the pattern of the measured results (Figure 3). Calibration of LEACHN using the 1998 Carterton data set (June to August) for the medium treatment of tree plot resulted in a estimated RMSE value of 51 and correlation coefficient, i.e., R= 73%. The predicted nitrate concentration in the leachate at 500 mm soil depth was between 18 to 25 mg/l. The results revealed that the model over-predicted the nitrate concentration throughout the calibration process. This could be accounted for by dilution of the leachate when it reaches the groundwater. The maximum difference between the predicted and measured values was 8.3 mg/l. A list of parameters used in the calibration process is given in Table 1.

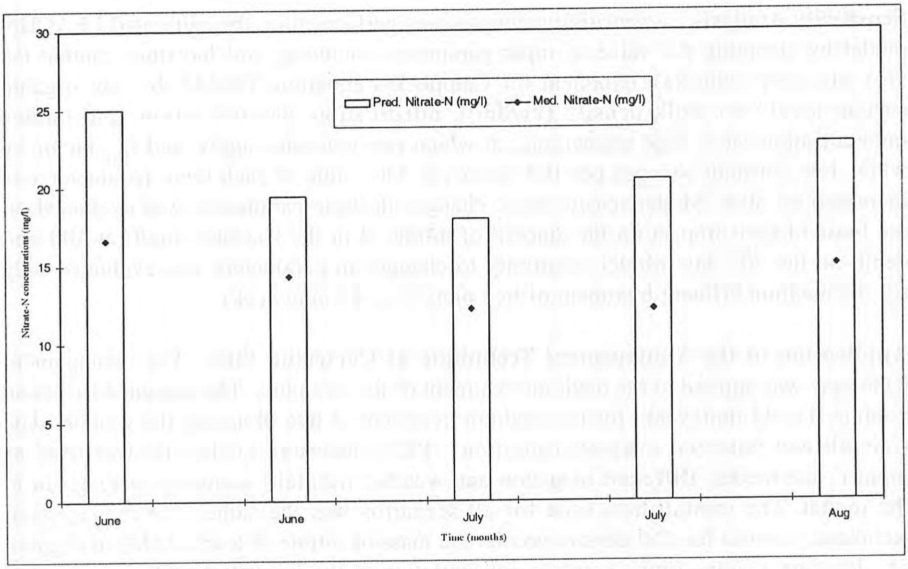

Figure 3. The predicted and measured nitrate concentrations in the calibration process.

The results also showed that the estimated depth-weighted leachate nitrate concentrations (from ground surface to water table level) were 18 and 14.7 mg/l, which were almost the same values as the observed groundwater nitrate concentrations of 16.6 and 14.7 mg/l for the two dates (1 June and 14 June 1998). Figure 4 showed the predicted leachate nitrate concentrations at different depths from the ground surface to the groundwater table. The top 500 mm were the predicted values. There were no values down to the water table and then there was an observed value of 16.6 for the screened zone (more than 2 m) in the water table.

Figure 4 showed that there was an increase in the predicted leachate nitrate concentrations and then a decrease. Perhaps this is because the model only predicts matrix flow and it took about 100 days (Figure 5) for the concentrations to reach a peak at 500 mm and, at some point, effluent irrigation ended and concentrations started to decline. The fact that it took some time to wet-up the profile (Figure 5) suggests that there was a lot of matrix flow and only a small amount of the flow was preferential flow. The problem could be that the screened depth was large so small pulses of nitrate due to preferential flow only change the groundwater nitrate concentrations by a small amount (due to the dilution factor). This means that the variation between the predicted and observed (Figure 3) could be due to a variety of flow processes and the dilution factor.

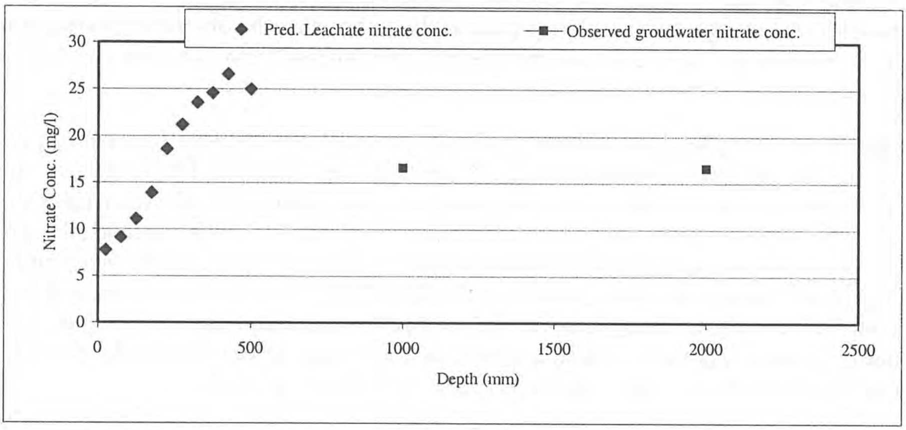

Figure 4: The nitrate concentration changes with depth from the ground surface to the water table.

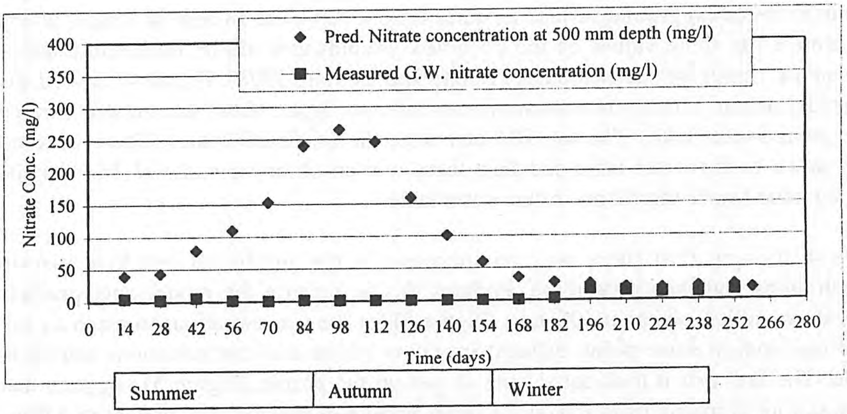

Figure 5: The predicted nitrate concentration values at 500 mm and measured groundwater nitrate concentration of the medium treatment of the tree plots (well 2).

Figure 5 showed that during summer and autumn the leachate nitrate concentration was high ranging from 49 to 265 mg/l in the medium treatment of the tree plots due to the low drainage volume. The leachate nitrate values predicted by the 'calibrated' model in the unsaturated zone were reasonably higher than the observed groundwater nitrate values for more than six months prior to June of 1998 (i.e., the calibration period). This confirms the approach used to calibrate the model.

Sensitivity Analysis - The model sensitivity to hydrology and nitrogen parameters was evaluated using predictions of nitrate-N in the leachate. The results of the sensitivity analyses demonstrated that LEACHN was sensitive to selected parameters (Table 2). Table 2 shows that initial soil moisture content (%), organic carbon (%), and nitrification and dentrification rates had a minimum effect on the nitrate-N concentration predictions. Nitrate-N values predicted with LEACHN were relatively more sensitive to the bulk density, air entry value (a), BCAM (b), mineralisation rate, base temperature, and Q_{10} factor. A positive variation represents an increase in the nitrate-N predictions and negative variation represents a decrease in nitrate-N predictions.

Parameter	%age increase in input parameter =A	Output NO_3-N (mg/l)	%age Change in output =B NO_3-N (mg/l)	Sensitivity =B/A
Initial SMC(v/v)	30	41.6 42.1	1.2	0.04
A (AEV)	30	41.6 38.8	-6.7	-0.22
B (BCAM)	30	41.6 37.9	-8.9	-0.30
Bulk Density (kg/dm^3)	30	41.6 73.9	77.6	2.59
Organic Carbon (%)	30	41.6 42.4	1.9	0.06
Nitrification rate	30	41.6 42.1	1.2	0.04
Denitrification rate	30	41.6 41.3	-0.7	-0.024
Humus mineralisation rate	30	41.6 49.3	18.5	0.62
Base temperature	30	41.60 33.20	-20.2	-0.67
Q_{10} factor	30	41.6 46.6	12.0	0.40

* *Variation in the output parameter was observed on day 70.*

Bulk density is a good indicator of soil compaction. As with all soil properties, bulk density varies spatially, and measurements from a number of locations are required to characterize a site. Bulk density usually also changes significantly with depth, and soil samples from different depths down the profile can be characterized. In this case, the predicted nitrate-N values were more sensitive to bulk density and it could be due to the initial and boundary conditions, initial carbon and nitrogen pools. The sensitivity analysis was undertaken for a short period of time (as limited data was available for this site). A further investigation is required to look at the affect of changing the initial and boundary conditions, carbon and nitrogen pools, and in order to undertake the sensitivity analysis for a longer period of time.

Application of Management Technique at Carterton Site - It was assumed that the groundwater nitrate-N concentrations should not exceed the maximum permissible limit of 11.3 mg/l at all times of the year. If the leachate nitrate-N concentration is high but the leaching volume is low then there is not a problem. Because of the dilution factor, i.e., the low leachate volume will be diluted with the high volume of non-static groundwater. At the site, it was observed that the groundwater nitrate-N concentration during June (i.e. 16.6 mg/l) exceeded the maximum permissible limit (MPL) of 11.3 mg/l. This increase in groundwater nitrate-N concentration was due to the high soil moisture level (i.e., 32 - 35%) in response to rainfall and to the mobile nature of nitrate-N with water in soil-water matrix. The LEACHN model was used to predict the mass of nitrate-N leached (M) in kg/ha/day during June, and the predicted mass of nitrate-N leached was reduced by the required percentage (by reducing the irrigation volume) to reduce the nitrate-N level in the groundwater by the same percentage. A reduction factor of 32% (i.e., [16.6-11.3/16.6] x 100) was found to reduce the groundwater nitrate level from 16.6 mg/l to 11.3 mg/l. The mass of nitrate-N leached (kg/ha/day) predicted during June was 5 kg/ha/day, which was reduced by the same percentage (i.e. 32%) to estimate the maximum permissible limit of mass of nitrate-N leached (MLM) in kg/ha/day. In this case, the estimated MLM was 3.4 kg/ha per day. The purpose of doing this exercise was to calculate the MLM because guidelines are available for the groundwater nitrate-N concentrations (i.e. 11.3 mg/l) but no guidelines are available for the corresponding mass of nitrate-N leached. It was assumed that the weekly average mass of nitrate-N leached should not go above the MLM (i.e. 3.4 kg/ha). If the mass of nitrate-N leached exceeded the MLM on consecutive days, it means this mass of leached nitrate is going to affect the groundwater conditions, and the groundwater nitrate-N concentration may exceed the MPL of 11.3 mg/l. Therefore, modification of irrigation volume is required to reduce the risk of groundwater contamination.

The results of the management technique run (Figure 6) showed that from December-97 to March-98 the weekly average mass of nitrate-N leached was below the MLM (i.e., 3.4kg/ha). Thus, there was no risk of groundwater contamination during this time of the year because of low leachate volume and high leachate nitrate-N concentrations. Two high peaks of approximately 5 kg/ha/day (i.e., > MLM) were found during early December and January. If the mass of nitrate-N leached exceeded the MLM (i.e., 3.4 kg/ha) just for a particular day or alternatively for two days, then it may not matter because of dilution in the groundwater. If the level is consistently above the MLM, then groundwater composition will change.

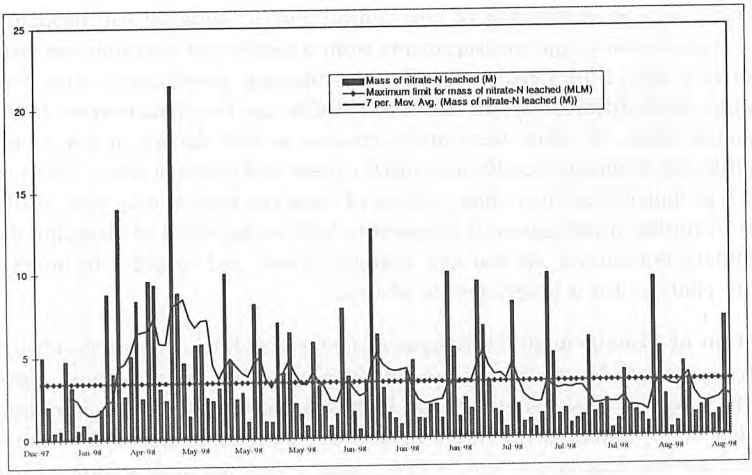

Figure 6: First run of the management technique.

The moving average trend line in Figure 6 showed that early April to the end of May is the crucial time when the weekly average mass of nitrate-N leached exceeded the MLM (i.e., 3.4 kg/ha). In the second run (Figure 7), the designed irrigation volume was reduced from 45 to 30 mm in order to reduce the risk of groundwater contamination during this time of the year (early April to end of May). Figure 6 showed that during the months of April and May the weekly average mass of nitrate leached fluctuates between 3 and 5 kg/ha. This mass of nitrate-N leached is not going to affect the groundwater conditions because of the dilution of the leachate when it reaches the groundwater. Few consecutive peaks of mass of nitrate-N leached greater than the MLM (i.e., 3.4 kg/ha) were found during the month of June (Figure 7). In the third run of management technique (Figure 8), the designed irrigation volume was reduced from 45 to 25 mm in order to reduce the risk of groundwater contamination in the month of June. The moving average trend line for mass of nitrate-N leached showed (Figure 8) that there were only two days during May and end of July when the mass of nitrate-N leached exceeded the MLM, but for the rest of the period it remains below the MLM (i.e., 3.4 kg/ha/day). Thus, it was safe to apply the designed irrigation volume during this period of the year at the site.

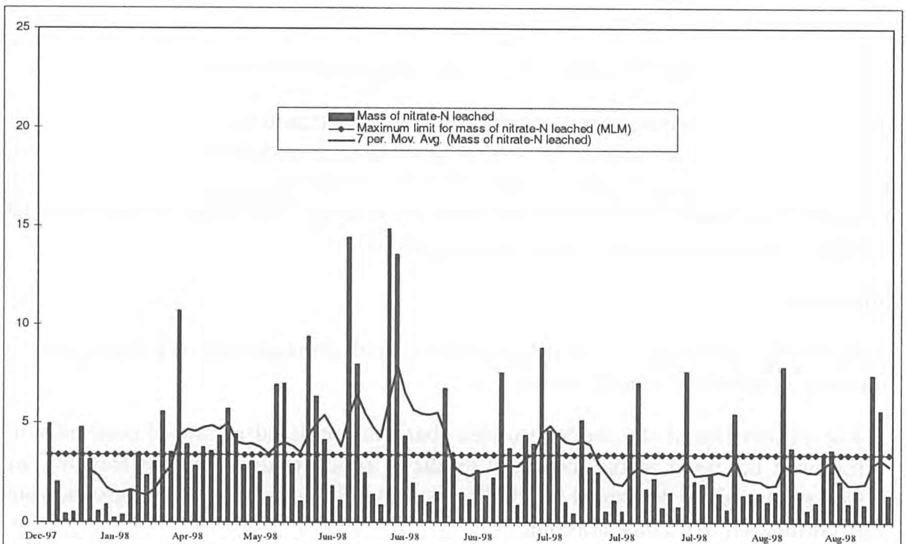

Figure 7: Second run of the management technique.

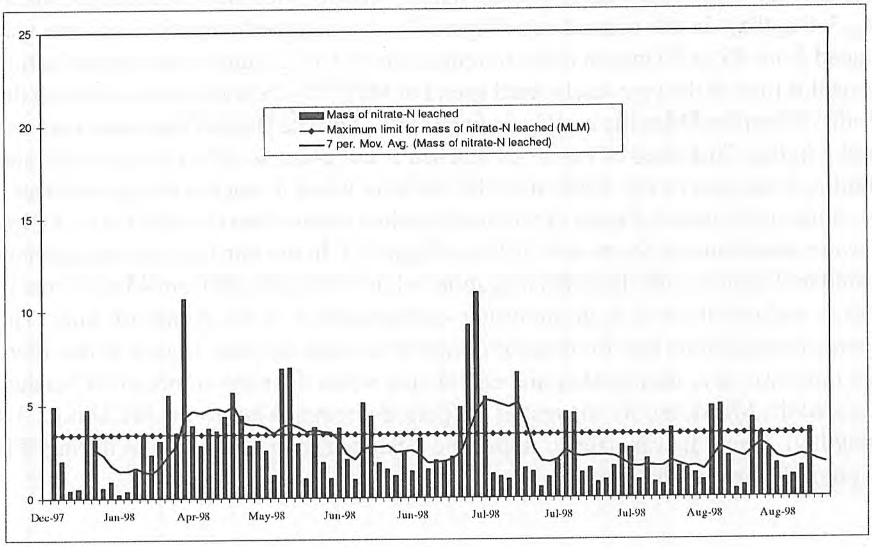

Figure 8: Third run of the management technique.

Application of the management technique to the Carterton land treatment site showed that it was possible to reduce the risk of groundwater contamination at the site by modifying the designed irrigation volume. It means that using with this technique the management decisions at LTS could be made by looking at the mass of nitrate leached (leachate volume x leachate nitrate concentrations).

Conclusion

Based on the calibration, sensitivity analysis, and management technique runs the following conclusions can be made:

- The calibration of the model showed that the predicted nitrate-N concentration followed the trend of the measured nitrate-N concentration, and the leaching was well predicted by the model that indicates some dilution occurs in the groundwater. A correlation of 73% was found.
- The model was not very sensitive to most of the changes with respect to nitrogen. The model was relatively more sensitive to the bulk density, air entry value (a), BCAM (b), mineralisation rate, base temperature, and Q_{10} factor than the initial soil moisture content (%), organic carbon (%), and nitrification and denitrification rates.
- Models are available to predict nitrate-N on anannual basis. This technique/model predicts nitrate-N leached over a much shorter stage (daily, weekly, monthly). This management technique is an interactive tool to make management decisions. The field scale application of the management technique showed that with use of this management technique, management decisions can be made by looking at the mass of nitrate leached (leachate volume x leachate nitrate-N concentrations). This

management technique takes account of the soil and plant ability to absorb the applied amount of water and nitrogen. This technique is conservative and safe because long-term management decisions can be made based on NO_3-N concentration in the leachate (as one of the output of the model). Whenever NO_3-N concentration in the leachate is above the MPL (i.e., 11.3 mg/l), planned irrigation volume should be reduced or modified. With the use of this management technique it might be possible to reduce the number of monitoring wells and the level of monitoring (frequency of monitoring) which means, in the future, a reduction in the cost of monitoring. The initial cost may be high but there will be long-term effects on the cost of monitoring.

Further Research: The developed new management technique was tested for a different soil and effluent at a laboratory scale level and the findings of which have been published elsewhere. Further Research is in progress to undertake a critical analysis of the new management technique in 'predictive mode' using the calibrated model.

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Wastewater Technology in Yemen

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ABSTRACT

In Yemen, due to the urban development and high population growth, the water and wastewater networks were constructed during the eighties. Serving the water to the households has led to the need for wastewater disposal means. Cesspits have been applied as wastewater disposal systems. In most cases, the cesspits are flooded into the streets in several main and secondary cities, while in some other cases; they were connected to an overloaded wastewater treatment system which overflowed with the pollution. In some other places, there is no wastewater treatment and the wastewater is discharged into the sea as the case of Al-Mukalla. On the other hand, the extension in building more wastewater treatment plants has resulted into production of more sludge which has become cumbersome in most cases, especially for the case of mechanical treatment such as the case in Sana'a and Ibb where there are hundreds of tons which could not be disposed. The partial mechanical systems such as trickling filters preceded by imhoff tanks as the case in Hajjah and the natural systems such as waste stabilizations systems as the case in most of the cities in Yemen have proved to be less problematic in terms of sludge production as well as in operation and maintenance. The objective of this paper is to review the existing situation, efficiencies and problems of sanitation in the Republic of Yemen, the activities and management considered by the operators, and finally suggestions for better improvement.

Key words: Yemen; Wastewater; Technologies; imhoff tank, Activated sludge, Trickling filter, sludge, treatment

Introduction

It is estimated that 75% of the population in Yemen lack service for wastewater while 50% of them cannot get drinking water. This percentage is slowly decreasing with time due to the continuous population growth and urban development which does go with extension of services at the same rate. For example, the percentage coverage in urban areas was decreased from 50% water; 26% sanitation in 2000, to 47% water and 25% sanitation in 2002.

Water scarcity in Yemen and its depletion is considered as a reason which led into not enough water reaching consumers. The consumption rate in rural mountainous places is around 30l/cd while in the urban mountainous area it is to 70 l/cd with a little bit more in the coastal hot areas. This low water consumption has led into an increased concentration in the pollutants in the wastewater. The wastewater BOD in the mountainous cities is three to four times the highest known concentration in the world while it reached only two times that of the coastal areas, although there is enough water available to some extent.

Wastewater treatment receives a high priority to control environmental pollution and the spreading of diseases. This is particularly true in Yemen where the urban wastewater flows have increased as a result of rapid urban development, and domestic sewage is characterized by a high BOD of up to 1300 mg/L, due mostly to water shortage and associated low water consumption as well as a discharge of **sullage** from cesspits and slaughter houses without any pretreatment.

Objective of this paper

This paper aims at reviewing the existing situation of the sanitation in the Republic of Yemen and will emphasize the different technologies and problems in sanitation in Yemen, with an evaluation of three different types of the systems adopted in terms of wastewater and sludge treatment and the possibility of reuse of wastewater and sludge after treatment.

Technologies and problems in sanitation in Yemen:

In some rural areas, dry sanitation is still applied which separates the solid part (**Faeces**) from the liquid (urine, washing, cleaning and ablution) with utilization of the water for irrigation while solids are used as fertilizer. However, due to the increasing the use of water and the unhygienic situation of the application of the solid part, this method is almost abandoned from the urban areas but is still being used in some rural areas.

Therefore, the sanitation problem in rural areas is considered a big problem where the solid part is thrown to the backside of the houses or buried in holes in the ground, which then becomes a health hazard to the population. In some rural and urban areas, cesspits are used as a means of wastewater disposal. Even in some places the people are using the dried drinking water wells as a cesspit which will be a threat to the groundwater basins.

Other factors which are considered a burden to successful treatment system operation is that the used mineral oil from mechanical equipment (power stations, vehicles, factories,....etc) is disposed into the wastewater network and then to the WWTP. With the absence of the oil treatment at the WWTP, this situation has led operators to use a bypass and discharge the raw wastewater to the wadi without treatment.

As there is only a limited number of industries who have their own pretreatment systems, the rest are discharging their wastewater into the public network without prior treatment with no monitoring system applied which could cause in some cases the death of the bacteria. In some other cases, the industries discharge their wastes into the cesspits directly which create a risk to the soil and the groundwater.

The treatment methods used in Yemen

In Yemen, three different technologies are used for primary and secondary (biological) wastewater treatment as shown in Table 1, as follows:

- Full mechanical system such as activated sludge systems (AS) as the case in Sana'a and Ibb cities in Yemen.

- Partial mechanical system such as trickling filters (TF), preceded by **Imhoff** tanks (IT) as the case in Hajjah city

- Natural system, mainly waste stabilization ponds (WSPs) as the case in most of the Yemen cities such as Aden, Amran, Rada'a, Al-Hodiedah, Dhamar, Yarim, Taiz...)

A lot of problems are facing the sanitation sector in the operation and maintenance of the full and partially full mechanical systems due to the need for high skilled labor, to the high cost of operation and maintenance and to the high cost of spare parts and their unavailability in the local market or that the manufacturers have stopped production of such spare parts. The need for experts to operate and maintain these complicated systems and the need for continuous training of the operators is considered as the second difficulty after the spare parts.

On the other hand, WSPs are easy to operate and maintain and their effluent is considered safe for irrigation provided that the retention time is at least higher than 10 days in order to have enough time for treatment and subjection to the sunshine (Veenstra et al., 1995).

Although these systems are easy, the water and wastewater corporations face a lot of difficulties in having enough available areas, which stands as a problem preventing the introduction and extension of these systems. Moreover, the corporations are not taking seriously the proper operation of these systems although; it is very simple as the case in Al-Hodiedah, Taiz and Dhamar.

Other problems facing the treatment process is the scarcity of water and lower consumption which is increasing with time. This low water consumption causes an increase in wastewater pollutant concentrations. For example, BOD was increased from 500 mg/l at the eighties to 800 in the nineties and 1200-1300 at beginning of the millennium 2000. This increase caused a malfunctioning of the treatment plants that have been designed in the eighties and constructed at the end of nineties such as the Ibb and Sana'a treatment plants (Table. 1). These WWTP became overloaded organically before the full hydraulic capacity was achieved and before the designed period was reached.

Table (1) Existing wastewater treatment systems at different cities in Yemen

Variables	Units	Sana'a	Ibb	Hajjah	Aden *	Amran	Rada'a	Al-Hodiedah	Dhamar	Yarim	Taiz
Used Technology	-	Activated sludge		Imhof tank followed by two stage-trickling filters	Waste stabilization ponds						
End Design period	Year	2005	2005	2005	2013	2005	2010	2000	2005	2005	2000
Situation of the WWTP	---	OVER LOADED		IN THE RANGE OF THE DESIGN CAPACITY				Over loaded	In the range of the design capacity	Just started	Over loaded
Start operation	Year	2000	1993	1998	2000	2002	1995	1983	1991	2003	1983
Actual flow rate (July 2004)	m^3/d	36000	7 000	1200	17000	1100	1500	18000	6 000	---	8000
Design flow rate	m^3/d	50,000	5256	2428	70000	1480	1881	12000	11000	1771	9000
Design BOD	mg/l	500		843	312	800	500	280	1000	700	500
Actual BOD	mg/l	1200	1316	1100	372	1518	900	582	900	962	1000
Reuse of effluent	---	Restricted irrigation									

*In addition to this treatment plant (Al-Arish), there is another treatment plant called (Al-Shaab), with an existing capacity of 15, 000m operated in two parts 1979 and 1989

A. Full Mechanical Wastewater treatment in Sana'a:

The treatment plant in Sana'a consists of the following steps (Fig. 1):

- Screen and Grit chamber: to separate large particles (grit and sand) from the wastewater. This is a necessary step to avoid passing the big solids into the pumps and further steps.
- Aeration tank: as a secondary treatment by mean of using surface aerators
- Final sedimentation tank to settle the sludge and to clarify the effluent
- The sludge is pumped into the drying beds for drying
- An upgrade is considered by additional drying beds, primary sedimentation tanks, digesters and odor control.

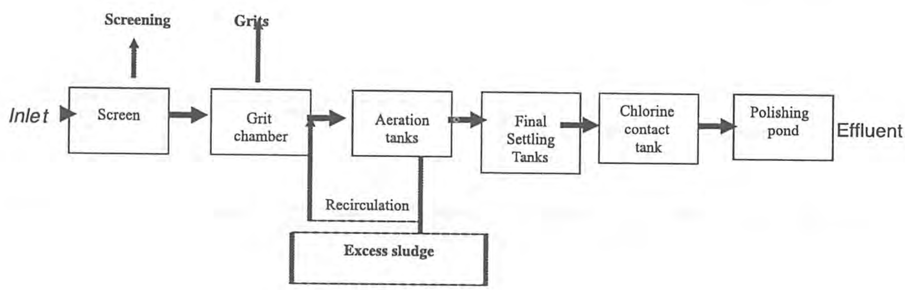

Fig.1 Flow chart of full mechanical wastewater treatment in Sana'a

In this full mechanical system, an extended aeration system is adopted. The operation has started since the beginning of 2000. The design values of the BOD load were 2500 kg BOD/day (Q=50,000; BOD_5=500) while the existing load (Jan 2005) has exceeded this value. It is now 43200 kg BOD/day (Q=36,000; BOD_5= 1200) indicating that it is already organically overloaded, although hydraulically still under loaded. This caused high a production of sludge which made the drying beds over loaded as well. Moreover, due to the lack of the drying beds, the sludge accumulated in the aeration tank and has become 12 kg MLSS/m^3 instead of 5 kg MLSS/m^3 as the highest allowable values in such systems, which caused bulking sludge.

The characteristic of the activated sludge has become less stabilized. This has forced the operator to use a polymer to coagulate the sludge to become easily dried. Moreover, the sludge still needs 20 days to become dry to achieve a higher dissolved solid content to be practical to desludge mechanically. The total numbers of drying beds are 20.

A consideration from the SWSLC is to upgrade the treatment plant by implementing an additional 44 drying beds, primary sedimentation, odor control, post thickeners and digesters, followed a gas holder and co-generation plant . The 44 additional drying beds are under implementation while the other measures are just in the final design stage.

B. Partial mechanical wastewater treatment in Hajjah:

The treatment plant in Hajjah consists of the following steps (Fig. 2):

- Screen and Grit chamber: to separate large particles (grit and sand) from the wastewater. This is a necessary step to avoid blockage in the Imhoff tank.
- Imhoff tank: acts as a primary settling tank at a retention time of 2 hrs, in addition to a sludge digestion by means of anaerobic bacteria at a retention time of 60 days.
- Two- stage trickling filters: BOD load is degraded by means of aerobic bacteria grown at the surface of the plastic media in the first stage and on the basalt stone in the second stage. The BOD concentration at the outlet is expected to be 20 mg/l. A part of the clarified liquid is recycled to the trickling filter to improve sloughing and therefore prevent clogging of the filter. The settled sludge in the final sedimentation tank is pumped to the Imhoff tank for further biological degradation of the sludge.

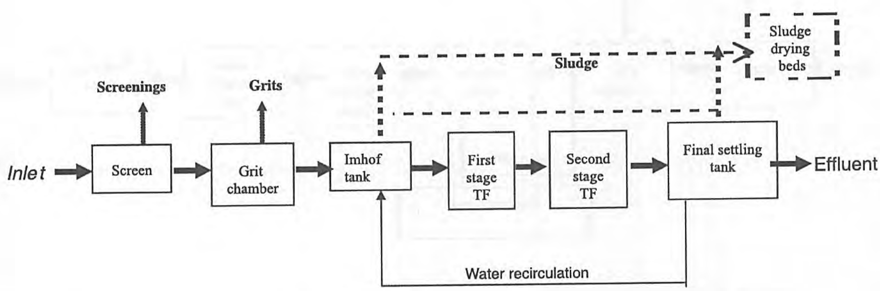

Fig.2: Flow chart of treatment plant processes in Hajjah

In this partial mechanical treatment, although the existing BOD_5 of 1200-1300 mg/L is also higher than the design BOD_5 value of 843 mg/L, the existing organic load is 1270 Kg BOD/d, which is still represents 62% of the design load. The overall performance of the treatment system is very good (97%) in terms of BOD removal. However, due to the absence of the final sedimentation tank after the first stage TF, no improvement was achieved by the second stage TF, which suggests that the second stage TF is clogged with the continuous receiving of the effluent with the sludge from the first stage TF. Moreover, IT is a proper choice as an anaerobic pretreatment system in hot, arid and semi- arid climates in developing countries as it does not need high skills and power input. Furthermore, the use of TF is still also proper, as it does need mechanical equipment for oxygen input.

C. Natural Wastewater treatment in Aden:

The treatment plant in Aden consists of the following steps (Fig. 3):

- Screen: to separate large solids. Grit chamber is not applied because no mechanical parts exist and therefore, there is no negative effect of accumulation of sand in the anaerobic pond.

- Anaerobic pond: acts as a primary settling tank and anaerobic treatment at the same time with a retention time not higher than 5 days, in addition to a sludge digestion by means of anaerobic bacteria at a retention time of at least 2 months. The settled sludge in the anaerobic ponds is transferred to the drying beds.
- Facultative ponds with an 18 day retention time. BOD load is degraded by means of aerobic bacteria grown at the upper part of facultative ponds BOD concentration at the outlet is expected to be further treated in the maturation ponds which will also decrease the pathogen bacterial and helminth eggs

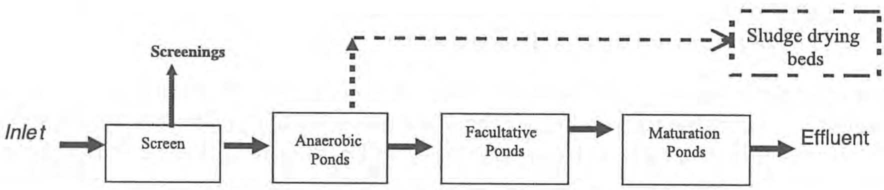

Fig.3: Flow chart of the treatment plant in Aden

In this natural treatment, although the ponds are easy to construct and operate, in the Al-Sha'ab plant, there is only one anaerobic pond while at least two anaerobic ponds should be available so that it can be alternatingly used to evacuate the sludge before the capacity of the anaerobic pond becomes low due to sludge accumulation. The design BOD value for the anaerobic ponds was 320mg/L, while the actual value is 624mg/L. with a design hydraulic load of 30,000m^3/d. This makes it already at full organic capacity with half the actual hydraulic capacity of 15,000 m^3/d. For the facultative ponds, it has become overloaded with 458 kg BOD/ha.d with a retention time of about 9 days which is not enough for the treatment (Veenstra et al., 1995). The same goes for the Al-Areesh plant where the design BOD value as 312mg/L with an actual BOD of 680mg/l. With the existing hydraulic loading of 22,000 m^3/d, the facultative ponds are just about at their organic capacity which means that the design hydraulic capacity of 70,000 is not possible to be achieved in the future. Thus, both the WSPs in Aden are having problems due to high BOD loading and inadequate operation, as also indicated by a purple color (Veenstra et al., 1995).

Therefore, although WSP is a proper choice as anaerobic followed by facultative ponds are a better choice for hot, arid and semi- arid climates in the development countries which does not need high skills and power input, they still have to be designed and operated well.

Sludge problem in Yemen

Sludge is produced during the treatment of wastewater. It is estimated as 60-80% (Metcalf and Eddy, 2003) of BOD concentration in the case of the mechanical treatment. In the activated sludge system in Sana'a, the sludge is produced on a daily basis and treated by thickeners and then dried in drying beds. At the moment, mechanical systems in Yemen are facing difficulty in drying the solids which require lager areas for the drying beds. This difficulty exists although the mechanical treatment systems were intentionally chosen to save land, which is normally needed and considered as the main disadvantages of the natural systems.Sludge treatment after the mechanical

WWTP systems should be done in such away that it could be easily taken away from the site and used as fertilizer. In the mean time, the sludge treatment is limited to thickening up to a solid concentration of 5%, then the sludge is pumped into a drying beds to be dried up to 40% dry solids. The period designed for the sludge to stay in the drying beds is 10 days. In the actual case in Sana'a, the sludge could not get dried within this period. Therefore, the Sana'a Local Corporation as operator has introduced a polymer to help in the coagulation- flocculation of the solids and therefore faster separation and drying. After this additional step, the sludge still need 20 days to get dried to 80% to make it easy for desludging by using a machine (small shovel). Moreover, in this situation, the sludge is still not suitable to be used as a fertilizer although it is purely domestic. It is still not treated to remove the pathogenic bacteria which could affect the crops, soil and farmers.

An important measure to be considered for the Sana'a WWTP through upgrading the project is to construct anaerobic digesters and the utilization of biogas by converting the biogas into electrical and thermal energy to be utilized in the site. Nevertheless, this process technology is complicated and also needs high skilled labor, while on the other hand, those cities which use WSP technology, the sludge is considered as more suitable to be used as a fertilizer as it stayed for 6 months to one year drying in the anaerobic ponds after stopping the inflow to the anaerobic ponds.

Reuse of treated wastewater:

Wastewater effluent is used at the moment in restricted irrigation although it still is not treated enough to be ready for irrigation (Table 1). The application of this wastewater is applied in different situations as follows:

- Wastewater effluent after treatment still does not reach hygienic specifications that can be used for irrigation.
- The effluent is mixed with raw wastewater which is by-passed from time to time, especially when incoming wastewater contains mineral oil (in the case of the Sana'a WWTP).
- Some people are opening the manholes before they enter the WWTP and clog the effluent pipe to stop the water to flow to the treatment plant and then pump this wastewater to the fields for irrigation.

These unusual situations are creating a health hazard, although there are a lot of efforts to stop these actions legally. Yemen still is in need for more efforts in this respect. Nevertheless, there is a plan to have several projects concerning the reuse of treated wastewater in a safe way and in line with international standards.

Conclusions and Recommendations

As Yemen is a developing country suffering from water scarcity (which on one hand increases the wastewater concentration and on the other hand needs to reuse the wastewater effluent after treatment as a non-conventional source), the treatment system applied should be robust in terms of efficiency and cost. Therefore, it looks as if the WSP in combination with an anaerobic system should be combined and applied.

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Factors Affecting Reuse of Wastewater Effluents in Irrigation: Towards Sustainable Applications.

Salah Al-Mogrin

FACTORS AFFECTING REUSE OF WASTEWATER EFFLUENTS IN IRRIGATION: TOWARDS SUSTAINABLE APPLICATIONS.

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ABSTRACT

Long-term effluent irrigation with low quality secondary effluents may lead to the accumu-lation of toxic substances in soil that reach hazardous levels and which eventual-ly may lead to the pollution of groundwater sources. Tertiary treatment is a safe-guard against the chronic effects of pro-longed reuse of effluents in irrigation that will accumu-late along with the time and should be consi-dered in the planning stage of new projects and/ or to be engineered to cope with changes in exist-ing facilities to satisfy stan-dards and criteria set for reuse. To ensure a sustainable wastewater effluent reuse programs, factors affecting this practice must be studied such as setting standards, selection of irrigation method, planning of the reuse projects, and management of reuse projects. In this paper, factors affecting wastewater reuse programs are presented and analysed. The objective is to attempt to introduce long-term sustainable wastewater reuse programs with negligible health and environmental impacts.

1. Introduction

Domestic wastewater is a complex mixture of minerals and organic matter. Most of the organic matter found is either suspended or is settable solids. Approximately 60 - 80% of the organic matter is biodegradable (Hammer, 1986). Despite the bio-availability of organic compounds such as detergents, pesticides, long-chained hydro-car-bons and cellulose as bacterial substrate, they are con-sidered non-biodegradable because of the time limit needed for their assimi-lation by bacteria. The inorganic chemicals include metals, ammonia and ammonium salts, nitrates, nitrites, sul-phur and phosphorus containing compounds, chlorides and bicar-bonates of sodium, potassium etc. (Viessman and Hammer, 1985).

Wastewater may also contain bacteria, viruses, helminths, and protozoa of which many spe-cies are pathogenic to humans and animals and wastewater is considered an excellent medium for their dissemination. Pathogenic micro-organisms that are discharged by infected persons pose a real threat to human life if they are com-municable. Because of the characteristics of wastewater, its use in irrigation must be approached with caution and a full alertness to its limitations.

Wastewater treatment: An evaluation perspective

Originally, domestic wastewater is 75% Gray water and 25% black water which arrive at the treatment plant.. These processes involve the separation of the solid fractions from the liquid phase to reduce, as far as poss-ible, the organic pollutants. The pur-pose of the biological treatment is to accel-erate natural purification under controlled condi-tions. The purified effluent can be then discharged to the environment and in many cases it is planned to con-vert the efflu-ent through proper treatment into a reusable resource for different objec-tives.

Wastewater treatment methods were developed through continuous research and develop-ment in response to concerns for public health and environmental damage caused by the discharge of wastewater onto land and into the surface water. Therefore, the main objectives of wastewater treatment include destruc-tion of the disease cycle, prevention of the environ-mental pol-lution of water sources such as surface- water and ground water, and more recently to convert wastewater effluents to reusable resources in planned recycling pro-grams. A wide variety of processes is engineered specially for achieving the required pollutant reduction and are availa-ble for application. Raw sewage cannot be reused directly and conventional wastewater treatment processes will hardly produce satisfactory effluents, which are suit-able for irrigation reuse. Many com-pounds are not removed by secondary wastewater treat-ment, which range from simple inor-ganic compounds to large and complex synthetics (Impelluso and Pfafflin, 1992; Pescod, 1994).

2. Reuse of wastewater effluents

Potable and non-potable reuse

The reuse of wastewater has been practised in many parts of the world in two general categories: potable and non-potable practices.

Indirect potable reuse is an old and common practice in many countries where cities upstream discharge their effluents treated partially or completely to rivers and cities down-stream abstract water as their drinking water source. The river self-purification capac-ity has an important role in the further treatment of wastewater, together with the dilution effect. Indirect potable-reuse is practised also via groundwater recharge. Direct potable reuse is less practised due to many reasons such as the high cost and com-ple-xity of treat-ment required rendering wastewater effluents good for drink-ing (pipe-to-pipe) (Ham-mer, 1986; Haarhoff and van der Merwe, 1995).

Non-potable reuse can be divided into four areas: a) municipal reuse with street-washing, wastewater-network flushing, and fire fighting, b) agricultural reuse, c) house-hold reuse such as toilet flush-in-g, lawn irri-ga-ti-on etc., and d) indus-trial reuse as boiler-feed coo-ling (Metcalf and Eddy, 1990).

Irrigation reuse

Agricultural reuse is by far the widest use spectrum estimated to be 70% of total reuse practices, which include crop and pasture irri-gation, horticul-ture, silviculture, and aquaculture (Tchobanoglous and Angelakis, 1995). Agricultural water demand accounts for 67% of global fresh water reuse, while 25% is de-manded for industrial reuse (van Leeuwen *et al.*, 1995; Asano and Levine, 1995). It is estimated that irrigation water demand accounts for an average of 70% of the total water demand (Arar, 1991). The use of water in crop production may be also higher in some areas so, for example, 74% of the total nation-al water demand is used for irriga-tion in Austra-lia (Anderson, 1995). According to Arar (1991), agriculture is the largest potential user since it uses a large quantity over vast areas and has been proved to be feasible in many parts of the world. Treated domestic wastewater has long been used for agricultural irrigation in many parts of the world with varying degrees of environmental safeguards and protection of public health (Burau *et al.*, 1987; Burgess, 1991). In recent years, it has been utilised for the production of cash crops, particularly in the arid world where water resources are limited (Burgess, 1991). There is an increasing interest towards the reuse of treated wastewater for crop irrigation, especially in the Near East which is the most exten-sive arid zone (Arar, 1991; Mara, 1994).

Since 1960, the total world acreage that is irrigated has almost doubled, and during the 1980's the development of irrigation was allocated high priorities in all continents (Asano and Levine, 1995). According to the Rio Earth Summit, it is projected that sustainable crop production from irri-gated agriculture must increase 3 to 4% annually to meet the ever growing food demand. This is caused by the steady increase of the overall world population growth especially in developing countries. Therefore, to meet the required level of agricultural production, maxi-mum reuse of wastewater is required in arid and semi-arid regions because water shor-tage is a limiting factor for agricultural activities and some branches of indus-try. Water shortage is an especiallyserious problem in Saudi-Arabia due to rapid popu-la-tion growth, industrialisation, coupled with the introduction of inten-sive agri-cul-ture techni-ques causing an increas-ingly heavy demand on water resources. Therefore, the reuse of treated effluents has become an attractive option in increasing water sources (Chughtai and Khrushid, 1978, Niedrum *et al.*, 1991).

Assum-ing a continuation of current population growth in arid regions, the deficit in the water budget will be severe. At the national level, increase of water use is attributed to: piped water supply to homes and industries, increasing use of labour-saving machines, the merging of smaller farms to larger incorporated farms (which use inten-sive heavy machinery) and water abstraction, lack of aware-ness, poor water management, and absence of water-saving plans (Chughtai and Khrushid, 1978).

Reuse of treated wastewater in irrigation represents a national option in affected countries for a supplemen-tary water supply. It represents an ecologically accept-able sol-ution. The soil repre-sents the most suitable recipient, not only because of the possi-bility of its enrichment, but also because of its smallest undesir-able effects on the envi-ron-ment.

Water scarcity has also led to a growing interest in the rational use and conservation of water which is essential. Therefore, the reuse of treated wastewater on a large scale has received increasing atten-tion. Large scale wastewater reuse schemes now exist in many countries, in industrialised as well as in develop-ing countries, for example, the USA (State of Califor-nia, and other states such as Nevada, Arizona and Texas due to drought), Australia's dry region and many countries in Middle East including Saudi-Arabia and Southern European countries such as Spain and France due to signs of desertification (Bontoux and Coutrois, 1995).

In many cases water sources are available but not always near the point of where it is required, and pumping and/or conveying cost may dictate the use of other supplies, especial-ly if marginal water quality is suffice (Impelluso and Pfafflin, 1992). Under these conditions, the pres-ence of treated wastewater (effluent availability as supplementary source) near the affected farming area provides a solution to mitigate irriga-tion-water scarc-ity (Pescod, 1994).

In some countries such as the Ara-bian Penin-sula, decision makers are involved in the optimum use of the available supply as well as finding a supple-mentary supply through treated wastewater reuse or desalination. However, reclamation of wastewater is more feas-ible than desalination which is a vastly more complex technology and much more expensive to operate and main-tain (Willard and Bustamante, 1972).

3. Factors affecting irrigation reuse projects

In the arid world, the success of irrigation reuse projects depends on six factors:

1) Quality vs. standards

2) Irrigation method

3) Planning of the reuse project

4) Management of reuse projects

5) Costs and benefits of reuse practices

6) Limitations of wastewater reuse

It is important to shed some light on these factors because of their significant role in the suc-cess of reuse projects.

3.1 Quality of the wastewater and standards:

Effluent quality is an important consideration in its suitability for reuse. Environmental and other regulating authorities set stan-dards on wastewater effluents to judge its quality for the protection of human health and the environment. In many cases, treated wastewater may have better quality for many uses than surface or even groundwater, but it is the psychological barrier that separates these waters, by their his-tory not their qual-ity. For example in Bahrain Island, an Ara-bian Gulf State, groundwater used in irriga-tion contains 2500-6000 (mg/l) total dissolved solids (TDS), while the range of TDS is 1000-3000 (mg/l) for treated wastewater (Akhtar and Madany 1991).

Wastewater quality control is a complex and sensitive issue in systems that provide resources under strict reuse specifications. Criteria for reuse have been issued by vari-ous coun-tries and International Agencies such as the World Health Organisation (WHO), and the Food and Agricul-ture Organisation of the UN (FAO). Standards usually set for irrigation with treated wastewater are based on effluent quality which can deter-mine the use on restricted or unrestricted irrigation. The restrictions imposed usually refer to the type of crop, irrigation method, harvest, and distance from houses of inhabited areas, and from the water supply.

Unrestricted irrigation requires a high quality effluent and, therefore, can be used for irrigation on all crops and any soil without any adverse effects on the soil, crop, human or animal. Restricted reuse, as the name implies, restricts the use only to crops that are consumed cooked and also include other limitations on the crop type and irrigation method.

There are different standards in different nations, as well as international agency guidelines issued for wastewater reuse in irrigation. The effluent quality is usually characterised by:

1. Microbial parameters: virus, bacteria, parasite, including protozoa and higher animals.
2. Agronomical aspect: salinity, sodium absorption ratio (SAR), bicarbonate (HCO_3), nitro-gen (N), chlorides (Cl), boron (B), heavy metals (HMs), and pH.
3. Organic matter: suspended solids (SS), biochemical oxygen demand (BOD), chemical oxygen demand (COD), and total organic carbon (TOC).

Physico-chemical quality of effluents should also comply with the guidelines of the FAO for the qual-ity of irrigation water which is based in the absence of industrial wastewater on electrical conductivity (EC), B, and SAR (Mara 1994). The FAO classifies irrigation water into three groups based on salinity and toxicity characteristics. This classi-fication will enable the evaluation of treated wastewater for its suit-ability for irrigation (Pescod, 1994). Some standards are unnecessarily restricted and they should be relaxed when-ever feasible according to case-by-case evaluation (Chughtai and Khrushid, 1978). How-ever, direct reuse of wastewater for irrigation regardless of its quality must not be per-mitted.

It is diffi-cult to issue global regulations and standards which are suitable for the reuse of effluents in irrigation for all regions. Guidelines and standards issued in different countries depend on regional condi-tions, such as cli-mate, soil, prime crop, other plant types, technology avail-able, etc. For example, Cali-fornia Standards are suitable for European countries (industrialised) while WHO guidelines are suitable for developing countries (Smith and Walker, 1992). Interna-tional agencies continue their efforts to establish guidelines to be fol-lowed in countries where wastewater recycling is practised in the absence of national stan-dards.

The most widely used guidelines available and adopted in reuse projects are:

1. Engelberg report 1985, as product of the Meeting of the World Bank and WHO in Engelberg, Switzer-land.

2. Health Guidelines for the Use of Wastewater on Agriculture and Aquaculture, WHO 1989,

3. Wastewater Directives EU Directive (91/271/EEC),

4. California Stan-dards 1987 (most restrictive) and US EPA 1992.

Recently, health-related chemi-cal guidelines by WHO for reclaimed wastewater and sewage sludge application on agriculture was published (Chang *et al.*, 1995). As an example of restricted irrigation, Kleene *et al.* (1993) reported that the State of Nevada per-mits the use of surface sprink-lers of irriga-tion using effluent provided thereis buffer zone of 122 meters at least between inhabited resi-dences and sites. Irrigation with treated wastewater, especially if mechanised, is recom-mended to be done at night time to minimise human expo-sure to bioaerosols and to avoid the high evap-oration rates in the daytime. Fur-ther-more, treated wastewater is not allowed to be used for irriga-tion in places with uncon-trolled access such as parks and play-grounds (Shannon *et al.*, 1986).

Regulations of the California State Department of Health strictly regulates the use of secondary effluents to unpro-cessed food crops (i.e., not to be consumed raw) and make the reuse of eaten-raw-veg-etables irriga-tion econ-omically invalid (Burau *et al.*, 1987). In the UK, taken as a European example, irrigation with treated wastewater is restricted to crops that are not for direct human con-sumption, e.g. corn, wheat, sugar beet, and sun-flowers (Mara, 1994).

Tselentis and Alexopoulos (1995) state that the trend towards more stringent standards is expected mainly to protect the envi-ron-ment and public health, and to meet the growing demand for higher environmental standards by the public. For example in the King-dom of Saudi Arabia, wastewater requires tertiary disinfec-tion for the unre-stricted use of irriga-tion. How-ever, many countries have less strin-gent stan-dards to avoid unnec-ess-ary restric-tions which have an impact on the economical success of the reuse projects. They claim that it could be practised without any detri-mental effects if proper pre-cau-tions are taken. Accord-ing to Mara (1994), up to 10 egg /l of nematode eggs is yet considered safe for restricted irrigation (passing the Engelberg report which limits the value to one egg/l).

3.2 Irrigation methods:

Mechanised irrigation is advantageous in the production of crops with a high com-mer-cial value (Arar 1987). Pumped and mechanised irrigation could reduce human con-tact with water to some extent. According to Pescod (1994), flood irrigation is the cheapest method but has the greatest exposure risks. He adds that border irrigation should not be used for vegetable irrigation and sprink-lers should not be used if the water quality is not good, except for fodder or pastures.

Subsurface or localised irrigation can give the greatest degree of health protection as well as using water efficiently but it is relatively expensive and a high degree of water quality is required to avoid clogging of the orifices. When drip irrigation is used as subsurface, a lower probability of crop contamina-tion is expected especially if drippers are covered by a plastic sheet (Avnimelech, 1993). Sur-face irrigation methods can utilise low quality effluent, whereas mechanised irrigation sys-tems require a higher quality, especially drip irriga-tion systems, to avoid clog-ging (Arar 1989).

3.3 Planning for reuse projects

Many countries embark on water reuse projects without careful planning. The reason may be the urgency caused by drought or the lack of experience in many factors associated with reuse. Regardless of the reason, significantly higher costs per unit water delivered than were esti-mated or disappointing water savings are usually encountered (Mills and Asano, 1995).

For example, a California Survey indicated an increase of about 25% in treated wastewater reuse from 1987 to 1995, but data showed that despite the increase only 65% of the planned yield was obtained. The actual demand exceeded the projected quantities in the planning stage (Deis *et al.*, 1986). According to Donovan and Woodward (1979), case studies are not only useful to model successful projects, but also to focus on problem areas. Accord-ing to their view the guideline for planned reuse should follow:

1. Preliminary investigation of local markets
2. Screening of potential markets
3. Detailed evaluation of markets according to collected data
4. Initial steps towards implementation

Asano (1991) reported that reuse planning involves three stages: conceptual planning, a feasi-bil-ity investigation, and facilities planning. In general, the intended water reuse application dictates both the wastewater treatment required for the quality needed and the method of distribution and application (Asano and Levine, 1995). Therefore, the entire proposed treat-ment pro-cess should be evaluated at the out-set in the planning stage (Tchobanoglous and Angelakis, 1995).

Reuse planning is important to conduct as early as possible and could be done either for the reuse of exist-ing facilities (upgrading, guideline review, review of irri-ga-tion method, appli-ca-tion of fertilisers to crops, etc.) or planning for totally new wastewater treatment plants (more suc-cessful). Factors that should be considered in planning may include: hydro geological details, hydrological data, topography study, soil type, and climatic conditions.

The early planning phase should include the evaluation of the role of wastewater reuse in the water resources development as a whole. Analysis of different possibilities of wastewater reuse in irrigation, and definition of the main problems and objectives should be conducted at this stage. Considerations to locally-available resources such as tech-nol-ogy, legal, financial, skills, etc. should be done as early as possible.

The utilisation of a geographical information system (GIS) is also important in the planning of wastewater reuse. GIS will help to specify the optimum location of wastewater reclamation and reuse situ-ations to aid deci-sion makers for the ranking of specific scen-arios con-cerning critical resource preservation and build to sustainable sound criteria (Despotakis, 1995).

3.4 Management of reuse projects:

Sewage irriga-tion involves a com-plex interaction between many parameters from different overlapping fields. Therefore, it is diffi-cult to assure its long-term operation with minimum impact. A clear policy should be established, and necessary legislation must be enacted to promote and control this activity.

The divi-sion of the respon-si-bil-ities of wastewater pro-grams among several gov-ern-ment institu-tions may lead to inad-equate co-ordination for exploi-ta-tion of all water resources (Arar, 1991). Elab-or-ation and imple-menta-tion of a nation-al reuse policy is necessary (Brissaud and Bahri, 1995). In developing coun-tries, attempts have often been successful technologically but the downfalls in the management of these reuse projects have been the main reasons hindering the accomplishment of the set goals. Problems encountered include:

1. Responsibilities which are divided between several centralised institutions leading to inad-equate co-ordination in this field and an inefficient exploitation of all resources
2. Inadequate pollution control and other environment-orientated monitoring programs
3. Absence of functional regulatory agencies
4. Absence of policies, standards and regulations to govern this issue
5. The adoption of European Standards, which are not adequate for the region (Arar 1987, 1991)

Arar (1987) also added that to achieve the aims set for agricultural promotion through reuse of treated wastewater, massive programmes must be mounted to reach each village in the affected region. Ser-vices must be established to formulate and guide these programmes. A higher level of services can be accom-plished through continuous training of public and private staff dealing with such projects, as well as vocational training of farmers and workers in the field together with research, development, and re-evaluation and modification programs. This is especially important for a country like Saudi Arabia where treated wastewater is the only feasible unconventional water source expected to play an important role in the, abolition of aridity in the country.

3.5 Cost and benefits of reuse of effluents in irrigation: Environmental and Monetary

i) Environmental costs and benefits

Numerous advantages of effluent reuse in irrigation have been cited in the literature to include its nutrient value and continuous availability as a source of water. Even in some countries without water shortage, treated wastewater is reused in agriculture irrigation as a cheap and safe disposal method (Arar, 1989). Therefore, treated wastewater reuse in irriga-tion can be seen as a step in sustainable liquid dis-charge technology and strategy (Arar, 1986; Moore *et al.*, 1988; Belic and Belic, 1995). It has been estimated that the per capita/year production of nutrients are: 2.75 kg N, 3.2 kg P as P_2O_5 and 24.2 kg K as K_2O (Lawrence, 1970). Arar (1991) estimated 55 kg N, 10 kg P and 30 kg K in 1000 m^3 wastewater. For example, Saudi Arabia imports about 300,000 tons of nitrogen and 200,000 tons of phosphorus fertilisers annually (UN, 1991). The utilisation of 1.5M m^3 of wastewater/day will yield about 33% and 5% of the country's requirements of nitrogen and phosphorus, respectively. At least 90% of OM and nutrients will be utilised on the top

30 cm of soil (Monnett *et al.*, 1996). OM and nutrients of the effluent will enrich poor soils (e.g. sandy soils) and enhance degradation of complex compounds in soil. It is reported that 0.5-1.0 kg of OM is required annually /m² of land for equilibrium (Steel and McGhee, 1979).

In some countries, wastewater disposal regula-tions are becoming stricter and this trend is expected to render wastewater reuse an attract-ive economi-cal option (Impelluso and Pfafflin, 1992) because costly nutrient removal methods are not required for this practice (Hrudey and Smith, 1983; Smith and Walker, 1994). According to Asano (1991), farmers will have a reduction in fertiliser costs and still have good production, jobs will be created in commun-ities, and municipalities will benefit from selling the wastewater. In Baltimore, USA, excessive pump-ing from wells close to the har-bour caused salt intrusion and a big steel plant in that area asked to buy secondary efflu-ent from the City of Baltimore which received revenue for a prod-uct previously discarded, and the draw down of the wells was reduced (Impelluso and Pfafflin, 1992).

ii) Monetary Costs and Benefits

Reported costs of irrigational reuse projects have varied widely depending on the location and distance the from source. In Tunis, the acceptable price by farmers ranged from 0.014 - 0.04 US$ /m³. In Morocco, irrigation reuse was estimated to cost 22% less than the use of fresh water due to savings on fertilisers of 0.06-0.16 $/m³ of effluent reused (Mujeriego *et al.*, 1995). Table 1 summarises the cost estimates for irrigation reuse projects from different sources. In arid areas, costs have sometimes been outstripped by the cost of diminishing groundwater pump-ing (Haarhoff *et al.*, 1995). For example, for 1000 m³ of effluent reused in Saudi Arabia, there will be an equivalent amount of groundwater saved for the next generation to find. The unforeseen benefit is diminished dependence on expensive desalination or importation of fresh water (1-6 US$ /m³) (Farooq and Al-Layla, 1987; George, 1987)

Table 1: Cost estimates for different kinds of irrigation projects.

Treatment Type	**Cost US $/m³**	**Reference**
Secondary + Chlori-na-tion	0.08-0.35	MOMRA, 1995
Secondary + Coagu-lation	0.56	Arar, 1991
Tertiary treatment, KSA	0.54-0.74	MOMRA, 1995
Tertiary treatment, USA	0.03-0.39	Asano, 1991
Reuse of drainage	0.07-0.26	Smith and Walker, 1994
Groundwater	0.45	Ukayli and Husain, 1988
Desalination	1-2.2	Ukayli and Husain, 1988
Drinking water	0.60	Appan & Rahman, 1998
Industrial water	1.0	Appan & Rahman, 1998

3.6 Limitations of reuse of effluents in irrigation

In this study, more attention was focused on the risks of reuse than on its benefits in order to ident-ify the prob-lems associated as the first step towards their avoidance, elimin-ation or mitiga-tion. Problems that might arise from the practice of reuse of treated wastewater can be identified as: public health impact, environmental problems, and agronomical effects (Wetzler and Roy, 1984; Hrudey and Smith, 1983; Arar, 1986 Kleene *et al.,* 1993)

A) Public health risks

Where raw or treated wastewater is reused, health risks arise from expo-sure to pathogens, including bacterial, helminths, protozoa and enteric viruses. From a public health perspective, the most critical group of pathogenic organisms are helminths and enteric viruses (EVs) due to the possibility of infection from low doses and the lack of rou-tine, cost effective methods of detection and quantification (WHO, 1984). In addition, the treat-ments that are effec-tive for bacterial removal may not be so effec-tive for other organ-isms (Asano and Levine, 1995). Public health risks are, therefore, a major concern related to wastewater reuse. Inad-equate reuse practice may endan-ger several contact groups, namely farmers and their families as well as other residents in the vicinity on one hand, and handlers, sellers, and consumers of the products on the other hand (Pescod, 1994). For example, the reuse of poor quality effluents has caused parasitic infections among Mexican farmers and their families (Siebe and Cifuentes, 1995; Hernandez Bico, 1998).

Possible transmission of diseases may occur not only through physical contact or consump-tion of con-taminated crops, but also through bioaerosols that enter the human mouth and lungs carrying viable pathogens (Avnimelech, 1993). The public health issue is the core of any standards or guidelines set for reuse projects. Next, restrictions of effluents reuse from this perspective are addressed.

In arid regions, sprinklers are used for their water-saving benefit but their use may cause the risk of infection from bioaerosols. The degree of hazard depends on many factors, such as microbial decay (time), mini-mum infective dosage, immunisation of the population, distance from the irrigation field, and the irrigation method. The meteorological factors affect the spreading of the bioaerosol (Bausum *et al.,* 1983; Moore *et al.,* 1988). When using sprinklers, 0,1 - 1% of the effluent pumped will be aerosolised which can be spread to a distance up to 750 m downwind Bausum *et al.* (1983) found that 66 - 78% of par-ticles were in the size range 1 - 5 μm in an irrigation site aerosol. Camann *et al.* (1988) concluded that spray irrigation with secondary wastewater effluent elevated the airborne levels of FC, FS, or Mycobacterium, and coliphages for about 200 m downwind.

There is an increase in aerosol transport of about 25 m with an increase of wind veloc-ity of 1 m/s (Carducci *et al.,* 1995). The State of Nevada per-mits the use of surface sprink-lers for irriga-tion using effluent provided there is a buffer zone of 122 m at least between inhabited sites (Kleene *et al.,* 1993). The mechanised irrigation with treated wastewater was recom-mended by Shannon *et al.,*(1986) to be done at night time to minimise the human exposure to bioaerosols. Fur-ther-more, they sug-gested con-trolled access to parks and play-grounds irri-gated with treated wastewater.

B) Possible environmental and agricultural impacts

Wastewater effluent may contain toxic chemical pollutants. The risk of contamination of groundwater through irriga-tion depends on the effluent quality, local conditions and the rate of application (Pescod, 1994).

Leaching and transport of contaminants is possible with the use of low quality effluents but is dependant on retardation factors and the nature of the contaminant involved. Degra-da-tion of the envi-ron-ment could be caused by organic mat-ter, odour or aes-thetic prob-lems due to anaerobisis in the soil (Arar, 1986). Long-term reuse of poor quality wastewater may cause the slow build up of toxic material, salinization of agricultural land and organic matter (Arar, 1986; Smith and Walker, 1994; Pescod, 1994) resulting in lower soil productivity and low crop quality or yield (Asano, 1991). However, this issue is not of major concern if effluents have only domestic origin (Vazquez-Montiel *et al.,* 1995).

Severe pollution impact is possible if industrial wastewater is dis-charged to publicly-owned wastewater treatment plants even after pre-treatment. For example, damage to crops occurred in China when trichloracetoaldehyde was present in treated wastewater that was used in irrigation (Arar, 1986). Preventing chemical pollutants from entering the sewage systems is the best solution to prevent this kind of event, but this is difficult to achieve, unless industrial areas are isolated and provided with separate wastewater treatment plants. Avery important issue regarding irrigation with recycled water is the toxicity of heavy metals due to their persistence.

C) Possible effects on soil

Possible long-term environmental effects of the reuse of treated wastewater might include the build up of toxic material and salinity in the soil (Arar, 1986; Pescod, 1994). Therefore, the fol-lowing para-meters of irrigation water are usually of concern regardless of its source (FAO, 1985).

i) Sodium absorption ratio (SAR): It is a measure of sodium hazard in effluents. High Na^+ con-centration will cause Na^+ to replace Ca^{++} and Mg^{++} in the clay min-erals of soil leading to a poor inter-nal drain-age struc-ture, and soil will form into hard and unmanageable clods when dry. After irrigation, such soils are only slowly permeable to air and water. They shrink and crack when dry again making it difficult for seeding (Law-rence,1970; Mara, 1994). SAR is recom-mended to be less than 12 which is achieved by low Na^+ con-tent and can be calculated from the relation:

$$SAR = \frac{Na^+}{(Ca^{2+} + Mg^{2+})^{1/2}} \quad (1)$$

ii) Salting: Soil salinization is a common problem in arid regions with the presence of high Cl^- ion content near to the coast. This will create a section gradi-ent which will produce an upward flow by which many soils become salinized. Absorption of an excessive amount of Na is detrimental to the physical status of soil and may be toxic to plants.

Sodium is unique among cations in its effect on soil. Na from irrigation water is absorbed or fixed in an exchangeable form on the surface of clay particles. After long continued use of irriga-tion water, the soluble sodium of irrigation water and the exchangeable sodium of the soil may reach a steady state. Exchangeable Sodium Percent (ESP) must not exceed 10 - 15% of total exchange-able cations (FAO, 1985) which can be obtained using a nomograph. The main sources of salts in wastewater are industrial, especially from food industries (e.g. meat process-ing) as well as water softeners (Avnimelech, 1993).

Salinity could be measured as electrical conductivity (EC, dS/m). Optimum plant yield is expected when EC, TDS, and Cl^- are less than 3 dS/m, 2000 mg/l, and 400 mg/l, respectively. When EC is less than 0.7 dS/m, full yield is expected (Lawrence, 1970; Peavy *et al.*, 1985) but when EC is more than 3.0 dS/m, change to more tolerant crops is advised (Arar, 1986).

Bicar-bonate and Ca ions can, under certain conditions, precipitate as $CaCO_3$ if irrigation water is applied to soil. As irrigation water is concentrated in the soil, Ca precipitates but Na remains in solution which results in an increase in SAR. The loss of Ca from irriga-tion water as a result of reaction with HCO_3, also increases the Na hazard (Lawrence, 1970). For salinity, the allowable concentrations of B, Li, and Na depend on soil permeabil-ity, irrigation practice, and plant sensitivity to these parameters. Salinity and alkalinity could be controlled by: water quality, irrigation practice, and/or drainage conditions (Arar, 1986).

D) Effects on plants

The quality of irrigation water has a direct effect on plant growth and, therefore, on crop yield and quality. The reuse of treated wastewater within recommended quality limits has given higher yields than fresh well water, raw wastewater and settled wastewater (Burau *et al.*, 1987; Mara, 1994). Treated wastewater was reported to be used in irrigation of different plants such as alfalfa, sweet corn, wheat, rice and cotton with an increase in yield due to nutrients and without any detrimental effects even with long-term application (Campbell *et al.*, 1983; Bielorai *et al.*, 1984; Burau *et al.*, 1987; Papadopoulos and Stylianou, 1988). In a study of crop quality, no significant differences were found between crops irrigated with treated wastewater and those irrigated with well water in physical features and shelf life (Burau *et al.*, 1987).

In contrast, irrigation with raw and diluted wastewater caused the yield to be lower than the control and an increased concentration of heavy metals (Cu, Zn, Cr, Mn) in rice and wheat (Chakrabarti and Chakrabarti, 1988; Chakrabarti,1995). The use of diluted wastewater decreased catalase activities in the roots and stems of wheat and caused an overall decrease in metabolic status of the plants (Chakrabarti and Chakrabarti, 1988), probably due to increased exposure to heavy metals. Also, the use of saline wastewater from leachate in sub-irrigation of red maple samplings caused photosyn-thesis rates to decline 62% compared to the control, as well as leaf discoloration and root anaerobosis (Shrive and McBride, 1995).

Therefore, concentration limits of heavy metals are included in different standards regulating reuse of effluents. Safe soil limits of some heavy metals are also imposed (Table 2).

Table 2: Limit values for concentration of heavy metals in soil in U.K.

Parameter	ppm
Cu	5 - 20
Mn	20 - 24
Zn	25 - 60
Fe	40 - 150
Co	10
Ni	1
Pb	13.5
Hg	0.30
As	14
Cd	1.5
B	2

(Adapted from Purves, 1977; Omran *et al.,* 1988)

4. Conclusions

Limited water resources and increased population are threatening problems in our region. Reuse of treated wastewater in irrigation, which consumes about 80% of the total water budgets, represent a rational strategic option to secure continuing water supplement. In our hot-arid zone, wastewater from domestic origin can be utilised better than in temperate climates even with a high BOD. The high temperature yields a higher biodegradation rate, which is useful in the treatment process and the ample sunshine, should be exploited in the inactivation of pathogens.

It seems that wastewater reuse practices in some GCC states are on anunsustainable path. Many reuse projects are embarked on without careful planning. As a result, higher cost per unit of water than that are estimated, or disappointing savings, will be (or have been) encountered. Wastewater reuse programs involve complex interaction between many parameters from different overlapping fields. Down falls in management of reuse, projects have been the main reasons hindering the accomplishment of their goals.

Wastewater reuse schemes are long-term practices, therefore, their sustainability should be considered for many generations to come. On the level of the legislator, a clear policy should be established containing a clear cut future vision. Legislation should be are then drawn to be enacted to promote this practice.

For sustainable reuse projects, key factors affecting wastewater reuse practices should be carefully studied by highly qualified teams. These factors include planning for the reuse program, management of the reuse practice, quality of the effluent to be used and standards or guidelines available, irrigation method and crop restriction, costs and benefits of reuse program, and limitations of this practice.

5. Recommendations

In order to plan for sustainable wastewater reuse practices in developing countries, the following are recommended:

1) Adoption of a statement of future visions that consider scenarios and analysis of their implications for sustainable reuse practice.

2) Planning should incorporate a total environment concept (i.e. comprehensive regional planning).

3) Policies should be crafted via legislation and updated to incorporate social, economical, health, and environmental aspects.

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Decentralized wastewater use for urban agriculture in peri urban areas: An imminent option for water scarce countries

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DECENTRALIZED WASTEWATER USE FOR URBAN AGRICULTURE IN PERI URBAN AREAS: AN IMMINENT OPTION FOR WATER SCARCE COUNTRIES

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ABSTRACT

Applied research conducted by the Inter-Islamic Network on Water Resources Development and Management (INWRDAM) over the last five years on decentralized wastewater treatment and use has focused on a holistic approach for the development of "state of the art" modular on-site greywater treatment and use units at the household level and implementing capacity building of the local peri-urban communities to enable them to practice sustainable Urban Agriculture (UA) by saving fresh water and safe guarding the environment. The outcome to date of this research resulted in the optimization of the modular units for greywater treatment and use by techniques of drip irrigation and selection of crops for home gardens. These findings were results of three projects; one entitled "Post Project Evaluation of Permaculture Techniques" and a second entitled "Greywater Treatment and Reuse in Home Gardens"; both projects were conducted in the town of Ein Al-Baida, Tafila Governorate in the southern part of Jordan and were funded by research grants from the International Development Research Centre, Ottawa, Canada (IDRC). A third project entitled "Community Involvement in Reuse of Greywater to Improve Agriculture Output" was financed by the Jordanian Ministry of Planning and International Cooperation of Jordan (MOPIC) and involved providing more than 800 households in 90 peri-urban sites throughout Jordan with greywater units and drip irrigation systems. INWRDAM also succeeded in setting up similar greywater activities in other Islamic countries such as Lebanon where the adoption of greywater units in a cluster of six towns in Lebanon is being implemented. The results of this research were well accepted by the local community and by the government of Jordan. More projects addressing greywater use are now being implemented in Jordan, Palestine and Lebanon with emphasis on conserving fresh water, improvement of sanitation and generating extra income for the poor in peri-urban areas and constituted sustainable urban agriculture practices. A recent evaluation of INWRDAM' greywater projects indicated that "INWRDAM has contributed to raising the profile of greywater use both in Jordan and in other parts of the world". The main aim of this paper is to concentrate on the methods and results of INWRDAM's greywater treatment and use experience in order to make it easily affordable to other countries.

1. Introduction

Fresh water is a finite; naturally renewable resource received by way of precipitation, but is significantly unevenly distributed in time and space. Falkenmark, Lundqvist and Widstrand in 1989 ranked countries according to their per capita annual water resources (1). For annual water resources of 1700 m^3 and above, shortage will be local and rare; for 1000 m^3 and below, it will hamper health, economic development, and well being; and for 500 m^3 and below, water availability will be a primary constraint to life (2). INWRDAM published in 1995 water scarcity data for 55 Muslim countries. This data shows that most of these countries have water availability less than 1000 m^3 and 10 countries out of the 55, including Jordan, have less than 500 m^3 per capita.

Centralized sewerage systems, the preferred choice of planners and decision makers, are inappropriately provided to individual communities and wastewater is transported from several scattered communities to centralized facilities. The high cost of conventional sewers is regarded as one of the major constraints to expanding wastewater services to small communities. A World Bank review of sewerage investment in eight capital cities in developing countries found that costs range between US$ 600-4000 per capita (1980 prices) with a total household annual cost of US$ 150-650 (3). Conventional sewerage systems cost more in small communities. Because of their size and layout, small communities do not enjoy the economies of building large systems. The low population density means that longer sewers are needed to serve each household. The per household cost in the Jordan Valley rural sanitation project was projected at US$ 2200, four times the average of all urban wastewater projects constructed in Jordan between 1996 and 1997 (4).

Conventional sewerage systems are designed as waste transportation systems in which water is used as transportation medium. Reliable water supply and consumption of 100 liters per capita per day (lpcd) are basic requirements for problem free operation of conventional sewerage systems. Conventional sewerage is not appropriate for small communities in the Middle East region where water supply is intermittent and only limited amounts of water are available. By transporting the wastewater away from the generating community, several reuse opportunities can be lost. Reuse opportunities are often located within the generating community for landscape or for agriculture. Recent research and development in the field of greywater and wastewater management suggests that centralized wastewater management is environmentally unsustainable (5).

2. Greywater use

A functional and sustainable wastewater management scheme begins at the household level and is largely dependant on the "software" or the human component. Only when perception of need and perhaps anticipation for a wastewater use system has been internalized at the neighborhood user level will planning and implementation be successfully executed (6). Local level support of a treatment and recovery scheme can, in turn, catalyze pro-active institutional and vertical support from the government. Once the software component has been integrated into the project development, the "hardware" or technological component can act to promote a comprehensive, integrated, and sustainable wastewater treatment and recovery strategy for the community- if it is well selected and "appropriate". Wastewater and greywater treatment technologies in the developing world must have one overriding criterion: the technology must be cost-effective and appropriate.

Greywater use represents the largest potential source of water saving in domestic uses. The use of domestic greywater for landscape irrigation makes a significant contribution towards the reduction of potable water use. In Arizona, for example, it is documented that an average household can generate about 135,000 to 180,000 liters of greywater per year (7). This illustrates the immense potential amounts of water that can be used, especially in arid regions like the Middle East and North Africa. Domestic greywater use offers an attractive option in arid and semi arid regions due to severe water scarcity, rainfall fluctuations, and the rise in water pollution. To ensure sustainable water management, it is crucial to move towards the goal of efficient and appropriate water use. Greywater use contributes to promoting the preservation of high-quality fresh water as well as reducing pollutants in the environment. Meeting different needs with the appropriate quality of water may prove to be economically beneficial and at the same time reduce the need for new water supplies at a higher marginal cost (8).

3. Health guidelines

Wastewater treatment for use must meet quality standards safe for human contact and consumption of irrigated crops. In most countries, guidelines and standards for greywater either do not exist or are being revised or expanded. The most frequent guidelines directing the use of greywater to a level considered safe to protect human health are those outlined in Engelberg Standards. These guidelines outline acceptable microbial pathogen levels for treated wastewater for use in restricted and non-restricted irrigation. Restricted irrigation refers to the irrigation of crops not directly consumed by humans (e.g., olive trees, fodder crops). For restricted irrigation, wastewater effluents must contain d" 1 viable intestinal nematode egg per liter. Unrestricted irrigation refers to the irrigation of vegetable crops eaten directly by humans, including those eaten raw, and also to the irrigation of sports fields, public parks, hotel lawns, and tourist areas. The criteria for unrestricted irrigation, contains the same helminth criteria as restricted irrigation, in addition to a restriction of no more than a geometric mean concentration of d" 1000 fecal coliforms per 100 ml/treated effluent. These guidelines have been introduced to directly protect the health of consumers who may eat uncooked crops such as vegetables and salads (9).

4. Greywater treatment processes

The daily quantity of greywater collected or recovered from a household in rural areas is usually small. The major difficulty presented for treatment of greywater is the large variation in its composition. For instance, laundry effluents contain high concentration of detergents and washed out dirt, this can double or even triple the organic content of the greywater. Cooking and frying oil and fat in the form of food remains on dishes and in cooking utensils result in the biggest source of organic pollution in greywater recovered from an average family house.

Treatment of greywater to a quality level suitable for irrigation of home garden crops not eaten raw can be achieved by a variety of methods, but low cost and low technology must be the main factors in selecting a treatment method. Anaerobic treatment systems, such as the upflow anaerobic sludge blanket or the confined space constructed wetland offer reasonable choices for greywater treatment. Anaerobic treatment processes are not affected by a wide variation in influent quality or shock loads as compared to aerobic processes.

The main pollution load from greywater is in the form of organic matter and pathogenic microorganisms. Greywater can contain about 10^5/100 ml of potentially pathogenic microorganisms. Stored greywater undergoes changes in quality, which include growth in numbers of microorganisms according to the limiting factors for each particular microorganism. Research has shown that counts of total coliforms increased from 10^0-10^5/100 ml to above 10^5/100 ml in stored greywater from various sources. Of greater concern is the potential infection route that greywater provides for viral infections. It is important that the nutrient resources (nitrogen and phosphorous) be conserved if the wastewater is destined for use in agriculture irrigation. Greywater use in irrigating home gardens in rural areas offers higher potential of success and public acceptance.

5. Characterization of greywater

The quality of greywater is directly related to the amount of water used in the house and is affected by certain habits of the occupants such as bathing, use of disposable or washable diapers and baby washing and if dishwashing is manual or done by a dishwashing machine. The flush toilet consumes much more water than the non-flush type commonly used by the rural poor and many of the Middle Eastern countries. Highly urbanized and high-income families use much more domestic water than do the rural poor. The kitchen is a major source for pollutants in greywater and cloths washing and laundry may come next. Some countries enforce regulations that prevent mixing greywater originating from kitchen sinks with greywater from other sources in the house. Use of soaps and body shampoos and excessive use of detergents and cleansing chemicals can in general significantly increase the pollution of greywater.

Greywater is relatively low in suspended solids and turbidity, indicating that the greater proportion of the contaminants is dissolved. The COD:BOD ratio may be as high as 4:1 (varies much higher than values reported for sewage), this is manly due to the use of detergents with low biodegradability. COD values could vary from 40 to 370 mg/l between sites, with similar variations arising at an individual site. Greywater is also deficient in macronutrients such as nitrogen and phosphorous (10).

6. INWRDAM experience in greywater treatment and use

The International Development Research Centre (IDRC), Ottawa, Canada provided INWRDAM during 2001-2003 with financial assistance to enable it to conduct an applied research project for greywater treatment and use in the irrigation of home gardens in Ein Al Beida village, Tafila Governorate in southern Jordan. The main objective of the research was to help the peri-urban poor in Jordan preserve precious fresh water, achieve food security, and generate income, while helping to protect the environment. To achieve this goal the following specific objectives were set for the project:

a) To increase greywater recovery and make it more convenient and safe to handle;

b) Minimize environmental impacts associated with greywater use and ascertain whether greywater treatment is necessary and cost-effective;

c) Improve gardening/permaculture practices;

d) Strengthen local capacity to safely and efficiently use greywater and enable women to be better managers of household resources;

e) Promote changes in policies to encourage greater greywater use in Jordan.

Baseline data was collected about domestic water consumption, greywater quality, and type of crops grown in the area and willingness of households to participate in the use of greywater. This research project targeted 25 low-income households, including a girl's high school and the main mosque as beneficiaries of the project. The average family size was 6.2 persons/household and domestic water consumption was on average about 120 lcpd, which is a little bit high for a rural area but the reason may be due to the use of fresh water in the irrigation of home gardens and in watering livestock. Table (1) shows typical greywater quantities from different sources in the house.

Table 1: Average daily water uses in rural areas in Jordan (11).

Sources	lpcd
Intake and cooking	10
Kitchen, hand dishwashing	15
Bath/shower	20
Laundry	20
Toilet, non flush	15
Miscellaneous and irrigation)	40
Total	**120**

The baseline data in Ein Al Baida, Tafila revealed the following:

1. The average BOD_5 of raw greywater ranged from 300 mg/l to 1200 mg/l due to low water consumption and because kitchen sinks were considered as a source of recovered greywater.

2. The detergent concentration in greywater, measured by the methylene blue active substances (MBAS) ranged from 10 mg/l to 300 mg/l.

3. Salinity of greywater was on average equal to 820 μS/cm, which nearly doubled from 450 μS/cm for domestic water supply.

4. The average background soils salinity; measured as sodium adsorption ratio (SAR) was about 2.

5. Main crops in home gardens were olive trees, most families preferred to raise chicken and goats.

6. Most households had no religious or cultural barriers against the use of treated wastewater and women showed the willingness and ability to learn new methods of irrigation and home gardening.

Female leaders in the village were identified and were trained to be trainers of other women and girls on subjects such as upstream pollution prevention in the house by wise use of detergents, good dishwashing practices and permaculture techniques. Local technicians were trained to carry out the operation and maintenance of the greywater treatment units.

6.1 Description of technology

On-site greywater treatment methods developed by INWRDAM were designed with the objective to achieve low cost and ease of construction and low operation and maintenance costs and to yield greywater of a quality suitable at least for restricted irrigation.

6.1.1 The 2-barrel system

Two plastic barrels constitute the treatment kit. The two barrels are lined up next to each other and interconnected with 50 mm PVC pipes. The volume of each barrel is 160 liter and has a large cover, which can be closed tightly. The first barrel or tank is fitted with pipes to allow grease, oil and solids separation and thus acts as a pretreatment or primary treatment chamber, where the solid matter in the influent greywater settles and the floating components, such as grease and soap foam float. When the cover is opened, the chamber can be emptied of both floating and settled material. The second barrel or tank acts as a storage tank for primarily treated greywater. As soon as this barrel is filled, a floating device switches on a small water pump which then delivers the water through the drip irrigation network. The two barrel kit was found suitable for small families such as pensioners and old couples with no kids.

6.1.2 The 4-barrel system

This system is an improvement of the two barrel kit. Two tanks each with 220 liter capacity and filled with gravel media that act as anaerobic filters are inserted between the pretreatment tank and final storage tank. The four barrels are lined up next to each other and interconnected with 50 mm PVC pipes.

Once solids and floating material settle in the first barrel, the relatively clear water enters into the bottom of the second barrel. Next the water from the top of the second barrel enters into the bottom of the third barrel. This water passes through the gravel lumps (2-3 cm size graded gravel) and from the top of the third barrel is taken into the fourth. Anaerobic treatment is accomplished in the two middle barrels. Anaerobic bacteria grows on the stone surface so that when the greywater passes through the stones, the bacteria works on breaking down components of the organic material found in the greywater. The last barrel acts as a storage tank for treated greywater. As soon as this barrel is filled, a floating device switches on a small water pump which then delivers the water through the drip irrigation network. For an average family home, 20-30 trees (olives, fruit etc) planted in the home garden can be irrigated.

With a resident time of 1 to 2 days in the 4-barrel treatment kit the influent greywater undergoes treatment levels equivalent to primary and secondary treatment and meets the World Health Organization's guidelines for *restricted* irrigation.

The greywater quality parameters are shown in Tables 2A, 2B, 2C. The parameters above show the degree of effectiveness of the treatment of greywater. The variation in greywater quality was substantial and was affected by the care of family members with respect to up stream prevention of pollution. The regular cleaning of the oil and grease separator barrel resulted in big improvements in treatment and reduction of coliform counts.

The greywater of these units were fit for irrigating olive trees, cactus and many fodder crops. Monitoring of the impact of greywater on soil and plants after two years of application revealed some increase in soils SAR, but it was below the level that could affect plant yield. All plant growth rates were improved due to regular complementary irrigation and there was no contamination of crops with fecal coliform.

INWRDAM developed during this project a special environmental friendly liquid dish washing detergent and bathing shampoo that contain potassium or magnesium ions instead of some sodium ions so that the long term impact of detergents is controlled. The long term impact of greywater application on soil and plants was also addressed and available data over two years monitoring indicate that no build up of harmful salinity and harmful chemicals is recorded.

The cost of the kit module that serves a family of 6 persons including a drip irrigations system for a 2000 m^2 garden area was in the range of US$ 230 and for the 2-barrel kit, US$ 370 for the 4-barrel module and US$ 500 for the confined trench which serves up to 12 persons including drip irrigation systems. A cost/benefit study indicates that the household income increases due to the irrigation with greywater and saving in the water bill, reduced septic tank disposal cost and improved crop yield is in the range of JD 10 to 30 per month. This means that cost of greywater units can be recovered on the average in less than three years. The life of the greywater units is estimated to be more than 10 years with minimal running cost.

7. INWRDAM's Phase II greywater project

INWRDAM conducted Phase I of this IDRC funded project from May 2001 to May 2003 in Tafila Governerate, southern Jordan. During phase I, INWRDAM installed 25 greywater units in low income households in the peri-urban community of Ein Al-Baida town in Tafila. Five different types of on-site greywater treatment units/modules were developed and tested over three years. Two out of the five modules were selected as potential units for further improvement. One module is known as the 4-barrel unit (see Figure 1), which consists of four recycled plastic barrels lined up in an arrangement to receive greywater and achieve physical and biological treatment. A small automatic electric pump is used to deliver treated greywater to a trickle irrigation system serving a small garden of trees. The second unit (see Figure 2), is known as the Confined Trench (CT) module. It consists of a stage for the removal of oil and grease as well as a dug trench of about 3.5 m^3 filled with gravel that serves as the treatment medium. The treated greywater is then pumped automatically through a trickle irrigation system. In addition to the units installed in Ein Al-Baida, INWRDAM has also installed over 750 greywater units of different types for the benefit of low income families across Jordan in 2002 through a project financed by the Jordanian Ministry of Planing and International Cooperation (MOPIC).

An additional 300 greywater systems will be constructed in Phase II for low income families in clusters of towns in selected peri-urban areas, so as to further improve technical aspects, remove social and institutional obstacles and build momentum, thus accelerating the adoption of the greywater system in Jordan and elsewhere in the region. A key component of the project will be to promote community participation in all stages of the project by adopting a participatory methodology. Phase II will involve cooperation between concerned agencies in wastewater use, social development, building codes and public health. The National Centre for Agriculture Research and Technology Transfer (NCARTT) will be involved in the environmental and agricultural research components of this project.

The goal of Phase II is to help the peri-urban poor in Jordan preserve precious fresh water, achieve food security, and generate income, while helping to protect the environment. The objective of Phase II is to evaluate previous greywater use projects in Jordan, validate existing approaches, address social and institutional obstacles to scaling up greywater use, and monitor and refine systems to ensure long-term sustainability. The implementing organizations of Phase II look forward to cooperation amongst all stakeholders to ensure a successful outcome of this applied research project.

Figure 1: Four-Barrel Greywater Treatment Unit

Figure 2: Confined Trench Greywater Treatment Unit

8. Conclusions and recommendations

The project resulted in many direct and indirect benefits to the community and the environment. Women in the community benefited most from this project through training workshops, dialogue and learning by implementing the acquired new skills to build a productive garden. The monthly domestic water consumption decreased by about 30% for all greywater users and the income of the poor increased on the average by US$50 to US$ 50 per month many beneficiaries no longer had to pay a large portion of their meager monthly income to regularly empty their septic tanks. Many families started to copy and imitate the practice of their neighbors with respect to greywater use.

The following recommendations can be made regarding the appropriateness of greywater use technologies:

a. The scheme or technology should be a felt priority in public or environmental health, and either centralized or de-centralized technologies should be considered.

b. The technology should be low-cost and require low energy input and mechanization, which reduces the risk of malfunction.

c. The technology should be simple to operate, be local labor intensive, maintained by the community and does not rely on expensive chemical inputs such as chlorine or ozone to meet quality guidelines.

d. The treatment should be capable of being incrementally upgraded as user demand or quality standards and treatment guidelines increase.

The results of this project convinced the Ministry of Planning and International Cooperation of the Government of Jordan that greywater use can preserve fresh domestic water, help improve agriculture productivity at household level and generate income. As a consequence of these tangible results the Ministry of Planning of Jordan funded INWRDAM in September 2002 to implement more than 700 greywater units in the rural areas of Jordan. This project was successfully completed on June 2003. IDRC has approved in February 2004 a grant worth CAD$725.000 for INWRDAM to further test and develop the greywater kits so that communities can be served with 300 of such kits over the coming 3 and half years in Jordan.

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Water Demand and Wastewater Management in Kuwait

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WATER DEMAND AND WASTEWATER MANAGEMENT IN KUWAIT

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ABSTRACT

Wastewater is being increasingly recognized as an important element in water management in Kuwait, where freshwater is limited and scarce. Currently, a huge amount of municipal wastewater in Kuwait receives conventional treatment up to tertiary levels. Limited quantities of treated effluent are used directly for landscaping and specific irrigation uses. A significant portion of generated effluent is, however, discharged and wasted into the Gulf. Public hesitation and health risk constraints limit the wide use of treated wastewater for directly producing human-consumable crops and other indirect uses such as the recharging of groundwater aquifers. This paper highlights the problem of increasing water demands in Kuwait and explores the potential of maximum utilization of treated wastewater meeting a significant portion of water demand in the country. It also outlines activities necessary to overcome the constraints that retard the wider reuse of treated wastewater.

Key words: Water demand, advanced treatment, wastewater reuse, guidelines.

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Introduction

Since the mid 1950s Kuwait has put effort and money to secure safe and renewable water resources that can be used for domestic, industrial and commercial purposes. Kuwait produced most of its potable water by multistage flashing (MSF) desalination plants. Water demand for agricultural and other uses is met by brackish groundwater. A portion of tertiary treated wastewater is used for fodder irrigation. However, there is increasing concern about the production cost of desalination water and depletion rate of the ground water. In this context, effective and efficient reclamation and reuse of wastewater is considered as an important alternative water source. It can provide a viable and reliable water supply for non-potable uses. Other alternatives measures for augmentation of water supply include, water conservation, efficient management, and development of a new water resources. Potential wastewater is about 60 to 70% of the daily freshwater consumption rate (Abdel Jawad and others, 1997). The average domestic water consumption in the year 2001 for a population of 2.2 million was 415 million m^3 (MEW, 2002).

In conventional wastewater treatment plants in Kuwait, the municipal wastewater effluent is usually treated to tertiary level. However, many of the pollutants found in wastewater are not completely removed or are partially removed by such conventional processes. The impact of these substances on environment and especially aquatic ecosystem are adverse in nature. For example the presence of phosphate and nitrate is often responsible for the depletion of dissolved oxygen in receiving water by stimulating algae and undesirable aquatic growth. Therefore, environmental controls have become more stringent in terms of allowable concentration of residual substances in the effluent of wastewater treatment plants.

Geographical and climatic conditions of Kuwait

Kuwait is located in the northwestern corner of the Arabian Gulf and is considered an extremely arid country. Kuwait with a population of 2.4 million (estimate 2004, MOP, 2003) has a total land area of 17818 km^2 including the land areas of the offshore islands. The climate is very adverse and harsh i.e. hot, dry, dusty, desert environment with average temperatures of 0°C in the winter and 49°C in the summer months. Rainfall is low and mainly occurs during the four winter months, from November to February. Average annual rainfall is about 110 mm per year whereas the mean annual evaporation rate is about 4000 mm (MOP, 2002).

Water Resources and Demand in Kuwait

Natural water resources in Kuwait are very limited and mainly consist of brackish groundwater. The non conventional water resources include seawater desalination, and reclaimed wastewater. Most of the fresh water supplies in Kuwait are produced through 5 multi-stage flash (MSF) desalination plants (MEW, 2002). Brackish water is produced from groundwater and utilized for blending with distilled water, for irrigation and landscaping. Treated wastewater is used in very limited quantities in fodder (animal feed) production and landscaping activities. Table 1 outlines the overall quantities and percent share of Kuwait's water supply resources for year 2001 (MEW 2002, and MPW 2002). Distilled and groundwater dominate and share more than 95%

of the available resources whereas renovated wastewater used for fodder production accounts for less than 5%.

Table 1: Overall quantities and percent share of Kuwait's water supply resources for year 2001. (Source MEW Statistical book, 2002 and MPW process yearly reports, 2002)

Water resources	Million m^3/year	% share
Desalination	386	69.2
Groundwater	145	26
Reused wastewater	26	4.7

Water demand

The demand on water is growing rapidly due to rapid population growth, urbanization and socioeconomic development. Kuwait's population has doubled in the past 30 years and is predicted to double in the next 30 years. Figure 1 shows the development of Kuwait's population and rate of total water production in the past 10 years (MEW 2002).

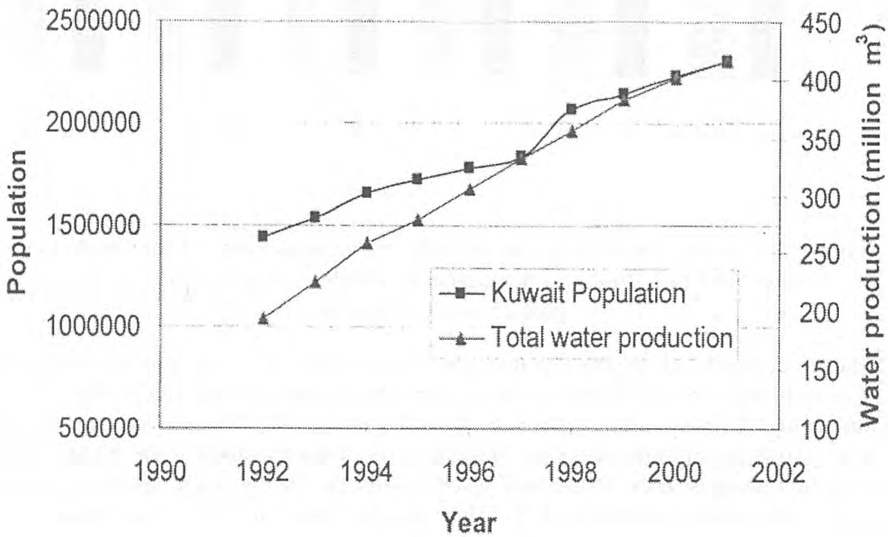

Figure 1: The development of Kuwait's population and rate of total water production in the past 10 years

Consumption of freshwater per capita was about 50 m³/year in 1977. Per-capita consumption has reached more than 180 m³ in 2003. This large quantitative growth in consumption per capita could not be sustained without corresponding growth in the distillation production capacity. Figure 2 shows the growth rates and annual changes in the production of desalinated water, and the gross and per capita freshwater consumption over the past 25 years in Kuwait (MEW 2003).

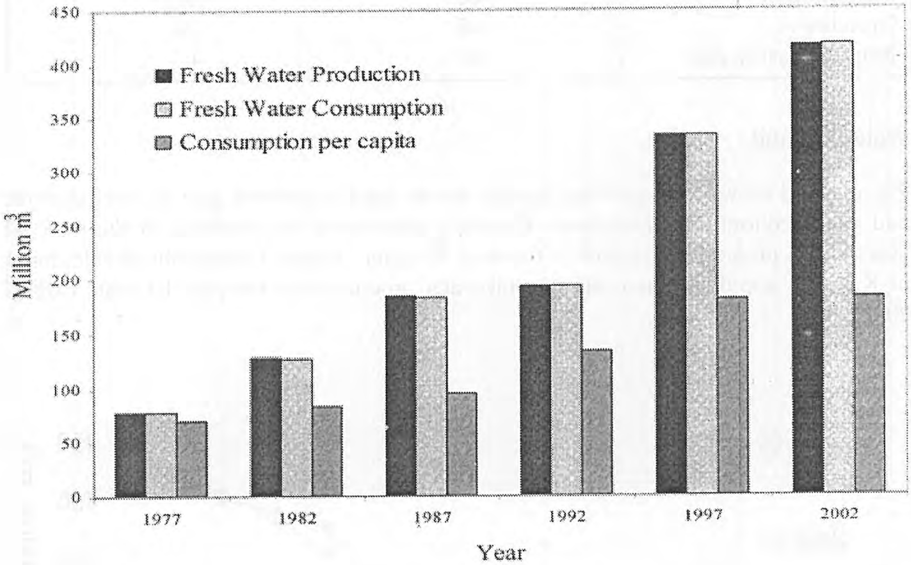

Figure 2: Growth rates and annual changes in the production of the desalinated water, and the gross and per capita freshwater consumption over the past 25 years in Kuwait

Domestic demands are the dominating share of total distilled water supplies in Kuwait. It was recorded to be 77 % of the total in year 1997 (Abdel-Magid, 1997). The sectoral withdrawals of water in Kuwait are shown in Figure 3. Kuwait is an arid country and has very little agricultural activities. However, irrigated agriculture is the biggest water user with a share of 60% of the total water resources. The domestic share of water is second in line and accounts for 37%. The industrial share of water is less than 2%.

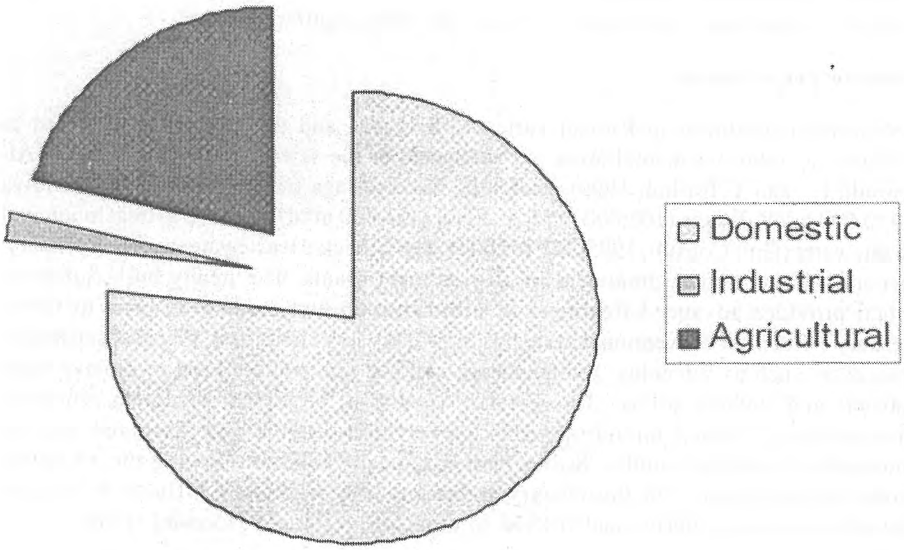

Figure 3: Sectoral withdrawals of water in Kuwait

Figure 4 presents projected 2025 sectoral and total water demands in Kuwait (The World Bank - Environment Depart 2001).

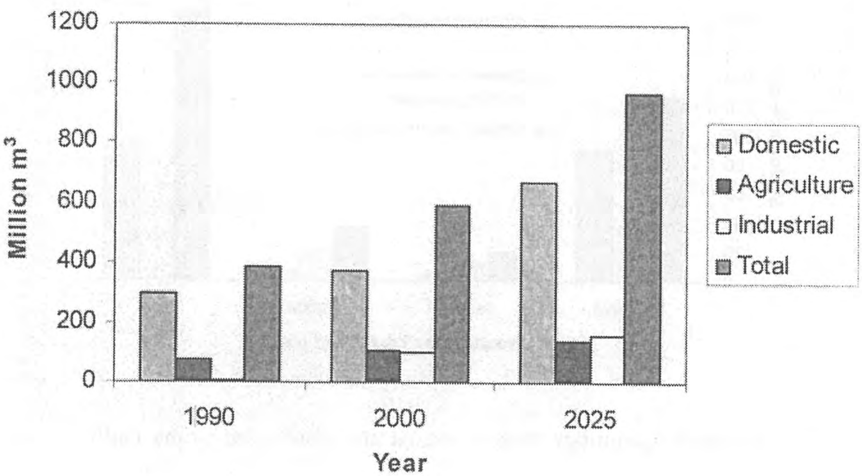

Figure 4: Projected sectoral and total water demand.

The increasing water demand particularly in non-domestic uses can be met by promoting utilization of treated wastewater especially in the agricultural sector. It will reduce groundwater withdrawal and save depleting aquifer reserves.

Wastewater in Kuwait

Wastewater produced in Kuwait varies with season and the amount is expected to increase as more residential areas are connected to the sewage collection system (Al-Awadi, E., and T. Rashid, 1999). Presently, three sewage treatment plants at Sulaibiya (350,000 m^3/d), Regga (102,000 m^3/d) and Jahra (54,000 m^3/d), are used to treat municipal wastewater (Gulf Consult, 1995; MPW 2002). The collected wastewater receives primary, secondary and tertiary treatment in all treatment plants. The newly built Sulaibiya plant provides advanced treatment of ultrafiltration and reverse osmosis to refine tertiary effluent of conventional systems. In primary level treatment, physical separation processes such as screening, sedimentation and grit removal are used to remove large objects and settable solids. In secondary treatment, biological treatment processes are utilized in which microorganisms convert non-settable and dissolved organic pollutants to settable solids. Sedimentation typically follows allowing these settable solids to settle out. In the tertiary treatment, the secondary effluent is usually disinfected using chlorine and filtered to eliminate residual suspended solids.

The total daily inflow into these treatment plants amount to about 185 million m^3/year or 44% of the total annual potable water use in the country. Part of the tertiary effluent is diverted for irrigation, landscaping and gardening use whereas a major portion of the treated effluent is discharged to the Gulf. Figure 5 shows quantities of treatment plant influent and effluents, reused and discharged to the Gulf (MPW, 2002). The sulaibiya quantities in Figure 5 are estimated from its initial data at the operation which started in August 2004 (personal contact BOT, 2005).

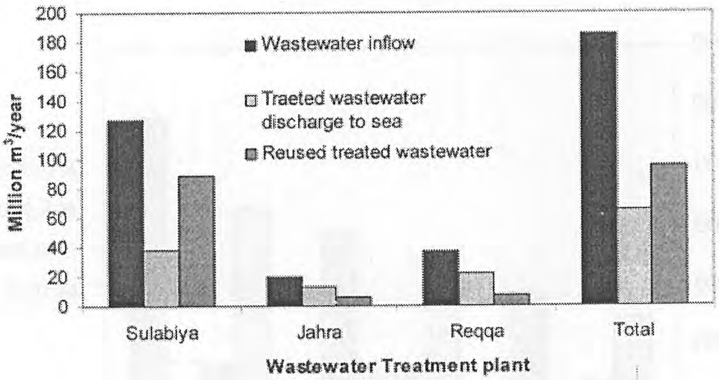

Figure 5: Quantities treated, reused and discharged to the Gulf

Given the limited available water-resources and increasing demand, municipal wastewater can have a special role in maintaining and augmenting the water resources of Kuwait especially since wastewater effluent is estimated to be 70-80% of the freshwater consumption per capita (Al-Awadi and others, 1992). Therefore, reclaimed water can be considered as a primary option for developing new water resources through water reuse in irrigation and landscaping. However, the extent of reusing this resource in agricultural activities is very limited because of health regulation and environmental concerns. Therefore, the potential of reusing conventionally treated wastewater effluent can be increased by reducing or eliminating hazardous substances. This can be accomplished by polishing this effluent with more advanced techniques. Membrane technology is a potential separation method that can be used as an advanced treatment for wastewater. Reverse osmosis (RO) and ultrafiltration (UF) are the most successful membrane separation processes which have been adopted recently in the field of wastewater treatment (Al-Shammari, S., and M. Abdel-Jawad 1998) and (Bou hamad and others, 1997). Two studies were conducted recently in the Kuwait Institute for Scientific Research (KISR) to evaluate the feasibility of RO and MF treatment of wastewater effluent from tertiary treatment plants (Abdel-Jawad 2002 and S. Bouhamad et al., 1998). The results indicated significant improvement of wastewater effluent quality of both MF and RO. For example, a significant reduction in the chemical oxygen demand (COD) and biological oxygen demand (BOD) were noticed in the filtrate and permeate of MF and RO, respectively. The main objective of Sulaibiya facilities is to produce an effluent of such quality to allow its beneficial "reuse" by consumers. The Sulaibiya Wastewater Treatment and Reclamation Project (WWT&RP) is intended to resolve two problems: the Ardiya WWTP was no longer able to treat the increasing amount of wastewater and could not be extended at its location. The brackish water resources are no longer sufficient to cover the increasing non-potable water demand. To resolve these two problems by means of one plant, the objective of the new Sulaibiya WWT&RP is to treat the wastewater to such an extent that it could be used for unrestricted non-potable Purposes. The capacity of the Sulaibiya WWT&RP (375,000 m^3/day with the option to be extended to 600,000 m^3/day) makes the project by far the largest of its type in the world. Approximately 85% of BOT inflowing wastewater will be reclaimed for reuse at its full operation and 15% will be discharged to the sea as brine. Total effluent use from three operating plants is about 51% of the total combined inflow of 185 million cubic meters per year.

Potential wastewater reuse in meeting Kuwait's water demand

The State of Kuwait used nearly 415 million m^3 (MEW, 2002) of water for domestic purposes in year 2001. Assuming 70% of the water use turned to be wastewater, the available wastewater is nearly 290 million m^3/year. Present uses of tertiary effluent is nearly 95 million m^3/year (Personal contact, MPW, 2004) which is about 33% of the potential available wastewater. The distribution of percentages for various uses appears in Table 2.

Table 2: Percentage distribution of treated wastewater use in Kuwait. (Source: Personal contact, MPW, 2004).

Item	Percentage of total potential amount
Direct use for fodder production	26
Treatment plant trees and landscaping	0.3
Roadside landscaping and others	6.7

Three planned agricultural farms presently use most of the irrigation water for fodder production. For the purpose, trunk-line and pump stations are used to convey the generated wastewater effluent to a central reservoir for distribution of the effluent to the point of use. The effluent from the Sulaibiya plant is expected to be of a quality that is appropriate for unrestricted irrigation.

Conclusion

The State of Kuwait has undertaken a pioneering step towards total utilization of treated wastewater effluent in the country for beneficial use. The process included the construction of infrastructure such as an effluent collection system needed to convey water to centralized reservoir, pump stations, trunk-lines, reservoir maintenance, water allocation, and distribution system to the point of use. Once a new system of effluent collection and distribution scheme is in full operational by year 2005, total treated domestic wastewater generated in the country will be utilized for irrigation and landscaping purposes. Other uses subsequently may also be justified based on the expected quality of effluent from the Sulaibiya plant. The step undertaken appears to be quite appropriate for a country like Kuwait which experiences extreme arid climate and immense water shortage.

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Wastewater Treatment and its Applications as a Water Supply in Libya

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WASTEWATER TREATMENT AND ITS APPLICATIONS AS A WATER SUPPLY IN LIBYA

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ABSTRACT

Groundwater in Libya has been strained due to large increases in water demand with very little recharge resulting in serious declines in water levels and quality, especially along the coastal strip where most of the domestic, industrial and agricultural activities are taking place. To develop extra non-conventional water resources, the authority started the establishment of wastewater treatment plants in 1963. Since then the development of wastewater treatment became feasible in most of the northern cities and villages. Most of wastewater treatment plants were designed to produce water for agricultural purposes. The total design capacity of installed wastewater treatment plants in1998 reached 500,000 m³/day. It is expected that in the coming years the improvement of the existing stations and construction of new plants will be able to meet the requirements of the population growth in urban areas. As the water demands exceed the available water resources every drop of water including wastewater counts. Both water resources and wastewater must be managed efficiently within the umbrella of integrated water resources management processes. The objective of this paper is to demonstrate the reuse of wastewater in Libya for agricultural purpose. To achieve this objective, an overview of the water situation is presented along with the status of wastewater treatment plants in Libya and its associated problems as an important source of water supply.

Keywords: wastewater applications treatment in Libya, water supply in Libya, water resources situation in Libya.

1. Introduction

Libya is a mostly arid and semiarid sparsely populated large North African country. Annual precipitation is low with more than 95% of the country receiving precipitation less than 100mm/year. Evaporation rates are among the highest in the world because of the dry climate with temperatures exceeding 40°C in some parts of the country. In the past three decades, a significant improvement in the standard of living has been witnessed because of Libya's vast oil resources. This has resulted in a rapid increase in its population and water consumption rates for domestic, industrial and agricultural purposes. This growth has resulted in serious pressure on the country's water resources that have suffered from depletion and quality deterioration. These impacts have promoted the search for non-conventional sources including water desalination and wastewater recycling and reuse. The objective of this paper is to demonstrate the use of treated wastewater in agriculture that can both meet irrigation demands for water and guarantee a sustainable way of disposing wastewater to the environment.

2. Water situation in Libya

Libya as many other countries in the Mediterranean region mines groundwater. This is a slowly renewable resource whose exploitation can not continue indefinitely. Several very large aquifers underlie the Libyan Desert. Examples as summarised in Table 1 are, the Kufra/as-Sarir and Murzek basins that are utilised for the "manmade river" project and which will jointly supply the Libyan coastal regions with over 6 million m³/day. The northern aquifers of Jabal al-Akhdar and Nafousah/al-Hamada receive a total of 650 million m³ annually of surface recharge, however the Gefarah plain is excessively suffering from mining groundwater since most of the agricultural activity takes place in this area.

Table 1: Major basins in Libya

Basins	Area, km^2	Utilisable water, million m^3/year	TDS, mg/l	Major problems and impacts
Jabal al-Akdar	145,000	250	1000->5000	Minor domestic and industrial pollution, seawater intrusion, water level declines
Kufra /as-Sarir	700,000	1800	200-1500	Minor domestic and agricultural pollution
Gefarah plain	18,000	250	1000->5000	Minor agricultural and industrial pollution, seawater intrusion, water level declines
Nafusah / al-Hamada	215,000	400	200-1500	Water level declines
Murzek	350,000	1800		Minor agricultural and domestic pollution

Perennial rivers and lakes do not exist in any part of Libya. Surface water resources are limited and mainly absent. The country is located in a region where the climate is characterised by hot dry summers and wet winters. The region suffers frequently from low rainfall. The total annual runoff in Libya is estimated as 385 million m^3, from which, 200 million cubic metres are in the coastal areas.

Non-conventional sources will become increasingly important. The country has already turned to desalination and reclaimed water to bridge the gap between supply and demand. Table 2 gives an overview of the available water compared to the current consumption. The quantities show a disproportion between the total withdrawal and the annual available volumes of local water resources. Clearly there is a deficit; leading to the deterioration of water quality. In 1998 the agricultural sector was responsible for the highest consumption quantity reaching 85% of the total water use. The domestic sector consumed only 11.5%, and the industrial sector only 3.5% representing the lowest portion of the total withdrawals. Groundwater is the major source of water in Libya supplying in total 95% of the nation's water needs. Surface water contributed 2.3%, whereas the desalination of seawater and recycling wastewater were the minor resources with very small shares of 1.8% and 0.9% respectively.

Table 2: Water situation in Libya in 1998

Water supply, million m^3		Water demand, million m^3	
Water source	**Water quantity**	**Demand sector**	**Water quantity**
Groundwater	2557.62	Agricultural	3259.27
Surface water	61.00	Domestic	448.30
Desalination	47.86	Industrial	135.64
Recycled wastewater	24.16		
Total	2689.64	Total	3843.21

1. Wastewater treatment application

The wastewater treatment technologies have been used in Libya since the sixties. The first wastewater treatment plant was established in 1963 in the city of Tobruk in the eastern part of the country. Ever since, their utilisation and fields of applications have grown markedly and have been feasible in most of the northern cities and villages as shown in the Figure (1) to match the growing urban sector.

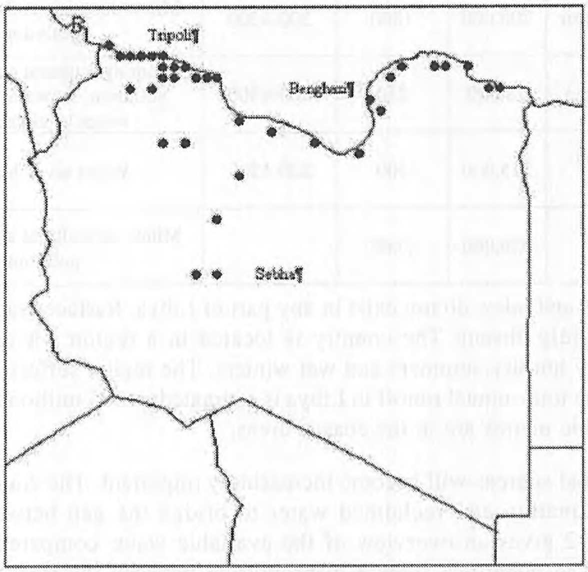

Figure 1: Wastewater treatment plants locations

3.1. Installed treatment capacities and uses

Figure (2) and Table (3) show the total design capacities, which have been installed in the country. One can easily say that the design capacity increased with a constant rate of 6500m³/day annually in the period between 1965 and 1975. It became 16000m³/day per year in 1975 and in 1985 was about 484735m³ per day. The increase rate then diminished and became almost zero in the following years. From Figure 2, it can be seen that the realisation didn't follow the design.

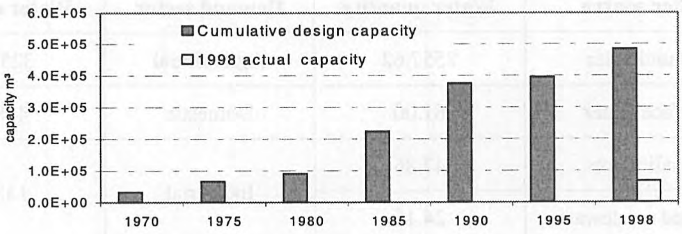

Figure 2: Overall cumulative designed and installed capacities of wastewater treatment plants.

Table 3: Overview of wastewater installed treatment plants

Treatment plants	Installation year	Design capacity, m^3/day	Existing capacity, m^3/day	Treatment kind	Remarks
Ejdabya	1988	15600	5000	Activated sludge	-
Benghazi A	1965	27300	-	Tricking filters	Out of order
Benghazi B	1977	54000	-	Tricking filters	Provisional test
Al-merg A	1964	1800	-	Activated sludge	Out of order
Al-merg B	1972	1800	-	Activated sludge	Out of order
Al-beada	1973	9000	-	Activated sludge	Under construction
Tobruk A	1963	1350	-	Tricking filters	Out of order
Tobruk B	1982	33000	-	Activated sludge	Out of order
Derna	1965	4550	-	Tricking filters	Out of order
Derna	1982	8300	-	Activated sludge	Under construction
Sirt	1995	26400	-	Activated sludge	Under construction
Abo-hadi	1981	1000	600	Activated sludge	-
Al-brega	1988	3500	2700	Activated sludge	-
Zwara	1980	41550	-	Activated sludge	Not used
Sebrata	1976	6000	-	Activated sludge	Out of order
Sorman	1991	20800	-	Activated sludge	Under construction
Zawia	1976	6800	-	Activated sludge	Under construction
Zenzour	1977	6000	-	Activated sludge	Not used
Tripoli A	1966	27000	-	Tricking filters	Out of order
Tripoli B	1977	110000	20000	Activated sludge	-
Tripoli C	1981	110000	-	Activated sludge	
Tajoura	1984	1500	500	Activated sludge	-
Tarhouna	1985	3200	1260	Activated sludge	-
Gheraan	1975	3000	-	Activated sludge	
Yefren	1980	1725	173	Activated sludge	-

Meslata	1980	3400	-	Activated sludge	Not used
Homes	1990	8000	-	Activated sludge	Not used
Ziliten	1976	6000	-	Activated sludge	Out of order
Misrata A	1967	1350	-	Tricking filters	Out of order
Misrata B	1982	24000	12000	Activated sludge	-
East Garyat	1978	500	-	Activated sludge	Out of order
West Garyat	1978	150	-	Activated sludge	Out of order
Topga	1978	300	-	Activated sludge	Out of order
Shourif	1978	500	-	Activated sludge	Out of order
Sebha A	1964	1360	-	Tricking filters	Out of order
Sebha B	1980	47000	24000	Activated sludge	-

- Most of the wastewater treatment plants were designed to produce treated water suitable for agriculture purposes.
- The Trickling filters (TF) technique was used in the first generation of treatment plants in the sixties while the Activated sludge (AS) technique is used to treat wastewater in most of the other plants.
- Twenty-five plants were built during the period (1965-1995). Three out of 25 operate with good efficiency; two with medium efficiency and the rest are either working inefficiently or out of order.
- The design capacities vary from 150 m³/day to larger ones of 110000 m³/day.
- Untreated and partially treated wastes are usually discharged to the sea or to the wadi beds outside the urban areas.
- Data of water quality are only scarcely available in all treatment plants due to the absence of either laboratories or skillful personnel.

In the framework of reusing treated water two agricultural projects were established in Tripoli and Benghazi, both cities benefit from the treated water. Table 4 shows a total of 6000 hectares being currently irrigated by treated wastewater.

Table 4: The irrigated crops in Tripoli and Benghazi areas

Project location		**Design discharge, m³/day**	**Irrigated area, Ha**	**Irrigated crops**
Tripoli	1st stage	27,000	2500	Fruit trees and animal fodder
	2nd stage	110,000	1500	Animal fodder
Benghazi	1st stage	27,000	360	Animal fodder
	2nd stage	27,000	658	Animal fodder
	3rd stage	27,000	1000	Animal fodder

3.3. Problems associated with wastewater treatment applications

The applications of wastewater treatment have been met by both successes and failures resulting in a substantial reduction in the actual production of treated water to only a small fraction of the installed capacity. These problems are associated with the operation and maintenance of treatment plants. The major obstacles associated with wastewater treatment plants are summarised below:

- The inability of operating most of treatment plants to its design capacities was caused by an incomplete sewer system in urban areas.
- Low sewage inflow due to the inefficiency of some pumping stations that lack spare parts and receive inappropriate maintenance.
- The shortage of spare parts and the absence of skillful personnel in operating have caused a smaller treatment production in some plants and a total closure of many others.
- Low salaries cause technicians to refuse work in this field and result in an administrative unstable situation concerning the operation of treatment plants.

It should be noted that the farmer refusal of using the treated wastewater due to high salinity had been expected before the transported water arrival, as some of the plants are small capacity ones and are managed by non-specialised personnel. However, two agricultural projects in Tripoli and in Benghazi being irrigated for many years thus disproving the misconceptions about the wastewater treatment technologies and its production reuse.

1. Future challenges

Libya is located in a region where the water situation is critical due to scarcity of water sources and the rapidly increasing agricultural demand. The country suffers from a shortage of water supply with serious socio-economic and environmental consequences. As conventional water sources are being confronted with serious depletions and quality deterioration, immediate measures should be taken to face these shortages including the reuse of treated wastewater in the agricultural sector.

The reuse of treated wastewater resources will become increasingly important in Libya for several reasons:

- The shortage and quality deterioration of conventional water resources.
- Excluding the cost of water treatment plants complexes, which is important to safeguard public health, a substantial and inexpensive water supply can be accessible.
- The existence of treatment plants in agricultural areas will drastically reduce costs.

As the urban sector is growing gradually, more wastewater will be produced; treated wastewater will become more competitive with other water sources for agricultural water supply. This is more likely since the conventional sources are being exhausted in the coastal region. Furthermore, treated wastewater use will free more water for other purposes. In this case, water will be better utilised nationally at low costs; thus strengthening the role of treated wastewater in the sustainable development of Libya.

Recycling domestic wastewater will thus become a potential and reliable water source of growing availability in the future of Libya's development, not only to ensure the continuous supply of water to existing agricultural projects in particular, but also to allow development of new ones.

2. Conclusions and recommendations

As the Libyan population grows, more water will be required to satisfy the needs. The present limited water resources will become more and more limited. This very high water supply demand for the future might result in a high financial and ecological price. Currently the water demands exceed markedly the conventional water resources capacities creating an urgent need for integrated water resources management with special focus on non-conventional water resources namely, wastewater reuse. The following conclusions and recommendations are made on the basis of a survey of domestic wastewater recycling that has been discussed in this paper.

- Recycling domestic wastewater has been applied successfully in Libya since 1963 however with serious problems associated with operation and maintenance.
- Wastewater treatment plants have been used to generate water only in two major cities for agricultural purposes.
- The existence of larger plants in major cities surrounded by agricultural areas makes the cost of treated water conveyance minimal and affordable.
- Water shortage, human health protection, and economic demands for efficient irrigation techniques will stress the future importance of the reuse of treated water.
- Public confidence on using products that are grown in areas irrigated by treated wastewater requires the development of educational health programs.
- The farmer's refusal attitude to the use of treated wastewater could be changed by a persuasive encouragement policy.

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WATER AND AGRICULTURE

Policies for Water Management and food Security Under Water-Scarcity Conditions: The Case of GCC Countries

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POLICIES FOR WATER MANAGEMENT AND FOOD SECURITY UNDER WATER-SCARCITY CONDITIONS: THE CASE OF GCC COUNTRIES

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ABSTRACT

After a brief presentation of the common physical and economic characteristics of the GCC countries, the paper reviews past policies on agriculture and water management in these countries, focusing on the incentive framework aimed at diversifying the economies and increasing the level of food security. It analyzes the impacts of these policies on agricultural development and highlights the main achievements in terms of increasing the level of food self-sufficiency, particularly with respect to cereals, fruits and vegetables, although the contribution of agriculture to national gross domestic products remained generally meager. In a second stage, the paper presents the water resources situation in the region, which consists essentially of non-renewable groundwater and non-conventional waters, and the analysis of the impacts of the above policies on the management of these resources, focusing on the depletion of strategic groundwater, water pollution problems and the low productivity and economic competitiveness of agriculture. To address this non-sustainable situation, the paper proposes a strategic option, in which some countries are already embarking, for planning water resources use in a pro-active manner, based on a framework that links agriculture and water policies and which is conducive to sustaining both water resources and a competitive level of agricultural production. Elements of the policy framework are also provided and include: 1) an in-depth analysis of supply and demand, 2) an assessment of the economic and social impacts of irrigated agriculture on national economies, 3) adoption of economy-wide agriculture policies, 4) adoption of policies to promote improved water use efficiency in agriculture, 5) introduction of water services cost recovery and their linkage to incentives, 6) and enhancement of water supply and alternatives to freshwater.

Key words: water management, non-renewable groundwater, water-scarcity, virtual water, food security.

Introduction

In the GCC countries, water resources are in short supply, while demand for water is growing. Given this situation, the question of the optimal management of water resources is clearly of crucial importance, with implications not only for the future development of those countries, but also for the sustainability of their past economic and social achievements. The dilemma arises from continuing growth in demand, which is the result of population increase and other social factors, in conjunction with the fact that the region is already exploiting all its annual surface water resources, while its aquifers are becoming depleted.

Until recently, water management policies concentrated on the supply side. Driven by the optimistic agricultural policy of achieving food self sufficiency, the governments of the GCC countries directed water development projects to the continuously expanding agriculture sector. In addition, economic incentives in the form of subsidies helped in boosting agricultural production, and consequently put more stress on the available water resources.

Lacking necessary planning and future insights has led to the consumption of fossil (non-renewable) groundwater aquifers as demands exceeded the renewable water resources. It is now feared that groundwater depletion will eventually lead the GCC countries to an even more severe water crisis. During the past few years, these countries started changing their water management and food security policies, but additional measures are still needed.

The GCC countries are facing a water crisis that is becoming more critical. It is imperative to adopt realistic policies and institutional arrangements that will enable to control demand for water, apportion the available quantities along economically efficient lines, and ensure that water is used more efficiently in various sectors.

This paper reviews and outlines the agricultural and water resources policies in the GCC countries with respect to food production and their impacts on water resources sustainability. It provides a rationale for a review of these policies and gives basic elements for a policy framework that links water resources and food security, with a view of balancing water supply and demand and sustaining economic development as it relates to these sectors.

1. Background and Main Characteristics

Countries of the Gulf Cooperation Council (Bahrain, Kuwait, Oman, Qatar, Kingdom of Saudi Arabia, and the United Arab Emirates) occupy most of the Arabic Peninsula and share similar economic, social, and physiographic characteristics. They have a typical desert climate, with the exception of the coastal strips and mountain ranges, characterized by long, hot, dry summers and short, cool winters, for the interior regions, and hot, somewhat more humid summers and mild winters, for coastal regions.

The total population of the region has more than quadrupled between 1970 and 2000, going from less than 8 million to over 30 million. It stands currently at around 36 million and is expected to reach about 58 million by the year 2030. In general, the GCC countries have a similar socio-economic situation in terms of features and development with the booming of oil industry and high revenues during the last 40 years. The economy is dominated by petroleum, which accounts for 80% to 90% of merchandise export earnings except in Saudi Arabia and Oman. Agriculture accounts for a small portion of the GDP and does not constitute an important source of employment, except in Oman and, to a lesser extent, Saudi Arabia (Table 1). However, agriculture is also embedded in the population culture and bears an important social value through amenity farms and secondary recreational residences, despite the non-profitability of this type of farming which covers an important part of the agricultural lands. However, agriculture in all GCC countries depends on irrigation and uses 80% to 90% of the water resources.

Table 1: Agricultural contribution to GDP of the GCC countries in 2002

Country	Agricultural Labor Force (1000)	Agricultural GDP (million)	Total GDP (million)	Agricultural GDP in % of total GDP
Bahrain	3	54 US$	7683 US$	0.7
Kuwait	14	142 US$	35369 US$	0.4
Oman	358	650 US$	20309 US$	3.2
Qatar	4	70 US$	17466 US$	0.4
Saudi Arabia	680	9612 US$	188479 US$	5.1
UAE	71	2555 US$	70960 US$	3.6

Source: FAO (2004) Compendium of food and agriculture indicators http://www.fao.org/es/ess/compendium_2004/default.asp

2. Agricultural Production and Food Security

Over the past 20-30 years, the GCC countries aimed at achieving a greater level of food self-sufficiency and accordingly most of these countries adopted a policy to expand the irrigated area through economic incentives. Table 2 shows the evolution of the area under irrigation during the period between 1965 and 2002. In general, this area increased by 5% per year up until 1990, then by 1.2% per year thereafter. The areas of most crops increased steadily between 1980 and 1999 in most countries, although there were marked declines in cereals in Saudi Arabia and the UAE between 1990 and 2003 (Table 3).

The net productivity of agriculture increased two-and-half folds between 1980 and 2000 (Table 4). It increased steadily in Oman and very rapidly in the UAE, but declined in Saudi Arabia reflecting the abandoning of some areas where groundwater resources have been exhausted and because of declining prices for some crop products. A severe drop in agricultural productivity in the UAE occurred between 2000 and 2004; it is also attributed to the depletion of groundwater resources and the consequent change in agricultural policy.

Table 2: Evolution of the area under irrigation in GCC countries during 1965-2002 ('000 ha)

Country	Year								
	1965	1970	1975	1980	1985	1990	1995	2000	2002
Bahrain	1	1	1	1	1	2	4	4	4
Kuwait	0	1	1	1	2	3	5	10	13
Oman	23	29	34	38	41	58	62	62	62
Qatar	1	1	1	3	5	6	13	13	13
KSA	353	365	375	600	1,150	1,600	1,620	1,620	1,620
UAE	35	45	50	53	58	63	68	76	76
GCC Countries	413	442	462	696	1,257	1,732	1,772	1,785	1,788

Source: FAOSTAT 2004

Table 3: Evolution of cropped areas in GCC countries during 1980-2003 ('000 hectares)

	Vegetables and Melons				Cereals				Fruit (including dates)			
Country	**1980**	**1990**	**2000**	**2003**	**1980**	**1990**	**2000**	**2003**	**1980**	**1990**	**2000**	**2003**
Bahrain	0.6	0.8	1.1	0.9	-	-	-	-	4.5	1.6	1.5	2.3
Kuwait	1.3	2.9	3.8	3.9	0.1	0.5	1.2	1.7	0.0	0.4	1.4	1.4
Oman	5.3	9.4	9.2	9.4	2.7	2.4	2.5	2.5	27.1	33.4	41.6	42.0
Qatar	1.3	2.2	3.3	2.8	0.2	1.1	1.6	1.5	1.2	1.4	1.8	2.0
KSA	53.5	102.3	79.5	100.7	453.0	974.6	616.4	666.1	72.0	90.9	193.4	194.0
UAE	4.7	7.5	38.0	10.9	0.4	1.3	0.1	0.0	7.7	25.1	187.8	188.2
Total	**66.7**	**125.0**	**134.9**	**128.6**	**456.4**	**979.8**	**621.7**	**671.8**	**112.6**	**152.9**	**427.5**	**429.9**

Source: FAOSTAT 2004

Table 4: Agriculture net productivity index in the GCC countries during 1965-2004

Country	Year								
	1965	1970	1975	1980	1985	1990	1995	2000	2004
Bahrain	44.1	45.0	79.2	141.4	100.3	87.8	101.0	103.2	110.2
Kuwait	14.4	19.8	23.1	44.4	64.9	53.0	55.0	91.0	124.6
Oman	13.6	15.4	21.5	35.0	53.5	59.3	66.7	97.5	91.1
Qatar	16.6	23.2	10.9	23.6	30.6	63.4	103.5	111.7	143.9
KSA	15.4	18.6	30.5	26.5	56.6	104.0	87.2	93.0	108.2
UAE	3.1	4.5	5.7	10.1	15.2	21.3	41.8	140.9	53.4
GCC Countries	107.2	126.6	171.0	280.9	321.0	388.8	455.2	637.2	631.4

Source: FAOSTAT 2004

The policy of GCC countries to diversify their economies and to increase the level of food self-sufficiency focused on economic incentives for expanding the area under irrigation and improving crop productivity. Government provided subsidies in several forms, including wells, fuel, energy, inputs, price support programs, trade protection, and free access to unlimited amounts of groundwater most of which is non renewable.

The absence of administrative and legal regulations to control pumping encouraged unlawful drilling and resulted in inefficient irrigation practices that led to the loss of more than 50% of the amounts of applied water. The subsidies increased the irrigated areas as well as agricultural production, but resulted in the mining of fossil aquifers. As a result, many farms have been abandoned in all countries and several aquifers have been either depleted or highly polluted. From the economic standpoint, the subsidies distorted costs and revenues and many of the agricultural activities were financially profitable only because of government subsidies and incentives. Although agriculture consumed about 80-90% of the available water in most countries, it contributes far less than 1% of the Gross Domestic Product.

Implementation of the policy to increase the level of self-sufficiency achieved its objectives to a certain extent, but at an extremely high economic cost and the loss of huge amounts of traditional reserves of fossil water resources. Yet, the countries continued to have low food sufficiency rates for the main food commodities, with the exception of fruits and vegetables to a certain extent, as indicated in Table 5.

Table 5: Self sufficiency percentages for main commodities in the GCC Countries (2003)

Country	Commodity	Production (1000 MT)	Imports (1000 MT)	Supply (1000 MT)	Self- Sufficiency Rate (%)
Bahrain	Cereals	0	76,532	76,532	0.00
	Fruits Total	22,010	77,479	99,489	22.12
	Vegetables Total	8,394	83,243	91,637	9.16
Kuwait	Cereals	3,300	559,976	563,276	0.59
	Fruits Total	11,511	73,952	85,463	13.47
	Vegetables Total	184,065	132,178	316,243	58.20
Oman	Cereals	5,771	425,630	431,401	1.34
	Fruits Total	294,636	108,692	403,328	73.05
	Vegetables Total	183,673	80,745	264,418	69.46
Qatar	Cereals	6,385	76,292	82,677	7.72
	Fruits Total	17,990	22,891	40,881	44.01
	Vegetables Total	45,630	80,880	126,510	36.07
KSA	Cereals	2,379,000	9,858,802	12,237,802	19.44
	Fruits Total	1,250,000	635,398	1,885,398	66.30
	Vegetables Total	1,854,000	382,285	2,236,285	82.91
UAE	Cereals	120	2,405,068	2,405,188	0.00
	Fruits Total	797,800	362,595	1,160,395	68.75
	Vegetables Total	546,129	461,018	1,007,147	54.23
GCC Countries	Cereals	2,394,576	13,402,300	15,796,876	15.16
	Fruits Total	2,393,947	1,281,007	3,674,954	65.14
	Vegetables Total	2,821,891	1,220,349	4,042,240	69.81

Source: FAOSTAT 2004

3. Water Resources Availability and Use

Under the prevailing climatic conditions, the GCC countries receive only scanty rainfall (less than 100 mm on the average) whereas the mean evaporation exceeds 300 mm per year. The region is devoid of perennial surface waters such as rivers or lakes. Freshwater demands have been met by groundwater extraction until the introduction of desalination of seawater in the 1960s and recycling of treated wastewater. The main sources of freshwater are groundwater, most of which is non renewable, and a limited amount of renewable surface water. Non-conventional sources, including brackish water, desalinated seawater and wastewater constitute important components of the annual water balance (Table 6.)

The other main source of water is non-renewable fossil groundwater stored in sedimentary deep aquifers. Huge amounts of groundwater are stored in these deep aquifers with a reserve estimated at 2200 billion cubic meters out of which about 1920 billion cubic meters are located in Saudi Arabia. The bulk of these resources is of brackish quality, with great variations in total dissolved solids (200 to 20,000 ppm), which allows the suitable use for domestic purposes in few areas.

Another feature of groundwater in the GCC region is quality degradation. As water tables drop due to excessive extraction, salinity increases and intrusion of seawater takes place in coastal areas. Several aquifers in Bahrain, Oman, Qatar and UAE are already severely affected by this phenomenon.

Table 6: Water resources in the GCC countries by source (millions of cubic meters)

	Conventional water resources				Non-conventional water resources	
Country	**Surface water**	**Ground water recharge**	**Ground water use**	**Total Renewable Resource**	**Desalination**	**Wastewater (+ drainage reuse)**
Bahrain	0.2	100	258	100.2	75	17.5 (3)
Kuwait	0.1	160	405	160.1	388	30
Oman	918	550	1 644	1468	51	23
Qatar	1.4	85	185	86.4	131	28
KSA	2 230	3 850	14 430	6080	795	131 (24)
UAE	185	130	900	315	455	108
Total	3334.7	4875	17822	8209.7	1895	364.5

Source: Compiled by ESCWA from country papers presented at expert group meetings and international sources, 1995, 1996, 1997 and 1999.

The increase in demand for potable water led the GCC countries to resort to desalination of seawater which started in the 50s and early 60s. The region has gained significant experience in desalination and produces more than 40% of the total desalinated water in the world. The use of natural gas as a fuel for desalination promises to be more efficient, less costly and more environmentally safe than the use of petroleum. However, the use of desalinated water for agricultural production is still not economically feasible.

The collection and treatment of municipal wastewater has reached about 60% in the GCC countries. In most countries, treated water is used for the irrigation of landscaping after treatment to the tertiary level. It is inevitable that these countries will depend largely on the use of this type of water, although its proper use is still controversial. Table 7 gives an indication of desalination and wastewater treatment capacity and production, by country, in the year 2002.

Table 7: Desalination and wastewater capacity and production in GCC countries in 2002

Country	Installed Capacity (Mm^3/year)	Desalination Production (Mm^3/year)	Treated Wastewater Production (Mm^3/year)	Treated Wastewater Used (Mm^3)
Bahrain	138	138	24	24
Kuwait	522	444	258	250
Oman	103	103	10	9
Qatar	178	158	44	44
Saudi Arabia	1278	1022	475	n/a
UAE	952	811	227-265	205

Source: unpublished reports

Table 8 gives and indication of the amounts and fractions of different sources of water allocated to various uses, around the end of the last century. Based on these trends, the following quantified insights analysis for the water situation in the region can be given:

Table 8: Water resources allocation by country

Country	Resource Type	Municipal (Mm^3)	Agricultural (Mm^3)	Industrial (Mm^3)	Total Use (Mm^3)
Bahrain	SW		0.2		0.2
	Ren.GW	52	48		100
	Fossil GW		120.3	37.7	158
	Desal.Sea	75			75
	TWW		20.5		20.5
	Total	127	189	38	354
Kuwait2002	SW		0.1		0.1
	Ren.GW		76	84	160
	Fossil GW		587		587
	Brackish	112	300	3	415
	Desal.Sea	375	27	18	420
	TWW	12	66		78
	Total	499	1056	105	1660
Oman	SW		918		918
	Ren.GW		465	85	550
	Fossil GW	474	620		1094
	Desal.Sea	51			51
	TWW		23		23
	Total	525	2026	85	2636
Qatar2003	SW		1.4		1.4
	Ren.GW	2.35	35.8	1.85	40
	Fossil GW	25.65	174.35		200
	Desal.Sea	119		42.8	161.8
	TWW		27		27
	Total	147	239	45	430
KSA2000	SW	470	1760		2230
	Ren.GW		2450	550	3000
	Fossil GW	1300	11740		13040
	Desal.Sea	1150			1150
	TWW		150		150
	Total	2920	16100	550	19570
UAE	SW		185		185
	Ren.GW		130		130
	Fossil GW	295	1060	130	1485
	Desal.Sea	455			455
	TWW		25		25
	Total	750	1400	130	2280
Total GCC	SW	470	2865	0	3335
	Ren.GW	54	3205	721	3980
	Fossil GW	2095	14302	168	16564
	Brackish	112	300	3	415
	Desal.Sea	2225	27	61	2313
	TWW	12	312	0	324
	Total	4968	21010	952	26931

- Irrigated agriculture is by far the largest water user with around 78% of the total water use in all GCC countries, in comparison with 18% and 4% for municipal and industrial uses respectively[1]. Most agricultural water (85%) is taken from groundwater, out of which 81% is non-renewable.

- Domestic water use relies on both desalination and good quality groundwater (usually fossil) with the exception of Saudi Arabia where surface water is available to some extent. Another striking feature with respect to municipal water use is that GCC countries have the highest per capita consumption in the world, in contrast with being the least endowed with renewable water resources. Potable water demand is around 70 percent higher than in rich countries considered to be high users of water such as the USA or Australia. Inadequate demand management and inefficient service delivery are believed to be the main reasons behind this situation.
- Although domestic wastewater is about 4860 Mm^3 and there are significant treatment facilities in most countries, the amount reused in agriculture is very small and almost negligible in all countries (about 2% of total water use).
- When considering all six countries, domestic water supplies depend equally on desalination and fossil groundwater (about 50% each). The percentage of desalination becomes 73% when excluding Saudi Arabia and Oman. The latter depends on groundwater for 90% of its drinking supplies, in comparison with 61% for KSA. Qatar and Kuwait rely essentially on desalination for domestic water supplies because of the depletion of fresh groundwater; they are followed by UAE and Bahrain.
- According to the available information, Kuwait is the only country reported to use treated water for municipal purposes (most probably toilet flushing).
- Industrial water use (4% on the average in all countries) relies for 94% on groundwater (of which 80% is non-renewable) and 6% on desalination (Kuwait and Qatar). However, treated wastewater is not used at all, despite its suitability for some industries. Excluding Saudi Arabia and Oman, the ratio of industrial water consumption becomes 7% in the other 4 States.
- Total amount of domestic water in the GCC countries is 4856 Mm^3, while the amount of reused treated wastewater (according to the available information) is only 324 Mm^3. This represents about 6.6% of the domestic water or 1.2% of the total water resources in the GCC countries. Excluding Saudi Arabia and Oman, this ratio becomes 10.7% for the other 4 States. Assuming that 50% of the municipal water can be treated and reused for agricultural purposes in the future, an amount of 2428 Mm^3 can be deducted from the agricultural demands, which represents about 11% of the irrigation water, or about 17% of the extracted fossil groundwater.
- However, if we exclude Saudi Arabia and Oman, the 50% of treated domestic wastewater for the remaining 4 States is 705 Mm^3. The corresponding agricultural water uses for these countries is 2584 Mm^3 from which 1942 Mm^3 is fossil groundwater. This indicates that it is possible to save about 27% of irrigation requirements or about 36% of the abstracted fossil groundwater.

[1] There are common features from the standpoint of water allocation from different sources, between Bahrain, Kuwait, Qatar and UAE, on one hand, and KSA and Oman, on the other. This is mainly due to the fact that KSA and Oman have some renewable surface water resources. When considering only the first four countries, the municipal water use ratio from water resources becomes 33% instead of 18%.

- The sustainability indicator of renewable water resources shows a glim future for all countries, unless courageous policy measures are taken and implemented urgently (Table 9). Figures of 10 to 20 percent indicate good management practices, whereas those over 40 per cent indicate mismanagement.

Table 9: Sustainability indicator of water resources use by country in the year 2000

Country	Total Renewable W. Resources (Mm^3)	Total Annual Water Use (Mm^3)	Annual/capita potable water use (m^3)	Annual/capita water Use (m^3)	Sustainability indicator (%)*
Bahrain	100.2	282	196.43	419.64	281.44
Kuwait	160.1	590	168.16	264.57	368.52
Oman	1 468	1847	107.29	756.35	125.82
Qatar	86.4	347	242.57	572.61	401.62
KSA	6 080	17765	109.38	826.89	292.19
UAE	315	2180	230.98	671.39	692.06
Total	661.7	23011	175.80	585.24	360.28

*Water use sustainability indicator = water use/renewable resource

4. Need for a Water and Agriculture Policy Framework

While water is a vital element for economic development, it is clear from the above analysis that the current policies on water resource use in agriculture are not viable on the medium and long range. Achieving a sustainable economic development under the prevailing water-scarcity conditions in the GCC region requires elaboration and implementation of water and agriculture policies that are closely linked under a consistent framework and which are likely to meet future development challenges. Such a policy framework would help countries to plan in a pro-active and most effective manner for appropriate measures such as investment in water resources, water allocation, water tariff application, support to the agriculture sector, cropping patterns, food security, etc. The social and environmental dimension should also be given due consideration. The strategic policy framework will be instrumental in moving countries towards well known objectives; however, it can have adverse effects when it no longer serves the purpose and becomes an end itself.

Some countries of the GCC are already embarking in this direction, in collaboration with FAO[2] and other organizations[3], although not all the recommendations below are being considered at present. The basic elements of the recommended policy framework are described below:

1. Water supply and demand analysis

The starting point would be an analysis to establish a baseline water balance, specifying supply and demand. The projection of supply is based on assessment and availability of surface and groundwater for agriculture, domestic and industrial uses; whereas water demand

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[2] FAO is collaborating with the government of Kuwait, Qatar and UAE for reviewing/ establishing their agriculture strategies including water use. A collaboration programme

also exists between FAO and KSA.

[3] In the case of KSA, the World Bank is collaborating with the Ministry of Water Resources and Electricity for elaborating a national water resources strategy.

is based on economic parameters such as population, income and agriculture economy. This important component will determine the scale of the problem of water scarcity and its deficit, to assess how far ongoing measures are succeeding and to offer options for supply enhancement and demand management policies to meet the growing demand.

2. Assessment of economic and social impacts of irrigated agriculture on national economies

The objective of this assessment is to evaluate the actual economic and social impact of irrigated agriculture on the economy, with a view of identifying the level of competitiveness of irrigated agriculture in comparison with other water use sectors. When a hectare of land is converted from rainfed to irrigation, it requires an investment the level of which depends on the irrigation system introduced and the associated land betterment, in addition to the required management. In return, this investment results in a value-added on the economy and for the farmer.

The traditional way of assessing the impact of irrigation is through a comparison of the financial situation of farmers' income with and without the project, in addition to the social improvements in the irrigated area in the case of medium and large irrigation schemes. It does not take into account the impacts of water use in agriculture on prospects for meeting pressing alternative demands as well as on the economy at large. To overcome this limitation, economic equilibrium models can be used for analyzing national economic data and extracting the macro-economic impacts of water use in agriculture. The latter includes both the economic value added upstream of farms and the impacts induced by irrigation downstream. These impacts, coupled with micro-economic and social impacts, constitute important decision tools for areas of investments on water use based on comparative advantages.

3. Economy-wide agriculture policies

Past food security policies were based on area expansion to support the objectives of increased self-sufficiency and enhance exports. The supply enhancement era witnessed unprecedented growth not only in irrigation but also in groundwater development with the advent of new technology, subsidy in credit and low energy costs. That era seems to have peaked out and its continuation has proved to be non sustainable. Future increases in agricultural production must come from the increased land and water productivity, both in terms of higher yields and cropping intensities for which scope still exists. This will lead to greater water savings by reducing wasteful water losses to low economic value crops and achieving more efficient water use and better agronomic practices.

It is also important to assess the incentive structure facing key food security crops like wheat and other strategic crops and to assess the comparative advantage in producing these commodities in the region. The cost of water is the prime determining factor in mapping economic feasibility of producing these crops. It is well known that the more a resource or input factor gets scarce, the more it becomes the limiting factor for its different uses. Under these conditions, both science and common sense indicate the necessity of optimizing the use of this factor. The activity of a product to which water is allocated should bear a value that justifies the amount allocated under the social and economic conditions of the country. The opportunity cost of water is determined by the highest value associated with its possible uses; but under severe scarcity conditions the real value of water becomes even higher and is more difficult to determine. The opportunity cost of water should also consider comparison between the cost of producing a commodity locally or importing it.

The reliance on virtual water for water scarce countries is obviously a policy option which can theoretically lead to a win-win solution. Importing countries save precious water resources and can reallocate them to more valuable uses in agriculture or in other sectors. At a global level this usually generates water savings. However, there are many obstacles that should not be underestimated: barriers to market access and lack of reciprocity in trade, issues of quality and safety of food products in developed countries markets, warranty and reliability in accessing staple food supplies for importing countries. In the case of GCC countries which are already large food importers, the concept of virtual water is well founded and its comparative advantage can easily be translated into a competitive advantage since these countries are politically strong and have well developed and diversified economies. Policies to eliminate high water consuming, low-value crops such as cereals, forages and other crops, can bring about a balance to water supply and demand in many countries of the GCC.

The food security and water link at the micro level is also very important. Under reform programs, most of the input subsidy other than water have been removed or are in the process of being removed. The emerging incentive structure is bringing a cost/profit squeeze on the farm sector. The key question is to cost water or design a cost recovery program that keeps the delicate balance between farm profitability, food availability and food costs, considering the issues of efficiency, equity and environment. This is a policy challenge that needs to be addressed and requires more analytical work. As the issue of virtual water should be analyzed in the above framework, any reform in the water policy area, such as putting a cost on water, has to be evaluated in the context of economy wide and sectoral policies.

4. Policies to promote water use efficiency in agriculture

Water demand management in agriculture opens avenues for producing more with less water. This potential for improvement resides in the generally poor performance of irrigated agriculture in the GCC countries in terms of productivity per unit of water, even under modern irrigation techniques. However, unlocking this potential is not as easy as it seems. It requires fundamental changes in attitudes, policies and practices as well as investments and willingness to take and implement decisions.

Given the limited water supply for irrigation in the long term, demand management measures should aim at increasing both the productive and allocative efficiency. In the future, agriculture is expected to use less water to produce more in to order to supply the growing demand for food and at the same time to release some of the water it is now consuming. There is a need for more efficient use of water in agriculture, which can come about by more efficient irrigation practices, improving water control at farm levels and by eliminating irrigation on marginally productive lands. This is already taking place in some of the GCC countries as increase in demand for quality food over the past two decades has led to the emergence of a few private farms specialized in high-value agricultural production, using up-to-date technologies and management tools and realizing marginal profit. Economic and technical mechanisms to achieve water use efficiency in agriculture should be analyzed in the specific context of the country.

5. Incentives and water services cost recovery

Recovering costs and setting water service tariffs are essential tools for sustaining reliable services and promoting water conservation. Incentives and service cost recovery constitute the most reliable means for controlling water use in the GCC countries since it is practically difficult to rely on other procedures such as the allocation of water quotas and well metering. Governments in the region need to formulate cost recovery policies and to suggest appropriate water service tariffs under the prevailing conditions. The issue of water service pricing should be addressed in a wider context of the incentive structure facing the farm sector. The current incentive framework adopted in most countries of the region encourages expansion of the irrigated area and water wastage. It should be re-oriented towards promoting water conservation and enhancing productivity.

The context of agriculture and water management in the GCC countries has specific characteristics including: 1) agricultural production relies essentially on groundwater which is difficult to control by nature; 2) farmers are private and have their own wells which they manage with no interference from public institutions; 3) farm owners are entrepreneurs rather than farmers; 4) irrigation labor is essentially foreign. Under this context, the most reliable means of implementing an agriculture and water policy framework is through a set of dissuasive and persuasive measures at the macro-level.

Economic and policy instruments offer a pathway to improving irrigation performance. Although the need for self-financing and cost recovery has been a goal in irrigation development for some time, it was the Dublin International Conference on Water and the Environment in 1992 that established the idea that water was an 'economic good'.

Principle No. 4 – Water has an economic value in all its competing uses and should be recognized as an economic good. Within this principle, it is vital to recognize first the basic right of all human beings to have access to clean water and sanitation at an affordable price. Past failure to recognize the economic value of water has led to wasteful and environmentally damaging uses of the resource. Managing water as an economic good is an important way of achieving efficient and equitable use, and of encouraging conservation and protection of water resources.

This is now accepted as a 'keystone' of the international consensus on future needs for water policy and resource management. Water pricing policy is recognized as a key instrument for improved water allocation, better conservation and quality preservation. It can induce better demand management of water resources and is seen by many as the ultimate solution in water-deficit areas where supply is limited or cannot be augmented. It has led to the current focus on water charging – policies, practical actions and mechanisms required to set a price for water, how charges will be levied and how revenue will be collected.

6. Supply enhancement and alternatives to fresh water

The focus of supply enhancement would center on building sound technical, institutional, and economic criteria for investing in water resources management infrastructure projects. Preparing strategies prioritizing water needs in the short, medium and long range, preferably based on results from decision support models/systems to enhance water allocation decision-making among competitive sectors; and high priority infrastructure requiring rehabilitation and/or nearing completion, with emphasis on improving quality-at-entry of projects by supporting project preparation. A large scope of increasing supply of water for agricultural use still exists in the GCC countries, but this potential has not received the focus it deserves. Investment priorities should include the following:

1. The use of all treated wastewater, through a review of the current policies to include food crops, setting-up of standards, regulations and monitoring systems, and capacity development for the safe use of these resources. To this end, an International Centre for Research and Technology on Wastewater Treatment and Reuse is under establishment in Kuwait and could benefit the entire GCC region.

2. The use of brackish water, both after desalination and raw. The region is endowed with large quantities of brackish water that is suitable for agricultural production, provided that the appropriate precautionary measures and use practices are applied. The International Centre for Biosaline Agriculture was established in the region to serve for this purpose, but additional commitment by the GCC countries to reap this potential is still needed.

3. The potential of water harvesting and groundwater recharge is very important in the region. Adapted research and pilot testing would open avenues for taping this potential and identifying the technology and practices adapted to the region.

4. Desalinization of brackish water for agricultural production has started in the GCC countries and demonstrated its cost-effectiveness. However, support to lower desalination costs and to pilot test the conditions for up-scaling the practices is still needed.

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Optimization of modern irrigation for biosaline agriculture

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OPTIMIZATION OF MODERN IRRIGATION FOR BIOSALINE AGRICULTURE

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ABSTRACT

Supplemental irrigation water is a must to offset the water requirement to produce profitable crops in most arid and semiarid zones, where fresh water resources are insufficient to meet the pressure of irrigated agriculture. This necessitates the use of poor quality water resources. These waters if not properly managed and used can cause serious soil related problems (salinity, sodicity, destruction of soil structure) in addition to decline in crop yields. Biosaline agriculture (using saline water on saline soils to grow salt-tolerant crops) becomes the only option for the farmer when both soil and water resources are saline and the water resource is scarce. In this regards key design considerations must be taken into account when irrigating with salty waters to optimize water uses and to reduce subsequent soil salinity development. Sprinkler irrigation systems are commonly used in irrigation of large-scale agricultural production systems. However, they tend to concentrate salts on the leaves of plants. For this reason discharge and degree of overlap between consecutive sprinkler heads, are key design parameters when applying salty waters. Trickle irrigation is the most efficient system and is gaining importance in the GCC countries in agriculture and landscape irrigation. The objective of this study was to optimize modern irrigation systems through development of design standards for drip (emitters spacing) and sprinkler irrigation systems (single head jet and overlapping) by applying saline water. The effect of emitter spacing (drip) and overlapping (sprinkler) were tested for the formation of salt contours in soil. The leaching ratio (LR) is the overall soil salinity within the rhizosphere divided by the average irrigation water salinity. In this study LR is used to evaluate the effectiveness of irrigation systems in developing soil salinity. From the present investigations it is concluded that when using saline water for irrigation, the soil salinity development can be significantly reduced by decreasing emitter spacing i.e. an ECe of 26, 90 and 126 dS/m was developed with 25, 50 and 75 cm emitter spacing respectively. Microsprinklers are more effective in terms of leaching capability as compared to impact sprinklers. Overlapping in sprinkler irrigation reduced the evaporation compared with single jet where no overlapping was made. This has a direct effect on soil salinity development. Wind has a significant effect on the water distribution (sprinkler experiment) and subsequent salinity development and can cause long-term salinity problems. Windbreak can offer solutions to this effect.

Keywords: Optimization; biosaline agriculture; trickle; sprinkler; leaching ratio

Introduction

World wide concerted efforts are being made to improve irrigation water-use efficiencies to enhance crop production in irrigated agriculture, where supplemental irrigation water is a must to offset the water requirement of crops and to produce profitable crops in most arid and semiarid regions. The general shortage of the good quality water in most of the semi-arid and arid zone countries necessitates the use of saline water. However, the injudicious use of saline water is often associated with the development of soil salinity.

It is generally recognized that saline water affects soil properties and plant growth. The misuse of this saline water ultimately converts the good soils in to saline soils. Saline soils are significant as formations of ecosystem on the earth affected by high concentrations of soluble salts, and as means of crop production with little economic value due to salinity. Many plants either fail to grow in saline soils or their growth is retarded significantly. However, few plants grow well on saline soils; therefore, soil salinity often restricts options for cropping in a given area. Under extreme saline conditions (soil and water) the only choice is the adoption of biosaline agriculture, which is gaining importance in countries where soil and water resources are degraded.

The success of irrigated agriculture lies both with the quality (fresh, brackish, saline) and quantity of irrigation water (irrigation budgeting). These waters if not properly managed and used can cause serious soil related problems (salinity, sodicity, destruction of soil structure) in addition to decline in crop yields. In this regards key design considerations must be taken into account when irrigating with salty waters. Sprinkler irrigation systems are commonly used in irrigation of large-scale agricultural production systems. However, they tend to concentrate salts on the leaves of plants. For this reason discharge and degree of overlap between consecutive sprinkler heads, are key design parameters when applying salty waters. Trickle irrigation is the most efficient system and is gaining importance in the Gulf Cooperation Council (GCC) countries in agriculture and landscape irrigation. The leaching ratio (LR) is the overall soil salinity within the rhizosphere divided by average irrigation water salinity. In this study LR is used to evaluate the effectiveness of irrigation systems in developing soil salinity. A sustainable production system is achieved when the LR approaches unity.

The objective of this study is to optimize modern irrigation systems through development of design standards for drip (emitters spacing) and sprinkler irrigation systems (single head jet and overlapping) applying saline water. The effect of emitter spacing (drip), overlapping (sprinkler) were tested for the formation of salt contours in soil. The results were used in models to validate and calibrate the salts distribution.

Resource Information

The International Center for Biosaline Agriculture (ICBA) Dubai United Arab Emirates, occupies an area 100 ha, of which 37 ha area is allocated for experimental purposes. The soils of ICBA are generally level, loose sandy surface, very deep and calcareous. Due to the sandy nature, the soils have very high drainage capacity (well to somewhat excessively drained), and are moderate to rapidly permeable. The soils are developed from wind blown sandy calcareous material and are highly prone to wind erosion, the windbreakers at the ICBA station offset the wind effects to a certain extent. Organic matter is very low (<0.5%) and the Munsell Soil Color-dry (GretagMacbeth. 2000) is 10YR 6/4 pale brown, which is a composite reflection from the dominance of carbonates and sand, with insignificant contribution of organic matter to color composition.

There is one water source (flow rate 45 m^3/hr) at the ICBA station that is saline (Electric conductivity (EC) 30 dS/m) and the quality fluctuates slightly with aquifer recharge after heavy rain. Water composition is shown in Table 2. Fresh water (EC = 3 dS/m) is brought from Dubai-Al-Ain area at Habab, which flows from Dubai Municipality water line at a rate of 40 m^3 per hour. There are two water pumps, which extract water from these two sources. From these sources two water lines run parallel to each other and enter into mixing chambers where two waters are mixed in different ratios to achieve desired salinity levels before entering into experimental plots.

Soil Taxonomic Class of the Experimental Site

The experimental site was assessed for taxonomic class using the norms and standards of the United States Department of Agriculture "Soil Taxonomy" (Soil Survey Division Staff, 1993; USDA-NRCS, 1999 & 2003). The soil is classified as **carbonatic, hyperthermic Typic Torripsamment**. Where carbonatic is the mineralogy class i.e., more than 40% $CaCO_3$ in fine earth fraction , hyperthermic is the soil temperature regime (the mean annual soil temperature is 22°C or higher, and the difference between mean summer and mean winter soil temperature is more than 6°C at a depth of 50 cm from the soil surface). Typic torripsamment indicates typical desert sandy soil at soil subgroup level of USDA Soil Taxonomy.

Physical and Chemical Characteristics of Surface Soil

The methods used are from USDA-NRCS (1996), except where otherwise stated. Complete Particle Size Distribution Analysis (PSDA) was made by using the modified hydrometer method (Day, 1965; Shahid, 1992) supplemented with wet sieving (that allows quantification of sub fractions of sand) suitable for soils with low organic matter contents. The data (sand, silt, clay) presented is on less than 2 mm basis. Textural class is reported by plotting the sand (2-0.05 mm), silt (0.05 to 0.002 mm) and clay (<0.002 mm) values on the textural triangle (Soil Survey Division Staff, 1993). The saturation percentage (SP) is determined by the volume of water added to a known amount of soil to prepare saturated soil paste; the SP value is plotted into the model suggested by USDA (USDA-NRCS, 1995) to determine water retention at 15 bars (W15) and available water capacity (AWC) of soils. The pH was measured on a saturated soil paste (pHs) and the EC in the saturation extract collected from the saturated soil paste under vacuum. The calcium carbonates equivalents were determined by the Back Titration procedure, where a known amount of soil was reacted with a known amount of 1N HCl, and the unused acid was back titrated against 1N NaOH solution in the presence of phenolphthalein indicator to a pink color end point. The soil results are presented in Table 1, which clearly reveals that soil is fine sand in texture, saline, moderately alkaline and strongly calcareous.

Water Quality

Irrigation water was analyzed for standard water quality parameters (water salinity, residual sodium carbonates-RSC, and sodium adsorption ratio-SAR). The importance of these parameters in relation to water quality for irrigated agriculture is discussed in detail by Shahid (2004). Water salinity refers to the total concentration of dissolved salts-salinity hazard. Sodicity-relative proportion of sodium cations to other cations particularly Ca and Mg i.e., SAR (SAR = $Na/[(Ca+Mg)/2]^{0.5}$) expressed as $(mmoles/l)^{0.5}$, where all concentrations are in meq/l. The high SAR deteriorates soil structure and reduces water penetration into and through soil. Similar to drought and salinity, excess proportion of sodium, in comparison to calcium and magnesium, reduce water availability to the crops. Residual Sodium Carbonates (RSC) – bicarbonate anion and carbonate anion concentration as related with calcium (Ca^{2+}) and magnesium (Mg^{2+}) cations [RSC = $(CO3^{2-} + HCO3^{-}) - (Ca^{2+} + Mg^{2+})$] where all concentrations are in meq/l.

Table 1: Physical and Chemical Characteristics of Soil (0-30 cm)

Physical Characteristics	
Gravels (2-5 mm)	<0.5%
Very coarse sand (2-1 mm)	3%
Coarse sand (1 – 0.5 mm)	3%
Medium sand (0.5 - 0.25 mm)	4%
Fine sand (0.25 – 0.1 mm)	51%
Very fine sand (0.1-0.05 mm)	37%
Coarse silt (0.05 – 0.02 mm)	0.5%
Fine silt (0.02 – 0.002 mm)	0.5%
Clay (<0.002 mm)	1.0%
Total Sand (2-0.05 mm)	98%
Total silt (0.05-0.002 mm)	1.0%
Total clay (<0.002 mm)	1.0%
Textural Class	Fine sand
Saturation Percentage	26%
Water retention at 15 bar (W15)	6.5%
Available Water Capacity (AWC)	4.13%
Chemical Characteristics	
Electrical conductivity of saturation extract (ECe)	12.3 dS/m
pHs	8.22-moderately alkaline
CaCO3 (equivalents)	53%

Table 2: Water quality at ICBA

Water Quality	
Water salinity	EC = 30 dS/m
Water Conductivity Class	C4 (very high salinity water)
Residual Sodium Carbonates	Nil
Sodium adsorption Ratio	31 $(mmoles/l)^{0.5}$
Water Sodicity Class	S4 (very high sodium water)
Water Class (Richards, 1954)	C4S4

Water Salinity and Sodicity Class – C4S4

C4 water is not suitable for irrigation under ordinary conditions, but may be used occasionally under very special circumstances. The soils must be permeable, drainage must be adequate, irrigation water must be applied in excess to provide considerable leaching, and highly salt-tolerant crops should be selected. S4 class is generally unsatisfactory for irrigation purposes except at low and perhaps medium salinity, where the solution of calcium from the soil or use of gypsum or other amendments may make the use of these waters feasible.

Experimental setup

Two field experiments were conducted at the ICBA station; 1) effect of emitter spacing on the formation of salt contours; 2) sprinkler overlapping and distribution to maximize leaching.

Experiment 1: Three emitter spacings (25 cm, 50 cm and 50 cm) were tested in this experiment. Each drip line was 2 meters apart (Fig. 1) and triplicated. To avoid the effects of replications, each replication was 3 meter apart.

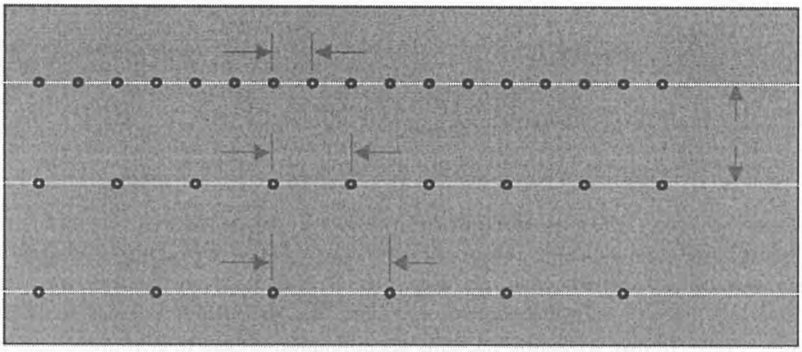

Figure 1: Experimental set up and emitter spacing

The initial soil salinity level was 12.3 dS/m. By mixing saline and fresh water a salinity level of 19 dS/m was achieved, and irrigation was applied through these driplines twice a day, 30 minutes in the morning and 30 minutes in the evening over a period of 90 consecutive days. Soil samples were collected at 0-20 cm depth for salinity measurement. The LR was used to determine salinity development with the effect of different emitter spacing. The LR was determined by this (ECe/ECw) relationship, where ECe is the electrical conductivity of the soil saturation extract and ECw the electrical conductivity of the irrigation water. The LR in relation to emitter spacing is presented in the form of salt contours around the emitters.

Experiment 2: In this experiment the performance of micro and impact sprinklers was tested by single jet and through overlapping. The objective was to maximize uniformity and to minimize evaporation. The water was collected in catch cans (placed at a distance of 0.5 m) and evaporation along the jet line was calculated based on salinity variations in the collected waters (Fig. 2).

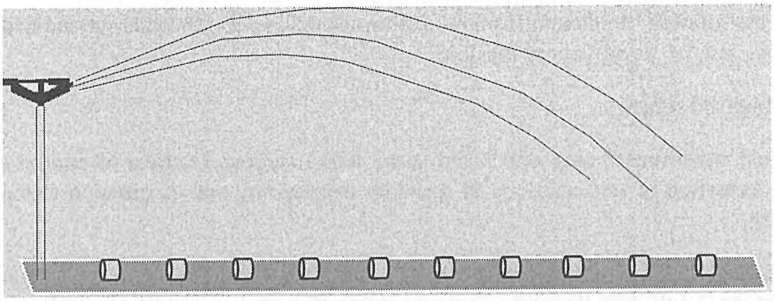

Figure 2: Setup for measurement of water evaporation by correlation with collected water salinity

The overlapping in micro and impact sprinkler was maintained at 90% during the course of the experiment. Soil salinity developed through sprinkler irrigation was evaluated by analyzing soil samples.

Effect of Emitter spacing on soil salinity development – trickle irrigation

In this experiment the performance of emitter spacing was tested in terms of LR of the soil. Soil salinity values obtained from the saturated soil pastes were used to prepare the salinity contour lines. The results are presented in Figures 4-6. These figures clearly show that the higher LR and salinity is recorded almost in the center of the emitters (Chhabra, 1986). In drip irrigation the salts accumulation occurs in two processes: in the first process, the soil becomes saturated and water and solutes spread in various directions saturating the neighboring voids and moving further (Fig. 3); in the second process which occurs between consecutive irrigation cycles, evaporation of water and uptake of water and nutrients by plants occur and the solutes are redistributed in the soil, the final buildup of salts in the soil results from the interaction of these two processes throughout the crop season.

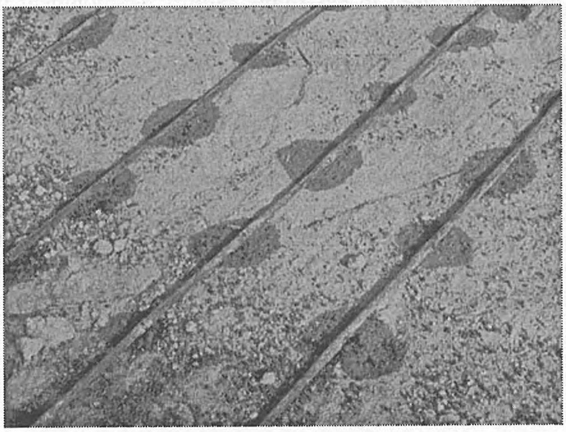

Figure 3: Initial stage of soil saturation with drip irrigation

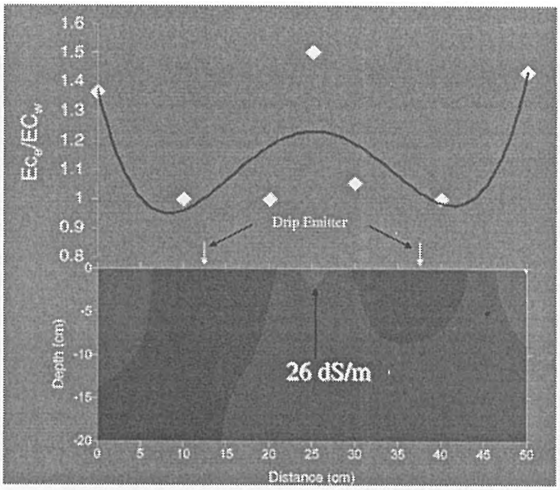

Figure 4: Soil salinity contours (leaching ratio) with respect to 25 cm spacing

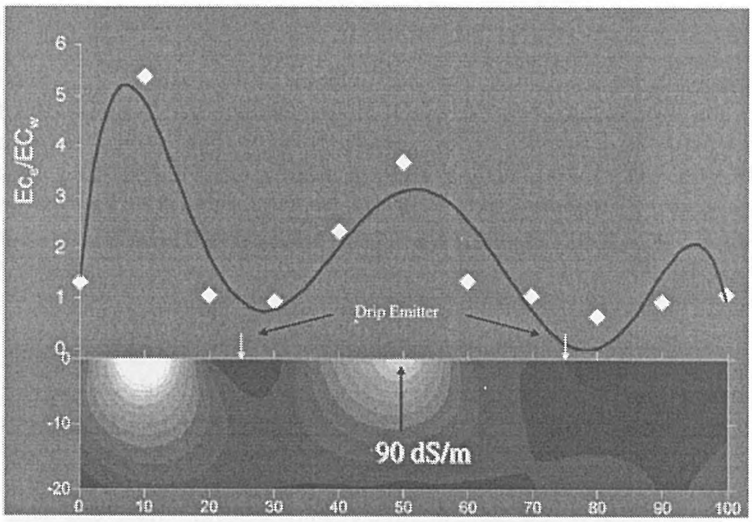

Figure 5: Soil salinity contours (leaching ratio) with respect to 50 spacing 50

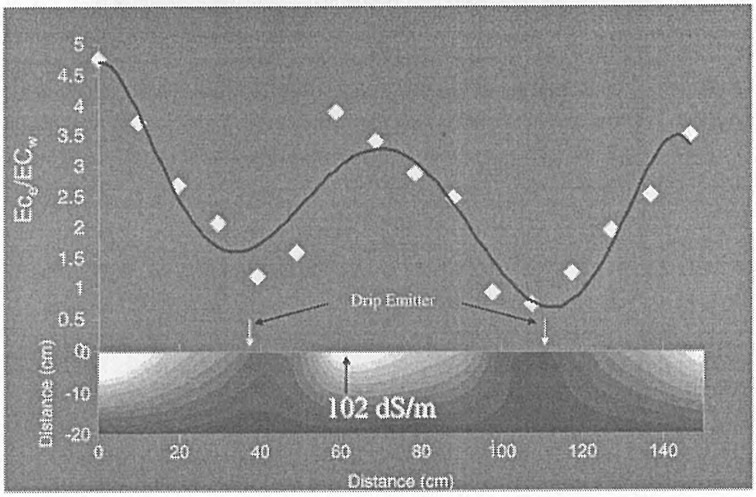

Figure 6: Soil salinity contours (leaching ratio) at a drip spacing of 75 cm.

The Figures 4-6 clearly illustrate that the minimum LR/salinity is developed near the emitters and increases towards the center of two emitters. It is very evident from the Figures 4-6 that by decreasing the emitter spacing from 75 to 25 cm, the LR as well as soil salinity is reduced significantly. The soil salinity with 75 cm emitter spacing was recorded as 102 dS/m compared with 25 cm emitter spacing where a relatively lower ECe (29 dS/m) was recorded. There was only a slight difference in LR and ECe with 50 and 75 cm emitter spacing. The effects of emitters spacing on soil salinity contours (top view) can be seen at a glance in Fig. 7.

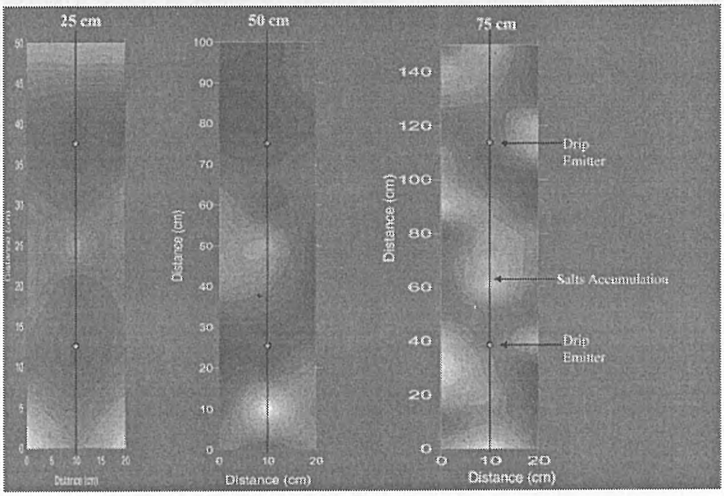

Figure 7: Comparative salinity contours in three emitter spacing – top view

Micro and impact sprinkler– single jet

In this experiment the performance of micro and impact sprinkler was evaluated in terms of water evaporation, which was calculated from the volume of water collected in catch cans and their salinity levels. The catch cans were placed in the experimental sites at a uniform distance of 0.5 m meter (Fig. 8). The results of water evaporation and water salinity development are presented in Figs 9-10.

Figure 7: A view of experimental layout

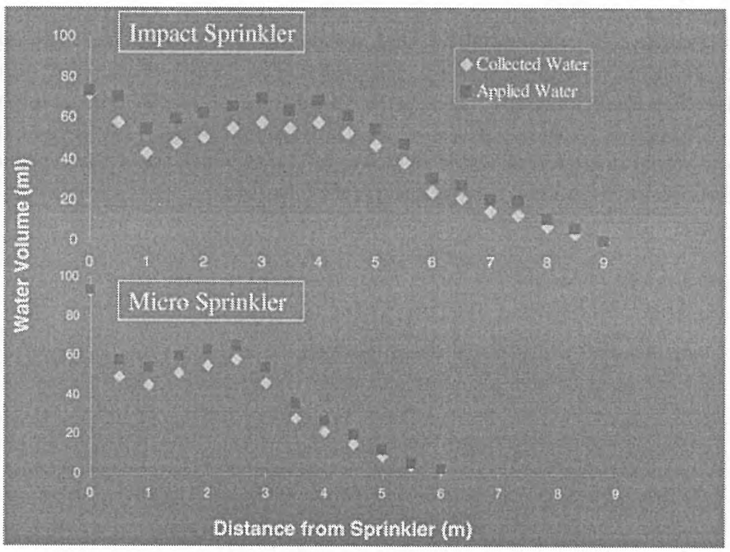

Figure 8. A view of experimental layout

Fig. 9 shows that the impact sprinkler performed better than the microsprinkler in reducing water evaporation. Similarly lower water salinity was recorded in the water collected in the impact sprinker compared to the water collected in the micro sprinkler. This experiment was conducted only for two hours, prolonged irrigation may cause significant salinity consequences in the soil. Therefore, a very careful irrigation management using saline water is needed.

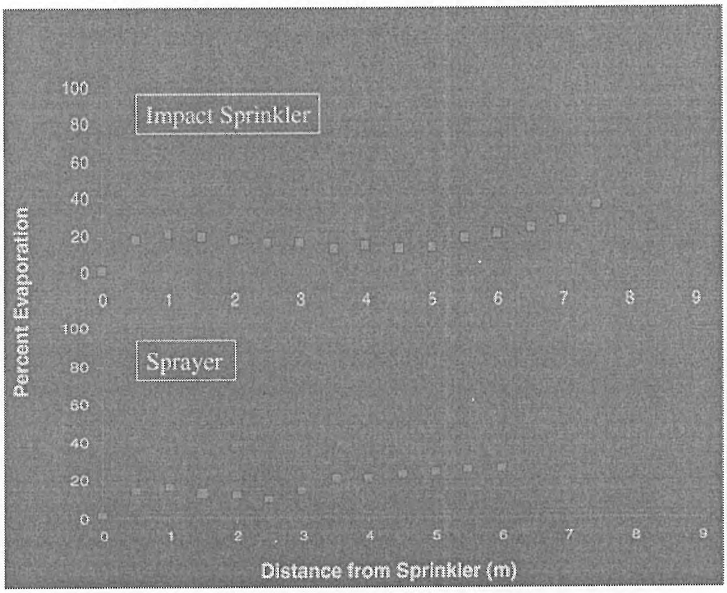

Figure 9: Comparison of impact ad micro sprinkler on water evaporation

Overlapping in Micro Sprinkler

The performance of a micro sprinkler as a single jet, and with an overlapping of 90% was tested in terms of water salinity development. The results are presented in Fig. 10. The comparison clearly shows higher water salinity development with a single jet at a distance of 6 meters; whereas an EC of 24 dS/m was recorded with 90% overlapping. This development is related to water evaporation, the distance the water particle travels, mixing through overlapping in addition to other factors.

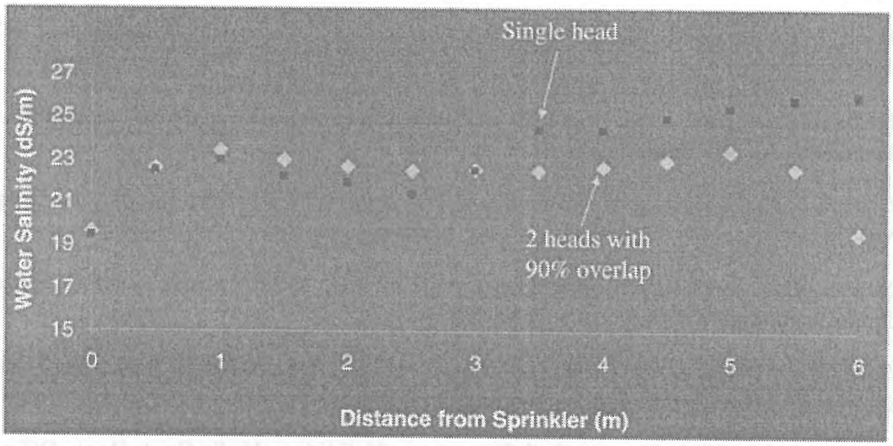

Figure 10: Salinity reduction through overlap of two consecutive sprinkler heads at 90 %

In another experiment water and soil salinity was tested in sprinkler irrigation. In this experiment wind has significantly affected the water distribution and the subsequent salinity development. The trend of salinity development is shown in Fig. 11. Since there was no control over the wind, it is concluded that in areas where wind can play an important role, windbreakers can offer a solution to the improvement in water distribution and salinity development. The blue area shows higher salinity due to uneven distribution of water through wind effect.

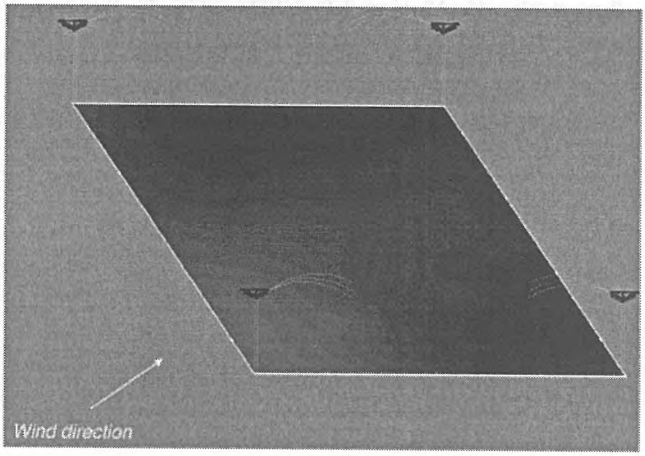

Figure 11: Salinity development through sprinkler irrigation

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Use of clay deposits in water management of calcareous sandy soils under surface and subsurface drip irrigation

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USE OF CLAY DEPOSITS IN WATER MANAGEMENT OF CALCAREOUS SANDY SOILS UNDER SURFACE AND SUBSURFACE DRIP IRRIGATION

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ABSTRACT

Water research studies in Saudi Arabia clearly show severe depletion of groundwater. Since agricultural activities account for more than 85% of the total water consumed a scientific-based applied research program related to water saving and conservation in agriculture is essential. The purpose of this study was to investigate the effect of irrigation (levels & methods) and type of clay deposits on lettuce yield, water use efficiency (WUE) and the distributions of soil moisture and salts in the root zone of sandy calcareous soils. A field experiment was conducted at the college experimental station in 2002-2003. It consists of three clay deposits, three rates (0, 1.0, and 2.0%), and four total irrigation applied water levels, 360 mm (T1), 520 mm (T2), 635 mm (T3), and 822 mm (T4), using surface and subsurface drip irrigation. Results indicated that yield was significantly increased with the increase of irrigation level, whereas WUE significantly decreased with increase of irrigation level. The average yield increased by 9.30% in a high irrigation level compared to a moderate irrigation level, and decreased by 14.2% at the more stressed irrigation level. WUE decreased by 49.0% at a moderate irrigation level and decreased by 55% at a high irrigation level. Types of clay deposits did not affect the yield; however, the yield was significantly affected by amendment rates. The differences between surface and subsurface drip on yields and WUE were also significant. Results indicated that the moisture content of the subsurface treated layer increased dramatically, while salts were accumulated at the surface and away from the emitters in subsurface drip irrigation. The advantages of surface drip irrigation were related to the relative decrease in salt accumulation in the root zone area where the plant roots were active and the water content was relatively high.

Keywords: Drip irrigation, clay deposit, lettuce yield, sandy soils

Introduction

The sustainable use of scarce water resources in Saudi Arabia is a priority for agricultural development. The pressure of using water in the agriculture sector is increasing to create ways to improve water use efficiency and take full advantage of the available water. Therefore, adoption of modern irrigation techniques must be emphasized to increase water use efficiency. Drip irrigation is the most effective way to directly convey water and nutrients to plants; it not only saves water but also increases the yield of vegetable crops (Tiwari et al., 1998a, b; Tiwari et al., 2003). Bryla et al. (2003) reported that drip irrigation improved production and water use efficiency of faba beans in California using different levels of irrigation based on the percentage of evapotranspiration. Ayars et al. (2001) reported in their studies on subsurface drip and furrow irrigation in the presence of shallow saline groundwater that the yield of the drip irrigated cotton improved during the 3-year study, while that of furrow irrigated cotton remained constant. Also, tomato yields were greater under drip irrigation than under furrow irrigation in the same study from the first year. Lamm and Trooien (2003) reported that a successful application of subsurface drip irrigation for 10 years in Kansas, USA reduced the irrigation water use for corn by 35 – 55% compared with more traditional forms of irrigation. The purpose of this study was to investigate the influence of irrigation levels and surface and subsurface drip irrigation on lettuce yield, water use efficiency, water and salt distributions in irrigated sandy calcareous soils amended with different rates of natural clay deposits.

Materials and Methods

A field experiment was conducted at the College of Agricultural Research Station at Dirab, (24ᵒ%25? N, 46ᵒ%34 E), 40 km southwest of Riyadh, Saudi Arabia. Selected properties of the soil and irrigation water were determined by the standard procedure (Page et al., 1982). The soils are non-saline, calcareous ($CaCO_3$ ranges from 269 to 353 gkg-1 soil) and sandy in texture, while irrigation water has high salt content (TDS = 3300 mg/l) and moderate alkalinity (SAR=7.69). Natural clay deposits were collected from different regions in Saudi Arabia e.g. the western (Khulays) and central regions (Dhruma and Rawdat areas). Deposit samples were prepared by grinding and sieving through a 2mm sieve, and then analyzed for their physical and chemical properties (Table 1). The three amendments (Khulays, Dhruma and Rawdat) were applied in each row as a subsurface thin layer at a depth of 15 - 20 cm and at rates of 1 and 2% of the soil. The experiment included surface (S) and sub-surface (SS) drip irrigation, with four irrigation water levels applied 360 mm (T1), 520 mm (T2), 635 mm (T3), and 822 mm (T4) for the whole season. The 30 m x 30 m field plot was divided into four equal subplots for the irrigation levels (T1, T2, T3, and T4). Surface and subsurface drip irrigations were installed in each half of the subplots, respectively. Drip tubing (GR type, 16 mm diameter) with 40 cm emitter spacing built in (delivering 4 Lhr^{-1}) was used in the surface and the subsurface drip irrigation treatments. The experiment was laid out following the complete randomized block design with three replicates for each treatment. Each treatment consists of 7 drippers (2.8 m tubing) and the distance between every two rows was about 1 m. Lettuce (Lactuca Sativa cv. Parris Island) seeds were sown in Jeffy-7 pots on 29 September, 2002, kept in a green house until 17 of December, and then transplanted to the field with three seedlings at each dripper. Irrigation by the surface drip system was commenced after transplanting in all treatments for the establishment. Then, surface and/or subsurface drip irrigation was continued every other day till the end of the experiment. The total amounts of fertilizer are 200 kg ha^{-1} N, 150 kg ha^{-1} P_2O_5, and 120 kg ha^{-1} K_20. Uniform fertilization was used to deliver the fertilizer requirements using (N, P, K) liquid fertilizer in all treatments. Nine soil samples were collected before irrigation from the root zone area on a grid base (15 cm apart) around the dripper at three growth stages namely vegetative stage, flowering stage and at the end of the season. Samples were collected from the lower and higher amendment rate treatments and then water contents were determined by the gravimetric method after oven drying at 105°C. Salt distributions were assessed by measuring EC in 1:1, soil to water extract, then three dimension figures for water and salt distributions in the root zone area were introduced using MATLAB software for the collected soil samples. Heads of lettuce were picked at the end of the season (March 22, 2003), weighed, and the total yield was determined.

Results & Discussions

Data show that differences due to water regime, surface and subsurface drip irrigation and the interactions between water regime and irrigation methods were highly significant (at 1% level) for both yield and WUE. Differences in WUE and yields due to amendment rates and the interactions between amendment rates and water regime or irrigation methods were also significant (at 1% or 5% levels) whereas the interaction between amendment types and rates was not significant. Data also showed that differences due to amendment types and the interaction between water regime and amendments or the irrigation methods and amendments were not significant. These results reflect the positive effect of water regimes, surface and subsurface drip irrigation and amendment rates on lettuce yield and WUE. The results are further elaborated in order to evaluate the effect of each treatment on lettuce yield and WUE. Effect of amendment types, irrigation regimes, irrigation methods and the amendment rates on lettuce yield and WUE are presented in Table 2. It indicated that at high irrigation levels (non-stressed T4 and T3 treatments), yield were high and decreased significantly at low irrigation levels (stressed, T2 and T1 treatments). The average yield increased about 12.8% in the T2 treatment when compared with T1 treatment, whereas average yield decreased in the T2 and T1 treatments by about 2.8 and 16.6%, respectively, compared to non stressed treatment T4. WUE decreased with non-stressed treatment by 35.4, 49.8 and 55.4% at T2, T3 and T4 compared to the stressed treatment T1. The drastic reduction in WUE could be due to the increase of the amount of water used and the possible accumulation of salts in the root zone area as a result of using irrigation water with quite high salinity (3300 TDS) without proper leaching which results in lower yield from T1.

The results showed that amendment types significantly affected yield when compared with control but the differences between the studied amendments were not significant. The yield increase was 12.6%, 10.75% and 5.6% for Rawdat, Dhruma and Khulays, respectively when compared with the control. The differences could be due to the differences in the clay deposit characteristics and to the differences in $CaCO_3$ content, ECe, CEC and the dominant clay minerals. Khulays deposit showed some desired characteristics such as low $CaCO_3$, high CEC and the dominance of smectite clays, whereas it has relatively high original salinity which could be leached out of the root zone area before cultivation. The average yield was increased by 8.9% and 20.1% at 1 and 2% amendment rates when compared with control. Such increase in yield could be due to the improvement of sandy soil characteristics particularly the available water content and nutrient status. Differences in lettuce yield due to irrigation methods were significant and the yield increase due to surface drip irrigation was about 43.4% over the subsurface drip irrigation. Also, WUE was significantly higher with the surface drip irrigation compared with the subsurface drip. It appears that surface drip irrigation created more suitable conditions in the root zone area for plant growth and productions.

Data of salt distribution (Figure 1) in the root zone area for all indicate that salt distribution shows specific distribution patterns in the amended soil when compared with non-amended soil (control) in both surface and subsurface drip irrigations. Such distribution patterns depend on the type of amendment, and amendment rates in the subsurface treatment. In non-amended soil, water content (not shown) was generally low (about 2-1%) on the surface and increased gradually with depth without a clear distribution trend (5-7%). There was no clear difference between surface and subsurface drip irrigation in non-amended soil where the soil profile was not modified. This trend could be due to water evaporation from the surface and hence decrease water content in the surface layer and the gradual increase with depth that related to the capillarity of the soil of the control treatment. The T1 treatment showed relatively high water content below 30 cm depth indicating deep percolation and partial losses of water below the root zone. This trend was not clear in T3 and/or T4 treatments. The amended soil water content was quite high either for the surface or the subsurface drip irrigation treatment particularly in the amended subsurface layer (Pw = 10-12% in the soil treated with Khulays clays).

Conclusion

It was clear that water seems to be stored in the treated layer with no or little percolation below 30 cm depth. The surface layer of the subsurface drip treatment was relatively dry and it seems to be uniform in dryness compared with the surface irrigation where dryness seems to be on the sides. Therefore, applications of clay deposits to sandy soils modifies the distribution of the water content in the root zone area where water could be retained by clays applied to the subsurface layer. The desired characteristics of clay deposits may be reflected on the improvement of soil texture, structure, swelling, increasing cation exchange capacity and soil water retention, hence improved soil water contents in the squash root zone.

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Table 1: Some physical, chemical, mineralogical and fertility characteristics of the clay deposits used in the experiment.

	Dhruma	Khulays	Rawdat
	Physical and Chemical properties		
Sp %	63.0	53.0	74.0
EC_e dSm^{-1}	7.15	22.0	3.35
pH soil paste	7.97	7.25	7.59
SAR	16.10	2.30	0.86
$CaCO_3$ gKg^{-1}	30.0	30.0	420.0
O.M. gKg^{-1}	19.3	29.0	88.7
CEC Cmol Kg^{-1}	29.8	39.6	21.6
Clay %	60.0	60.0	59.0*
Silt %	12.0	36.0	40.0
Sand %	28.0	4.0	1.0
Texture	Clay	Clay	Clay
	Clay Mineralogy		
Smectite	++	++++	++++
Kaolinite	++++	++	+
Vermiculite	-	+	+
Accessory Minerals	Q	Q,F	Q
	Fertility Status		
N			
P $mgKg^{-1}$	2.9	2.0	21.4
Fe $mgKg^{-1}$	14.87	12.92	155.0
Zn $mgKg^{-1}$	0.97	1.30	1.97
Mn $mgKg^{-1}$	2.65	2.17	43.3
Cu $mgKg^{-1}$	0.84	1.35	2.98

* after the removal of $CaCO_3$

SAR Sodium Adsorption Ratio , CEC Cation Exchange Capacity.

++++ High , ++ Medium , + Low , Q quartz , F Feldspars.

Table 2: Effect of clay deposits (type and rates), irrigation regimes and irrigation methods on Lettuce yield (ton ha^{-1}) and WUE (kg m^{-3}).

Treatments	**Yield (ton ha^{-1})**	**WUE (kg m^{-3})**
	Effect of clay deposits type	
Dhruma	7.52	9.68
Khulays	7.17	9.96
Rawdat	7.65	9.32
LSD $_{0.05}$	n.s.	n.s.
	Effect of irrigation water regimes	
T1	6.82 C	14.88 A
T2	7.79 AB	9.60 B
T3	7.24 BC	7.52 C
T4	7.95 A	6.64 C
LSD $_{0.05}$	0.66	0.88
	Effect of irrigation methods	
Surface drip	9.49 A	12.32 A
Subsurface drip	5.40 B	7.00 B
LSD $_{0.05}$	0.46	0.60
	Effect of amendment rates	
Control	6.79 C	8.52 C
1%	7.39 B	9.64 B
2%	8.16 A	10.8 A
LSD $_{0.05}$	0.57	0.56

*The same letter in each column represents no significant difference at 5% level.

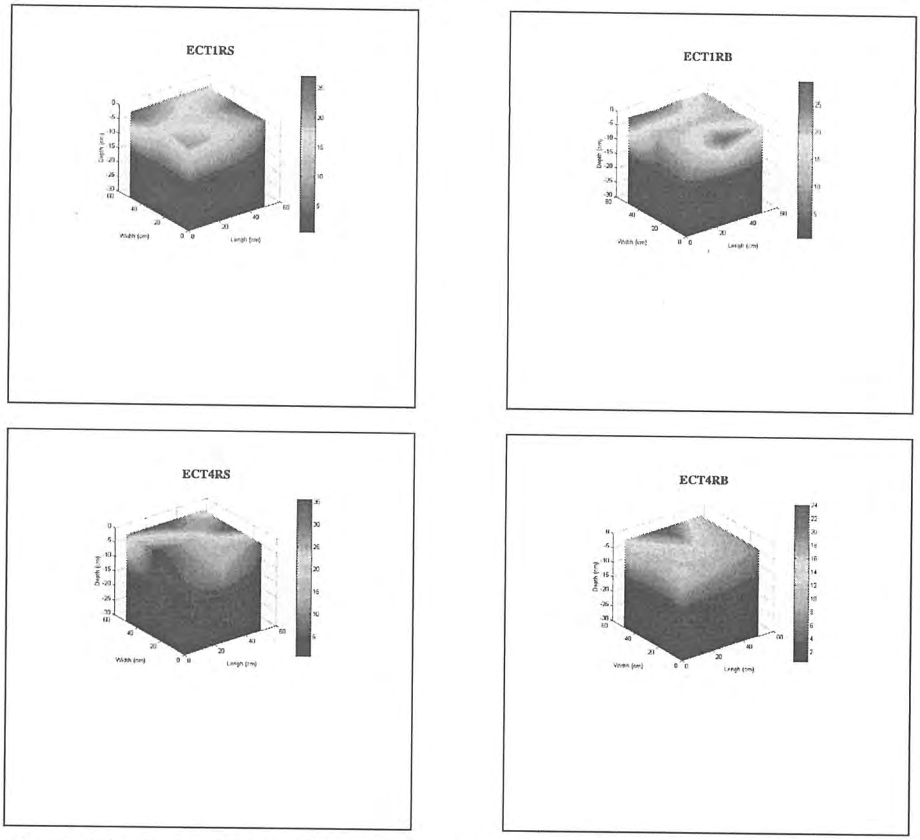

Fig. 1: Salt distributions (EC dS/m) in the root zone area in surface and subsurface drip irrigation as affected by amendments.

Modeling of rainwater harvesting in semiarid areas

M.H. Rubarenzya, G. Wyseure, J. Berlamont, J. Feyen

MODELING OF RAINWATER HARVESTING IN SEMIARID AREAS

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ABSTRACT

This paper presents the findings on a study analyzed and designed for Rainwater Harvesting (RWH) in a semiarid region using the PARCHED-THIRST model. The model is a decision-support tool for farmers and provides advise on the most suitable RWH system, including planting dates for a given crop and season. The feasibility of RWH for the cultivation of maize in the Katakwi District, Uganda was investigated. The biannual growth of a maize crop was modeled as part of the design and optimization of a RWH system. The results show the importance of utilizing open-air runoff areas to harvest rainwater. On average, the yield in the first season was greater than in the second by 0.32 t/ha for the same maize variety. Therefore, if maize must be grown in only one season, the first growing season should be selected. The soil profile has the crucial task of storing as much water as possible for as long as possible. The runoff generating capacity of the catchment is important. The crops need to sustain short-term flooding and longer-term droughts. The lack of control over the runoff, soil water storage and water schedule for the crops requires a good knowledge of these different processes. The potential erosion risk associated with very large area ratios is highlighted. Increased area ratios do not necessarily translate into increased yields. Thus, there is a need to optimize the system so as to obtain the maximum dividends per given amount of land. Optimum and sustainable growth of maize requires both careful selection of crop variety, and use of moisture conservation techniques. The study recommends that research be carried out into new ways of remotely gathering meteorological data, such as through satellite imagery. This information can then enable the development of more reliable decision support tools.

Keywords: Decision support tool, Optimization, PARCHED-THIRST, Rainwater harvesting

1. Introduction

Water is essential for all life and is used in many different ways – for food production, drinking and domestic uses, and industrial use. It is also part of the larger ecosystem on which biodiversity depends. Precipitation, converted to soil and groundwater and thus accessible to vegetation and people, is the dominant precondition for biomass production and social development in dry lands. The amount of available water is equivalent to the water moving through the landscape. It also fluctuates between the wet and dry periods. However, water is becoming scarce not only in arid and drought-prone areas, but also in regions where rainfall is abundant: water scarcity concerns the quantity of resource available and the quality of the water because degraded water resources become unavailable for more stringent requirements (Pereira, et al., 2002). In the case of the crop, lack of water is caused by low water storage capacity of the soils, low infiltration capacity, large inter-annual and annual fluctuations of precipitation, and high evaporative demand.

Over the past 20 years, parts of Uganda have become increasingly semiarid and prone to drought. This is a naturally occurring situation in which there exist temporary imbalances in water availability, consisting of persistent lower than average precipitation, of uncertain frequency, duration and severity, the occurrence of which is difficult to predict. This has resulted in crop failure, most notably the maize crop. Irrigation could be considered as an option. The irrigation potential that exists in South and East African countries exceeds the presently irrigated area. However, irrigation in Africa is a privilege since the cost of providing irrigation to one hectare could be as high as US$ 20,000/ha, which is not affordable (Sivanapan, 1997). Thus, an approach to overcome the emerging threat of droughts is to develop and adopt a variety of soil moisture and water conservation technologies that reduce the cost of irrigation, and promote sustainable small-scale irrigation on a watershed basis. RWH as a dryland cropping system could be adopted instead. However, a lack of technical knowledge has been identified as one of the primary factors preventing the widespread adoption of RWH in the region (Rwehumbiza, et. al., 1998; Young, et. al., 2002).

Rain and storm water harvesting techniques for agriculture are not entirely new and were extensively practiced throughout a vast region of North America, through the mountains and basins of the Mexican northwest, and through the civilizations that flourished in the south central highlands of Mexico. These techniques were also practiced throughout the Middle East, North Africa, China, and ancient India (UNEP, 1983).

1.1 Literature review

The Katakwi district is located in north-eastern Uganda, and covers an area of 5114km^2. Generally a plateau with gently undulating slopes, a few areas of the district lie at about 1,036m - 1,127m above sea level. It has a climate that is characterized by two seasons. The wet season runs from March to October, the dry season from November to the end of February. The mean annual rainfall varies from 1000mm to 1500mm. The rainy season has a principal peak due around March-June and a minor peak around August-October. Recently, however, rainfall has been unreliable and unpredictable, with the December-February period being the driest. The average annual temperature is in the range of 23°C to 25°C. The warmest period is January through March, and the coolest period is June through September. The soils are mainly of sandy sediments and sandy loam. The bottomland contains widespread deposits of alluvium. Though not very fertile, the land is productive with Amuria County to the west being the most fertile. Maize is the staple crop in this region, serving both as a food crop and a cash crop. However, maize prices are generally low and crop failures frequent, which contribute to the impoverished state of many people in the area. In addition, the unreliable rains have been a contributing factor to recent droughts and crop failures in the area, as occurred in 2003.

1.2 Rainwater Harvesting (RWH) and it's modeling

Water harvesting in general is the collection of runoff and its use for the irrigation of crops, pastures, and trees, and for domestic and livestock use (Finkel et al., 1986). Since this definition could cover almost all fields of water resources development, it must be added that the term "water harvesting" is usually used for the development of marginal waters in arid or semiarid regions. Water harvesting may also be defined as the deliberate collection of rainwater from a surface (catchment) and it's storage to provide a supply of water (Oweis et al., 1999; UNEP, 1983). This process is distinct from the natural runoff of water into perennial rivers, which can be controlled and stored in reservoirs. There are many regions in the world where rainfall is heavy for some months of the year and light for the rest; rainwater and storm runoff, harvested in season and then stored, would help in alleviating the problem of water shortage during the dry season (see Figure 1).

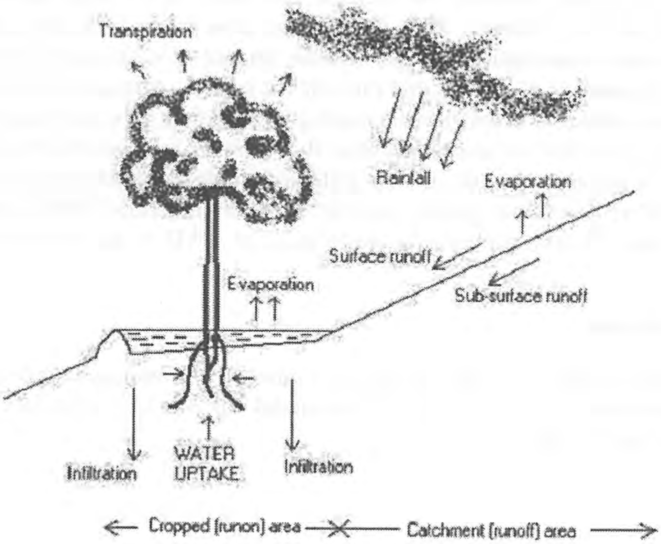

Figure 1: Model of a typical RWH system

A RWH system has the following characteristics: It is practised in arid and semiarid regions, where surface runoff often has an intermittent character; it is based on the utilization of runoff and requires a runoff producing and a runoff receiving area. Because of the intermittent nature of runoff events, storage is an integral part of the water harvesting system. Water may be stored directly in the soil profile or in small reservoirs, tanks, and aquifers. Water harvesting supports a flourishing agriculture in many dry areas, where rainfall is low and erratic in distribution.

The objective of the research study was to evaluate the potential of RWH for improvement in crop production of resource-poor farmers in Katakwi district. This involved the designing of a suitable RWH system, optimization of the system, and evaluation of the potential improvements in crop production.

2. Materials and Methods

2.1 The PARCHED-THIRST model

The Predicting Arable Resource Capture in Hostile Environments during the Harvesting of Incident Rainfall in the Semi-arid Tropics (PARCHED-THIRST) model is a process-based model, which simulates the key processes influencing the performance of RWH systems. It can also be applied to a range of other water conservation techniques through manipulation of its parameters (SWMRG, 2003; Young et al., 2002). The model is designed to assist agricultural planners and advisers whose aim is to improve cropping systems in dry-land environments, where the major factor limiting crop performance is the imbalance between water supply and demand (Growing et al., 2001).

In arid and semi-arid climates the major factor limiting the growth of crops in most years is the imbalance between the demand for water by the atmosphere and the supply from the soil. There is little that can be done to decrease the atmospheric demand but water conservation can increase the amount of water stored in the soil by reducing the proportion of rainfall that runs off the surface. Alternatively, water can be harvested from adjacent areas thus increasing the amount of water available to the plant (RWH). In order to assess whether this type of intervention by farmers is worthwhile, the benefits in terms of crop yield need to be determined over a number of years with different seasonal rainfall patterns. While PARCHED-THIRST can be used to simulate rainfed crop growth, the simulation of RWH is an integral part of its functionality.

2.2 Model overview

The PARCHED-THIRST model comprises a number of component sub-models that are linked together as shown in Figure 2. The model is driven by daily rainfall and other agro meteorological data.

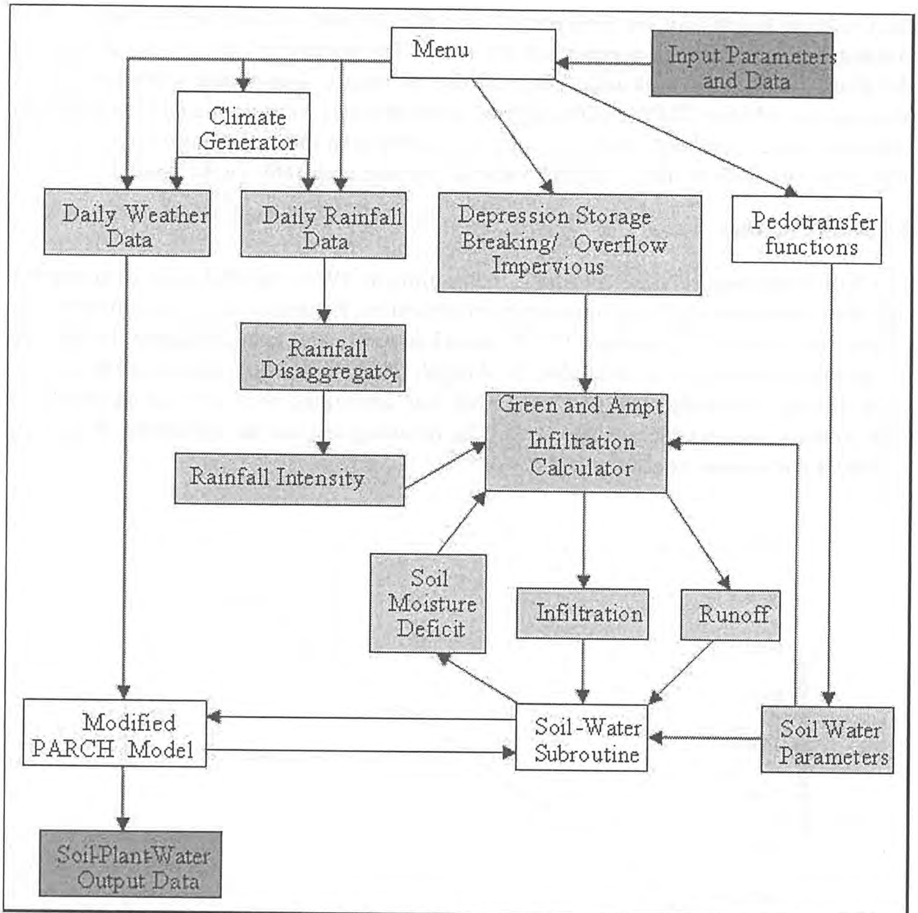

Figure 2: The PARCHED-THIRST model showing interactions between sub-models

There are three main components to the system, namely the climate module, the soils module, and the vegetation module. Agro meteorological data input into the PARCHED-THIRST model included: rainfall (mm), pan evaporation (mm), maximum temperature (°C), minimum temperature (°C), saturated vapour pressure (KPa), radiation (MJ/m³), relative humidity (%), and wind speed (km/d).

2.3 Design of scenario analysis

The Area ratio (r) is defined as the ratio of the catchment area (A) to the cropped area (a) (Oweis et al., 1999). The catchment area is also known as the runoff area, and the cropped area as the run-on area. It is the most important output in the design of RWH systems, integrating the effects of runoff coefficient, rainfall characteristics, soil, and crop factors. This value is typically less than 10, but for macro-catchment RWH systems, this ratio may be in the order of hundreds (Prinz, 1994).

Five different scenarios were simulated for the area ratio, r, which included 1:1, 2:1, 3:1, 5:1, and 10:1. Simulations were made for an existing biannual growing pattern, with the first season beginning on February 1 and the second season on October 1. The simulations were run for a period of 15 years. The parameters being varied included the plant variety, the area ratio, and the use of bunds. Two maize varieties (named Maize.cul and Maize TMV1) with different characteristics were simulated. The following scenarios were simulated: constant variety, varying area ratio; varying variety, varying area ratio (without bunds); varying variety, varying area ratio (with bunds).

2.4 Model inputs

The following inputs were available beginning in 1971: rainfall, pan evaporation, sunshine, minimum and maximum daily temperatures. Estimates for the remaining input parameters were both generated by the model and obtained from literature. In the case of rainfall, some data was available for Arupai, which is another station located in the same district. A double mass plot of rainfall was performed over a 6-year period when both stations had recordings (Figure 3). The resulting test for the reliability of the data yielded a correlation coefficient of 0.998.

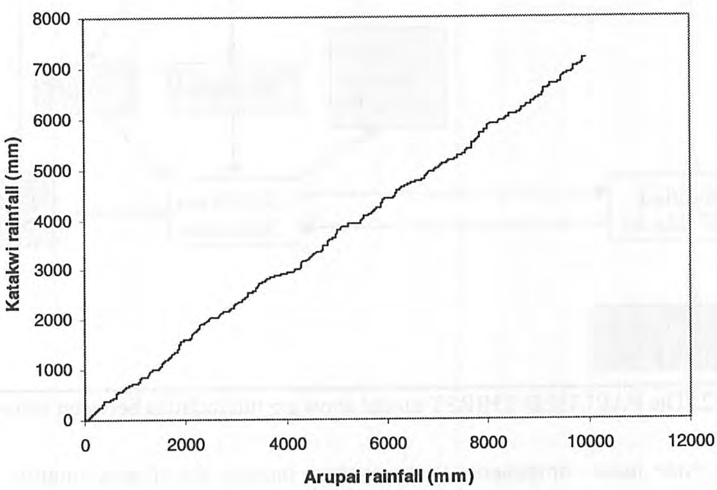

Figure 3: Double mass plot for Katakwi and Arupai stations

2.5 Data recovery rate (%)

The data recovery rate (DRR) which is the percentage of records collected (n) at a station as a percentage of the total number of records possible (N) over the given time interval was determined for various data over the period 1971-2000.

$$DRR = \frac{n}{N} * 100\% \tag{1}$$

Table 1: DRR for simulation data

Data	DRR (%)
Rainfall	94
Pan evaporation	3
Sunshine hours	25
Max. Temperature	49
Min. Temperature	49

3. Results

The annual total maize yields were observed to vary with each year of simulation, with values between 0.8 to 1.8 tons/ha. In addition, above a threshold area ratio, the annual total maize yields were not much dependent on the choice of area ratio for a given RWH system. It was further observed that significant differences of yield occur between the planting seasons of February and October. These were in the order of -0.3 to 0.6 tons/ha (see Figure 4), and followed similar trends irrespective of area ratio (scenario) simulated.

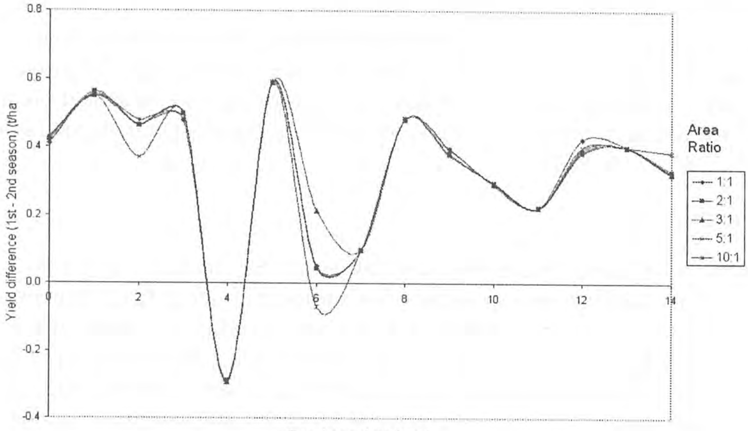

Figure 4: Yield difference between seasons with same area ratio for 5 simulated scenarios of Maize.cul

Figure 5 represents the Water Use Efficiency (WUE) of different scenarios. The groups of four columns represent Maize.cul, Maize.cul planted with bunds (30cm high), Maize.TMV1, Maize.TMV1and planted with bunds (30cm high), respectively. It was observed that for the four planting scenarios, the WUE increased with increasing area ratio, reached a peak, and then begun to drop. The initial increase would be due to the evapotranspiration demand of the crop being met resulting in greater production of plant biomass. The decreasing tail would be the result of excess water, water logging, and plant anaerobiosis. The latter was seen in an increase in surface runoff, which increased from 80mm to 500mm as the area ratio increased from 1:1 to 10:1.

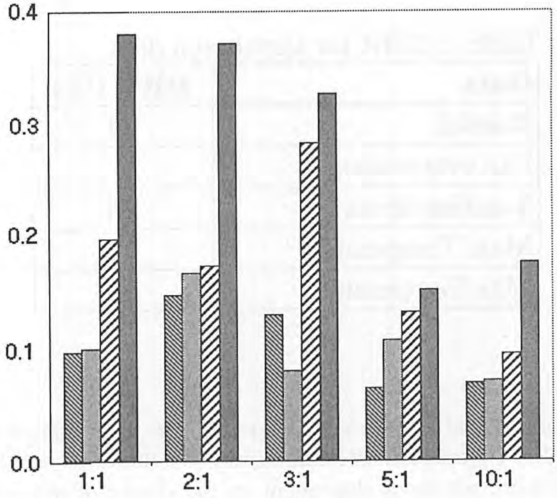

Figure 5: WUE for the different area ratios

In half of the scenarios involving the two maize varieties, simulations were carried out for the case that 30cm-high bunds are built around the fields. The impact of these bunds on the output was very significant. The bunds resulted in decreased surface runoff from the fields; increase in the WUE, and to increased average annual yields. Maize.cul realized an average improvement in WUE of 10% over the various area ratios as a result of including bunds (see Figure 5). The corresponding improvement for Maize TMV1 was 64%. Maize.cul realized an average increase in annual yields of 4% over the various area ratios as a result of including bunds. The corresponding value for Maize TMV1 was 11%.

4. Discussion

The variations in annual yield that were observed for the same area ratios may be attributed to the differences in climatic conditions that occur from year-to-year and over the two growing seasons. Where crop stress occurred in the course of the growing season, the simulated yield dropped. It was observed that the annual yield difference observed between the first and the second growing season is in the order of 22% of the total annual yield for any given area ratio. Such a significant difference is relevant especially if a decision must be made to adopt only one growing season, for instance, if demanded under a crop-rotation system. These significant differences illustrate the point that, although RWH leads to more efficient use of rainfall by increasing soil moisture storage, it cannot resolve the temporal problems of moisture deficit in dry seasons. This system cannot harvest and avail rainfall in the dry season as there is none, yet this is when crops may need water the most. The result is crop stress and the accompanying decline in crop yields.

It was observed that the WUE for a given area ratio increased with the application of bunds (see Figure 5), and was dependent on the maize variety that was simulated. It attained a peak between the ratios 1:1 and 3:1, depending on the scenario. The 68% increase for Maize.cul (with bunds, 2:1 ratio) is the most prominent increase in efficiency with respect to the 1:1 scenario. Figure 4 shows that Maize TMV1 (with bunds) offers the most water efficient scenario. In all cases, an area ratio of between 1:1 and 3:1 yielded the best WUE. Large values of surface runoff were obtained at the higher area ratios, (especially for simulations without bunds). These raised concerns because of the adverse environmental impact such waters have.

Impact of bunds

A mean increase in average annual yields of up to 11% was simulated at smaller area ratios. The yield then reduced inversely with increasing area ratio, which may reflect the adverse effects of water excess. Such a relationship is important when maximizing the economic benefits from a RWH scheme. It is clear that there comes a point when the addition of water does not result in meaningful percentage increases in yield, eventually resulting in diminishing returns; and this while consuming more and more potential agricultural land. There was a mean increase in WUE of up to 64%. There was no runoff simulated in the presence of bunds. The latter is probably due to the fact that the bunds allow run-on and rainfall more time in the fields, which allows the water to fully infiltrate into the soils. This water may then satisfy the evapotranspiration demand, or remain in the soil, or percolate down.

Data availability

This research revealed that for effective functioning of the PARCHED-THIRST model, it is necessary to have at least some data on the whole spectrum on input parameters, after which the model can fill in missing data. This is consistent with work by Wyseure, et al. (2002). However, this presented a problem given that semi arid areas tend to have poor coverage of meteorological stations, and this was true for Katakwi.

5. Conclusions

RWH is necessary for the reliable cultivation of maize in the study area. Since maize is such an important food crop and cash crop for the people, the modernization of its cultivation will contribute enormously towards poverty alleviation. The model was able to show that on average, the yield in the first season was greater than in the second by 0.32 t/ha for the same maize variety. Therefore, if maize must be grown in only one season, the first growing season should be selected. Two maize varieties were simulated, Maize.cul and Maize TMV1, and the yields obtained from the former were about half those of the latter. This illustrates the point that not all crop varieties will do well at a given place. The PARCHED-THIRST model, through scenario simulation, provides an excellent way to determine how suitable a region and a RWH system are in advance of planting.

The results show that increased area ratios do not necessarily translate into increased yields. Thus, there is a need to optimize the system so as to obtain the maximum dividends per given amount of land. In addition, while the WUE was found to generally decrease with increasing area ratio, the runoff increased, and with it the possibility for environmental degradation of the fields through erosion. From an analysis of the simulated annual yield, WUE, and surface runoff, a design ratio of 1:1 is recommended. Bunds should be applied to the fields. This area ratio is at the lower limit, but gains in yield from the increase in planted area can then surpass the minimal gains that could have been obtained with a larger ratio. The study recommends that research be carried out into new ways of remotely gathering meteorological data, such as through satellite imagery. This information can then be gathered in areas with poor data recovery rate and enable the development of more reliable decision support tools.

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INNOVATIVE DESIGNS AND TECHNIQUES FOR IRRIGATION WATER SAVING

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ABSTRACT

Water scarcity is becoming an increasingly global issue. To tackle this issue on a macro level is expensive and time consuming, albeit unavoidable, while its conservation measures at micro level have been found to be relatively inexpensive, feasible, and workable. Micro resource conservation includes water saving in the irrigation systems (especially at tertiary level) by reducing farm irrigation application losses through improved irrigation practices for its efficient use for producing more crop per drop (Gill Mushtaq, 2004). Promotion of water resource conservation technologies at the farm level seems feasible and can greatly help to achieve sustainable irrigated agriculture under the acute water shortage scenario. Thus, there is an imperative need to develop and promote the adoptability of water resource conservation technologies for improving water productivity, lowering production costs, getting higher farm returns, and combating the water crisis.

Efforts made for developing water saving technologies at the Water Management Center, University of Agriculture, Faisalabad-Pakistan are presented in this paper. The efforts include (i) the development of a Four Row Wheat Bed Planting Machine which helps to save about 50% of the irrigation water with 15-20% increase in wheat grain yield and (ii) the use of border irrigation in conjunction with sprinkler irrigation under the irrigated agriculture conditions for increased crop yields at maximum water use efficiency. The paper also includes a briefing on the development of the Ring Automatic Irrigation System (RAI-System) which has a potential to replace expensive dripper/emitter in drip irrigation system for orchard irrigation. The system not only suppresses surface evaporation but also enhances water and other input use efficiencies.

Key Words: Bed planting, Sprinkler, Irrigation technologies, Water productivity

Introduction

Agriculture is the single largest sector of Pakistan's economy, although its contribution to the Gross Natural Product (GNP) has been steadily declining over the years as other sectors have expanded. Agriculture contributed 24.0 percent of GNP in 2002-2003. About 68 percent of the rural population depends on agriculture, which employs over 70 percent of the labor force and accounts for more than 80 percent of foreign exchange earnings. A little expansion in the agricultural area of Pakistan took place during the last forty years but the total area under agriculture is almost the same. Although the water resources of the country increased due to the construction of dams, development of canal system etc. during the last four decades thereby increasing the irrigated area this development has not been consistent with the population growth for achieving sustainable agriculture.

The crop productivity in the region is very low as the majority of farmers are still practicing traditional production techniques. Moreover, the cost of production has increased many times due to the rising prices of fuel and other agricultural inputs. At the same time, the existing production technologies also do not allow effective and efficient utilization of natural resources, particularly that of water.

Pakistan has the largest single contiguous gravity flow irrigation system in the world, known as Indus Basin Irrigation System which supply on the average about 142 Billion Cubic Meter (BCM) of water annually. The Indus Basin Irrigation System comprises three major reservoirs, 16 barrages, 2 head-works, 2 siphons across major rivers, 12 inter river link canals (all in the Punjab), 44 canal systems (23 in the Punjab, 14 in Sindh, 5 in NWFP and 2 in Balochistan) and more than 140,000 watercourses. The aggregate length of the canals is 34,834 miles (56,073 km). In addition, the watercourses, farm channels and field ditches cover another 1 million miles (1.6 million km). The watercourse commands range between 200 and 800 acres (80 to 320 ha). The overall efficiency of the Indus Basin Irrigation System (IBIS) is low (about 22%) and needs to be improved. The agricultural sector also utilizes an estimated quantity of 41.6 MAF (51.3 BCM) of groundwater pumped through more than 700,000 tubewells to supplement the existing canal supplies.

Out of all the four provinces, Punjab is the major contributor of agricultural produce in the country. There are 14 major barrages on the five rivers flowing in the heart of this valley, with a total off-take canal capacity of 0.12 million cusecs of irrigation supplies and another about 0.11 million cusecs capacity of inter river links. There are about 2,794 distributaries and minors (31039 km) feeding canals with a total length of over 58,000 outlets to supply irrigation water facilities for an area of fertile lands in the province.

The country has, however, recently faced serious water crises in its history. In fact, Pakistan was thrown into a water disaster in the 60s when water rights of three eastern rivers flowing through Pakistan's Punjab were completely provided to India under the Indus Water Treaty. Resultantly, the fertile lands of Pakistan particularly those of Punjab were converted into deserts due to the severe shortage of water due to the stoppage of flows from these rivers. The shortage of water supplies was further aggravated during the last few years due to lesser rainfall (than the normal), triggered mainly due to the disturbance in the main weather system responsible for generating water resources of the area.

The water shortages registered during the last few years were as high as 40 50 percent. The total inflow of western rivers during the Kharif season fell to 93.21 BCM in 2001 from 142.74 BCM in 1998, while in the Rabi season it dropped to 19.00 BCM in 2001-02 from 27.74 BCM in 1998-99. The total inflow of the western rivers fell to 112.21 BCM in 2001-02 and 195.69 BCM in 1992-93. Resultantly, canal withdrawals in the Punjab during the Rabi season fell to 12.03 BCM in 2001 02 that were more than 24.60 BCM from 1992-93 to 1995-96. The canal water deficit in the province usually remained around 2.45-7.06 BCM, which was recorded over 12.30 BCM during the regime of water crises. On the basis of the current water shortages and rapidly increasing future demands, the experts have foreseen that if this situation continues, it would not sustain the country's agriculture, which is the foundation of national economy.

Need to Conserve the Present Water Resources

The population of Pakistan was only 50 million in 1960 and has increased to over 140 million during the last four decades. Per capita water availability has also reduced from 5,650 m^3 to 1,400 m^3 during this period. Although the land resource has also declined on a per capita basis but still enough land resource can be brought under agriculture if additional irrigation water is made available. Thus, the availability of irrigation water is becoming a serious threat to the agriculture of the country.

The above facts demand an immediate change in the crop production practices through adoption of new improved technologies which can help to achieve efficient use of available water resources. Otherwise, it will become extremely difficult to meet the food demand of the increasing population of the country. The most favorable option for effective utilization of the available resources and increasing productivity is through promoting the adoption of high efficiency irrigation technologies.

Water Conservation Technologies

The Indus Basin Irrigation System operates at very low efficiency on account of significant water losses occurring mostly below the canal outlet (mogha) expressed in terms of delivery and application losses. As a consequence, more than 40% of the total diversion never reaches the farm gate. Out of the total diversion, about 26% is lost due to inefficient application. Lower conveyance efficiency further contributes to various unfavorable environmental conditions such as sediment deposition, rising of groundwater table and increase in drainage and salinity problems.

Watercourses which are about 135000 in number constitute the tertiary irrigation conveyance network. Generally farmers operate and maintain these watercourses. Command area of one watercourse is about 150-250 hectares (375-625 acres) and is shared by 40 to 50 farm families. Improvement of the watercourse may annually help to save a significant amount of water. Improvement of watercourses in the country started with a pilot on farm water management project during the year 1979. So far 40% of the watercourses have been improved under different on farm water management programs. The present government has initiated the improvement of the rest of the water courses under the "National Program of Watercourses Improvement". Thus, enough efforts are underway to control conveyance losses on the farm. However, there is much to do to control irrigation application losses by replacing the traditional irrigation methods (flood irrigation) with the high efficiency irrigation methods. Here are some of the innovative water saving technologies developed and being introduced on the farmers' fields for achieving the improvement in irrigation application efficiency.

I. Raised Bed Planting

The raised bed planting technique has shown an immense potential to achieve maximizing irrigation water saving and increase crop yield. It has been reported that bed planting also offers better weed control, water management and fertilizer use efficiency along with less crop lodging (Hobbs and Gupta, 2004). Improvements of root proliferation in bed planting (Peries et al., 2001) also enables better crop stand. It has been further reported that bed planting increases yield by at least 10%, reduces production costs by 20-30 % and saves irrigation water up to 35% as compared to conventional planting (Fahong et al., 2003). Therefore, bed planting can be considered one of the most feasible water conservation techniques to improve irrigation application efficiency in the country. However, presently bed planting is only used to grow some vegetables in Pakistan and not grain crops. The non-availability of proper machinery for the purpose is the major reason why farmers do not use bed planting for grain crops. Mexico is using bed planting for the concept of permanent beds and growing grain crops like wheat, maize and cotton etc (Sayre and Moreno Ramos, 1997). The beds are maintained for many years; however these are reshaped every year before growing crops. India has developed a bed planting machine called "Happy Seeder", which is used to grow 2, 3 and 5 lines of wheat in beds. The water saving has been reported to be 30% by the use of this machine.

Keeping the effectiveness of the technology in view a "Four-Rows Wheat Bed Planting Machine" was designed and developed at the Water Management Research Centre, University of Agriculture, Faisalabad – Pakistan.

Design and Development of Wheat Bed Planting Machine

Studies were conducted in the experimental area of the University of Agriculture, Faisalabad to evaluate a bed-furrow irrigation system for growing wheat. The following different sizes of furrow-bed planting were compared with 18 ft (5.5 m) border irrigation methods. The soil on the site was sandy loam. Each treatment was replicated thrice on the basis of Randomized Complete Block Design (RCBD) for data analysis.

- Six rows of wheat on a 135 cm bed-furrow system of 90 cm bed and (45 cm × 23 cm) furrow.
- Four rows of wheat on a 90 cm bed-furrow system of 60 cm bed and (30 cm × 23 cm) furrow.

The results of the study showed highest water use efficiency with the 60 cm bed of (45 cm x 23 cm) furrow for wheat. The study was repeated comparing only the 60 cm bed with 5.5 m border irrigation method. The detail of the results is presented in Tables 1 and 2. Keeping in view, the encouraging results for water saving by 60 cm beds, efforts were made to design a four-row wheat bed planting machine. The briefing on the machine is presented in the next section.

Table 1: Depth of water applied in each treatment (cm) and water saving

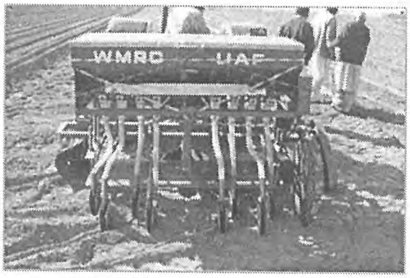
No. of irrigation	Depth of irrigation in borders (cm)	Depth of irrigation in bed planting (cm)	Water saving in wheat on beds
1st	7.5	3.83	49 %
2nd	7.44	3.53	52 %
3rd	7.25	3.45	52 %
4th	7.10	3.55	50 %
Subtotal	29.29	14.36	51 %
Irrigation for seed bed preparation	7.5	7.5	-
Total	36.79	21.86	40 %

Table 2: Grain yield and water use efficiencies for each of the treatment

Treatment	Depth of water Applied (cm)	Application efficiency (%)	Grain yield (kg ha^{-1})	W.U.E (kg ha^{-1} mm^{-1})
Flat Sowing (Border Irrigation)	35.60	57.93	4342	12.20
Furrow-bed (60 cm)	23.00	72.68	5124	22.28

DEVELOPMENT OF FOUR-ROW WHEAT BED PLANTING MACHINE

Tables 1 and 2 clearly show that four-row wheat bed planting on 60-cm beds not only saved 40 to 50 percent water but also increased yield by 15 to 20 percent. Based on these encouraging results, need was felt to design and develop a Bed planting machine for wheat, a crop which is a staple food in the country and which covers an area of above 65% in the Rabi season (spring season) when there is a big water shortage in the country. The machine was tested for growing wheat in an experimental area during the Rabi season (2003-04). On observing marvelous results of obtaining 50 % saving in irrigation water and 15-20 % increase in yield, the machine has been recommended to be used on the farmer's fields during the on-going wheat season (2004-2005) under the "Technology Transfer Program" of the University as a demonstration of the furrow-bed technology to the farmers. About 150 farmers used the machine on their farms under different field conditions. Some of the farmers have intercropped sugarcane in the furrows of bed planted wheat. Moreover, the machine has also shown its suitability to allow growing of maize and cotton. Keeping farmers response in view, it is expected that this will gain large scale popularity among the farmers to grow wheat in the country.



Salient Features of the Bed Planting Machine

The newly designed and developed wheat bed planting machine has the following salient features:

1. Planting Mechanism

The machine has been equipped with three adjustable furrow openers. These openers have the provision to change both the depth and top width of each furrow, separately. The provision of this adjustment allows the user to obtain the required size of furrow considering the type of crop, soil and its seed bed preparation. The machine develops two beds and three furrows in a single operation. The machine is designed to plant four rows of wheat in one bed. The geometry of sowing on a bed is that the machine develops two lines of crop on both sides of the bed and leaves 8 inches (20.32 cm) buffer zone in the center of four rows. The first line of crop is sown at 3 inches (7.62 cm) while the second line of crop is sown at 8 inches (20.32 cm) from the adjacent furrows. The center-to-center distance of two furrows is 36 inches (90 cm) with a bed of 24 inches (60 cm). Thus, the machine has provision to plant 4 lines in 36 inches width while maintaining the traditional plant population common with flat sowing by a traditional Rabi drill. Each furrow has to irrigate only 8 inches (21 cm) of the adjacent beds while there is buffer zone (21 cm) in the center of the bed. Thus, the machine not only addresses the dry stress problem but also salt stress, if there is any irrigation from saline water as the salts accumulate in the buffer area. The schematic diagram of the 4-row wheat bed planting system is shown in Figure 1.

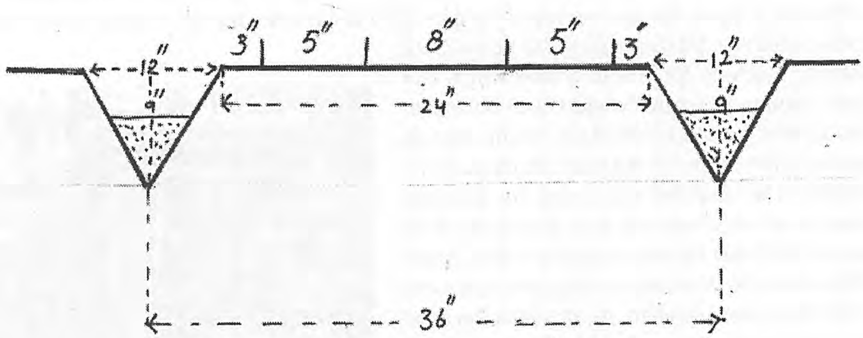


Fig. 1: Schematic diagram of the machine

Figures 2 & 3 show operation and outlook of the wheat sown by the wheat bed planting machine. The machine has a provision of adjustable seed planting mechanism as well, which not only allows adjusting the planting depth but also allows adjusting the spacing within the line to line distance of the crop if required. The seed planting system consists of discs rather than the traditional tines. The disc system in comparison with the tines does not disturb the shape of the bed and provides suitable small opening like fine channels to place the seed at a proper depth.

Fig. 2: Operation of the Bed planting Machine

All of the four discs are bolted on a rectangular iron bar through spring covered iron strips, which have been fixed on the main frame from where the sowing depth can be adjusted as a whole and/or on an individual disc bases. The spring provided on the iron strip between the disc and iron bar helps to keep the proper depth of opening in the bed. There is an adjustable planer fixed on the main frame prior to the seed planting mechanism, which not only presses and levels the bed top but also provides a flat surface on the bed for precise planting. As the planer covers all three furrows and two beds at a time, it results showing an in-built furrow-bed system with sharp and compacted edges of the furrows. Later on, the level beds are helpful for the harvesting of crop.

Fig 3: Out look of the wheat bed planting

This planting machine has the provision to apply fertilizer underneath the beds. However, fertilizer is applied in the center of two adjacent rows on each side of the bed at deeper depth than the seed. Moreover, fertilizer applicators are fixed on the front side of the main frame while the seed sowing mechanism is fixed on the side of the main frame.

II. Combined Use of Raingun and Surface Irrigation Method

The irrigation application losses approximately amount to 2.0 million hectare meter (MHM) in Pakistan. The improvement in irrigation application is not only important to save irrigation water, which is just available for the consumptive use of crops but also to reduce the loss of agriculture inputs. Almost 80% of the application losses occur through heavy I & II irrigations by traditional irrigation methods when only light irrigations are needed due to the small crop root zone. Haq (1990) compared sprinkler and drip irrigation with surface irrigation and concluded that water saving in case of drip irrigation and sprinkler irrigation were 37% and 30%, respectively. Jain and Kumar (1984) reported that sprinkler irrigation increased the yield of wheat and fodder maize by 48%. On the other hand, pressurized irrigation methods are expensive to apply. Keeping all these facts in view, studies were conducted applying irrigation with raingun sprinkler (FY-30) and raingun cum border irrigation and compared with irrigation applied by the border (22.5 m x 67 m) irrigation method for wheat. The soil on the site was sandy loam. The irrigation was applied on 50% depletion of available water in the soil.

Data for depth and number of irrigation times revealed that maximum water was applied by border irrigation (51 cm) through four irrigation times, followed by raingun sprinkler (41.5 cm) with eight irrigation times and then by raingun sprinkler cum border (39.25 cm) with six times of irrigation. The results showed that water savings using raingun sprinkler and raingun sprinkler cum border in comparison with border irrigation were 19 and 23.14%, respectively. Grain yield, water use efficiency and benefit cost ratio for each irrigation method is presented in Table 3. The Table shows that the raigun sprinkler cum border irrigation method gave maximum yield (5624 kg ha^{-1}) in comparison with the raingun sprinkler (5123 kg ha^{-1}) and border irrigation (4097 kg ha-^{1}). The percent increase in wheat grain yield in raingun sprinkler and raingun sprinkler cum border in comparison with border irrigation was 25 and 38%, respectively. The calculation made for water use efficiency, a ratio of crop yield to the amount of water applied , show that raingun sprinkler cum border irrigation gave a higher value of 1.43 than the raingun sprinkler (1.23) and border irrigation method (0.81). Similarly, a higher value of benefit cost ratio was also observed for raingun sprinkler cum border irrigation method.

Table 3: Grain yield, water use efficiency and benefit cost ratio for each of three methods.

Irrigation method	**Yield (kg ha^{-1})**	**Increase**	**Percent Increase**	**Water use efficiency**	**Benefit cost ratio**
Border	4096.69	-	-	0.81	1.00
Raingun sprinkler	5123	1026.3	25	1.23	1.82
Raingun sprinkler cum border	5623.7	1556.6	38	1.43	2.77

The studies revealed that the combined use of irrigation systems (Raingun and border irrigation systems) not only saved 23% irrigation water for wheat but also gave 38% higher yields. Similar results have been obtained for maize crop. Therefore, use of raingun and surface irrigation is being promoted at farmer's level.

III. Ring Automatic Irrigation System (RAI-System) for the Irrigation of Orchard

Although there is a marked interest to introduce trickle and sprinkler irrigation in the country but the actual field utilization is small. This is in partly due to the high initial costs and capital investments necessary for these systems and partly to the insufficient availability of reliable low cost equipment and parts. To reduce the initial cost of the systems, improve its reliability and acceptability at farmer's level, efforts are underway. It is believed that "traditional techniques relatively cost little and when combined with appropriate modern technology, can prove extremely effective".

Keeping in view the above objective and the experience of working with pitcher irrigation as a replacement to emitter in drip irrigation, the Ring Automatic Irrigation (RAI) system was developed for orchard irrigation. The system consisted of a small rectangular water control box, connected to a perforated pipe. The box provides a water reservoir for the perforated pipe which is buried around a tree at a depth of 4 to 6 cm. The water flow in the system follows the infiltration rate of the soil, where a float fixed in the box controls the inflow of water. The system is being tested to irrigate different fruit plants. The data collected so far is under analysis; however, the advantages observed using this method of irrigation are summarized below.



1. The system suppresses the evaporation loss as water seeps in soil through a perforated pipe buried at a depth of 4 to 6 cm.

2. The system is able to apply agricultural inputs directly in the root zone through a water control system.

3. The system can carry unfiltered canal water and works under low pressure.

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Improving conveyance and distribution efficiency through conversion of an open channel lateral canal to a low pressure pipeline at Al-Hassa irrigation project, Saudi Arabia

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IMPROVING CONVEYANCE AND DISTRIBUTION EFFICIENCY THROUGH CONVERSION OF AN OPEN CHANNEL LATERAL CANAL TO A LOW PRESSURE PIPELINE AT AL- HASSA IRRIGATION PROJECT, SAUDI ARABIA

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ABSTRACT

This paper describes the design procedures and use of a semi–open, semi-buried poly-vinyl chloride (PVC) pipeline system in Eastern Province, Saudi Arabia and its contribution in improving water conservation. PVC pipes were selected to construct a pipeline, 362 m in length, at Al-Hassa irrigation project. Easy maintenance, durability, modification flexibility and the low cost of the PVC pipes in Saudi Arabia, give them the potential to be the best economical alternative. An energy head, 2.7m of water, was used in determining the pipeline capacity and its internal diameter, using Poiseullie's law. Dismantling of a concrete canal and installing of the pipeline took place simultaneously without water stresses to the being grown crops. The conveance and distribution efficiencies increased by 10% and 25% respectively due to installation of the pipeline. Water losses, such as leakage and evaporation, were eliminated completely

Keywords: concrete canal, low-pressure pipeline, poly-vinyl chloride, distribution efficiency, conveying efficiency

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1. INTRODUCTION

Al-Hassa irrigation distribution system, 1100 km in length, consisted of elevated concrete open canals with a parabolic profile. Irrigation water was transmited via main and sub main canals to laterals by gravity from elevated reservoirs. Most of these lateral canals reached their life expectancy age and showed considerable deterioration. The average estimated conveying efficiency (Ec) at Al-Hassa irrigation net work is around 80% [1] due to seepage, leakage and evaporation.

The misuse of thehose siphons method at the lateral canals by farmers also created a poor distribution efficiency (Ed), about 50%. Usually, up stream farmers use more hoses than recommended and lower their trash boxes with respect to the lateral canal in order to gain more hydraulic head in order to deliver large quantities of water. Consequently, down stream farmers suffer water shortages and that encourages them to seek other alternatives. Excessive irrigation practices at the Al-Hassa irrigation project resulted in doubling the designed agricultural drainage water up to 3.73 m/s [2] and [3]. More than 33% of the available irrigation water goes to deep percolation.

Low-pressure pipeline systems with turnout structures are common in irrigation distribution systems, being used for both lateral and feeder lines. They have been used extensively since the 1950's. Most buried-pipe distribution systems for surface irrigation operate with a maximum pipe pressure which does not exceed 5 m and is frequently <3 m hydraulic head. Most low-pressure pipe systems have, until recently, been constructed from non-reinforced concrete pipe, but PVC pipe materials are now increasingly being used [4]. In [5], it was justified that a pipeline system is easier to automate than open channels. Also, it was indicated that a pipeline system offers greater seepage control, ease of water diversion and reduced maintenance. A low pressure pipeline system is efficient in water delivery and requires less energy head compared with an open channel system [6].

The use of buried-pipe systems and flexible surface hoses in the Hebei Province, China and their contribution to improved water use and increased management flexibility was described by [7]. A substantial increase in irrigation command areas has resulted from the adaptation of existing buried-pipe systems to incorporate the use of surface hoses to distribute irrigation supplies from the outlet.

Another example of a low-pressure pipeline system took place in Bangladesh. Data were collected from a fieldwork from 1989 to 1991 on eight buried-pipe systems in Shakipur and Ghatail Upazilas in the Tangail District, Bangladesh [8]. Transit losses on the eight concrete buried-pipe distribution systems was averaged 0.7 l/s/100 m, which was 10% of the transit losses measured on earthen channels in Bangladesh, and <25% of losses measured on lined systems.

Literature review showed little research data on the performance of buried-pipe distribution systems for surface irrigation. However, various reports noted the advantages of pipeline systems over open-channel systems [9], [10] and [11],and these advantages can be summarized as follow:

1-reduced losses of water from the system by seepage and leakage (transit loss),
2-reduced time for water to flow through the system to fields (travel time),
3-reduced land taken for the system and
4 -increased equity of distribution.

2. MATERIALS AND METHODS

2.1 Canal and hose siphon data

The entire concrete lateral canal, P1H4, totals about 888 m in length. Its upper half, 362 m, was chosen to be replaced with a PVC pipeline and controllable turnout structures. This section serves six farmers, for a command area of 1.1 hectare, of palm trees. Observation of farmers' misuses of hose siphons were recorded during the irrigation time without any interference. Data collected were shown in Table 1. Irrigation water is released in the lateral once a week. Although the water Distribution Department at Al-Hassa irrigation project recommended a four inch, hose siphon to irrigate 1000 m2 area for one hour, Farmers do not abide by this rule, it was noticed that they exceeded the recommended number. Farmers No. 1, 2, and 3, as shown in Table 1, irrigate on a different day from farmers No. 4, 5 and 6, and each group irrigates once every two weeks. Time of irrigation is within the range of 4 to 6 hours, including the uptake time of down stream users. Times of irrigation and hydraulic head (H) for hose siphons were noticed to be controlled by the farmers and consequently that affected the diverted amount.

Table1: Shows the P1H4 lateral canal farmer's misuses of hose siphons

Farmer No.	Trash Box	Area m^2	H cm	Recom. N Siphons	Farmer N Siphons	Q L/s	Irr. T hrs	Time seconds	Quantity m^3
1	1 -R	810	59	1-4"	2-4"	15.4	0.45	1620	49.896
1	1 -L	810	59	1-4"	2-4"	15.4	0.45	1620	49.896
2	2 -R	5280	35	2-4"	3-4"	11.8	3.5	12600	446.04
3	3 -L	710	30	1-4"	2-4"	11	2	7200	158.4
4	4 -R	760	25	1-4"	2-4"	10.2	2	7200	146.88
5	5 - R	620	20	1-4"	2-4"	9.5	2	7200	136.8
6	6 - R	2165	20	2-4"	3-4"	9.5	3.5	12600	359.1

Recom. =Recommended, Irr. = Irrigation, N=Number

In general, the average slope of the concrete open channel system at Al-Hassa is about 0.6 per 1000 m. However, for this particular lateral canal the elevation was 147.7 m a.s.l. at the inlet and 145.6 m a.s.l at the end. A steel pipe line, 15.3 m in length and 40 cm in diameter size, existed along with a trash box at the midway of the canal. They were left in place, in order to reduce the initial cost of the pipeline and at the same time to let the upstream tapped air to get released. Water is delivered into this lateral by inertial and gravity forces which initiated at the Swidra reservoir. The head inside the reservoir reaches 5 m during winter time, due to overnight storage, but, that decreases with time and as do the heads at the mains, sub mains and laterals. Maximum total average head in the sub main, which supplies the lateral, was estimated 2.7m during the winter time, when the maximum averaged discharge rate into the lateral was 0.102 m3 and flow depth was 60 cm.

2.2 Flow rate data

Initially we assumed maximum water flow-rate, *Q*max, and velocity, *V*max, of the pipeline would be the same as the maximum values of the concrete lateral canal. First, the parabolic flow cross section area (A) of the canal, at steady state flow, was determined by multiplying the product of the flow depth and the flow width by 2/3. Then, the flow-rate characteristic data at centerline of the canal were measured with manual and automatic current meters. Standard equations along with the characteristic flow measured data during the winter season of 2002/2003 were used in determining the averaged values of *Q*max and *V*max, via the canal centerline. Current meter Fan No.1-87026 equation was used in determining the velocity:

$$V = 0.2585n + 0.006 \quad \text{...1}$$

Where:

n is number of turns per second

The averaged values of 0.102 m3/s for flow-rate and 0.768 m/s for the velocity were taken as the maximum values via the pipeline centerline. As will be shown in the discussion, these values were used with the Hagen-Poiseuille equation to estimate the pipeline diameter (*D*).

3. Results and discussion

To achieve the objective of this work, which was an economic sound pipeline with controllable turnout structures in place of a concrete lateral canal and hose siphons, the following design features results were considered and discussed.

1-Pipeline diameter
2-Pipe material
3-Pipe friction head loss
4-Turnout structures head loss
5-Implementaion
6-Pipeline efficiency

3.1 Pipeline diameter

Poiseullie's equation [12] was rearranged and used with previously measured flow data in calculating the pipeline diameter (*D*).

$$D = \sqrt{\frac{4Q_{max}}{\pi V_{max}}} \quad \text{...2}$$

Where:

Q max = flow rate at the centerline of the pipe (m3/s)
V max = velocity at the centerline of the pipe (m/s)
D = pipe diameter (m)

Though the calculated diameter was 410mm (16 in), for the reason of minimizing the initial cost of the pipeline a smaller size of 305mm (12 in) was preferred.

3.2 Pipeline material

The availability of low-cost polyvinyl chloride (PVC) pipes in Saudi Arabia and their easy handling because of their lightweight gives the PVC pipeline the potential of being the best alternative to replace concrete open channel.

At the Al-Hassa Irrigation Project, a rigid PVC irrigation pipeline, 10 mm wall thickness and 300mm inside diameter, was selected to replace the P1H4 concrete lateral canal. PVC pipes are inert to almost all chemicals that are likely encountered during irrigation and easy to manipulate in terms of developing more connections [13]. PVC pipes can be exposed to sunlight without deterioration (Jensen, 1983),and that is why they were picked to suit Al-Hassa environment.

The standard dimension ratio (SDR) of the chosen PVC pipe was 30 (Pipe diameter divided by thickness) and from Table 11.12 in (Jensen, 1983, taken from ASAE Standard S376.1). The pressure rating was about 700 kPa (7 m of water) at 23 oC. This high value of pressure rating allow for any future upgrading.

3.3 Pipeline friction head losses

The pipeline friction head losses were estimated to obtain accurate design values. These losses depend on the internal pipe surface and the fittings. In estimating the head losses, Reynolds number (*Re*) and the relative roughness of the pipe (*e/D*) were determined first. Then the kinematic viscosity of the irrigation water, v =1.12X10-6 m2/s, the pipe flow velocity, *Vmax* =0.768 m/s, and its internal diameter, D = 0.305m were used in determining the *Re*.

$$R_e = \frac{VD}{v} \quad \text{...3}$$

Where:

v : is the kinematic viscosity of the fluid (L2/T).

A moderate value of the *Re*, 27777, was calculated for the PVC pipeline, which then was used in determining the friction factor, *f*.

The most fundamentally sound method for evaluating head losses due to friction in closed pipe conduits was the Darcy–Weisbach equation [14] and [15].

$$h_f = f\frac{L}{D}\frac{V^2}{2g} \quad \text{...4}$$

Where:

hf :is the head loss due to friction that has the unit of length (L).
L: is the length of the pipe (L).
D: is the internal diameter (L).
V: is the average velocity (L/T).
g: is the acceleration due to gravity (L/T2).

Based on equations 3 and 4, and the Moody chart, the head loss of the PVC pipeline was found to be 1.6 cm.

Local water head losses through the turnout structures of the pipeline were calculated by the following equation:

$$H_f = \frac{KV^2}{2g} \quad \text{...5}$$

The resistance coefficient, K, for a turnout of a diameter 10.15 mm (4-inch) was 0.68; it was extracted from Table 11.9 (USDA, SCS, 1968 as shown in Ref (Jensen, 1983)).

Thus, $Hf = 0.68*(0.768)2 / (2*9.81) = 0.0204$ m. Since there were seven turnouts operating at a time, the total local head local losses were: $Hf = 0.0204*7 = 0.14$ m. Therefore, the total head loss of the pipeline and the turnouts was: HT= 1.6+14 = 15.6 cm.

3.4 Turnout flow rates

Two different diameter sizes for the turnout structures were chosen to substitute the traditional hose siphons sizes; these were 101.6 mm (4 in) and 76.2mm (3 in). A submerged pipe flow equation was used to determine the discharge of the turnout:

$$Q = CA\sqrt{\Delta H} \qquad \text{...6}$$

Where:

Q: is the pipe discharge in L/s
A: cross sectional area in cm2
DH: water head between the main pipe and the outlet in cm
C: is 0.03, discharge coefficient

Accordingly, the flow rate of the four inch (101.6 mm) diameter was 14 L/s, when 7.9 L/s were for the three inch diameter (76.2mm) turnout. That means the flow rate of the controllable turnouts exceeded the traditional hose siphon flow rates, which were 11.8L/s and 7.3 L/s respectively.

3.5 Implementation

A sketch of proposed pipeline along with turnout structures as shown in Figure 1 was prepared and provided to local contractor for implementation. A control valve was placed at the end of the pipeline to adjust the outflow.

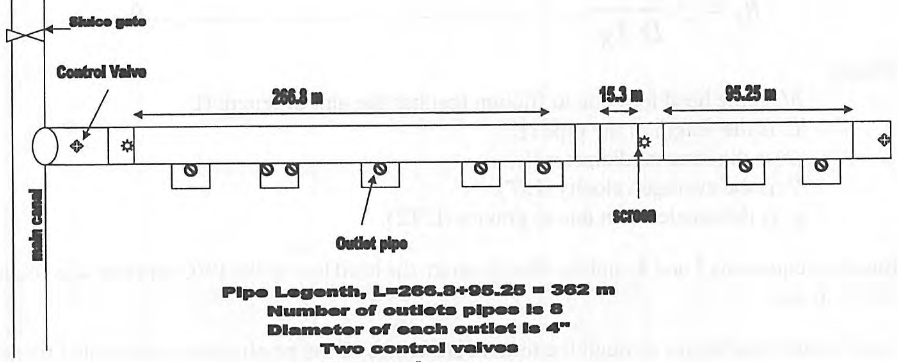

Fig. 1: A sketch of proposed pipeline and turnout structures

Removal of the concrete canal segments and replacement of the PVC pipes took place simultaneously after an irrigation day. The connected pipes were placed on bases of the concrete canal, as shown in Figure 2, to maintain the same canal's slope. That was because of the limited hydraulic head, 2.7 m of water, available to deliver the water into the pipeline and onto the farms. Figure 2 shows the pipeline installation and operation.

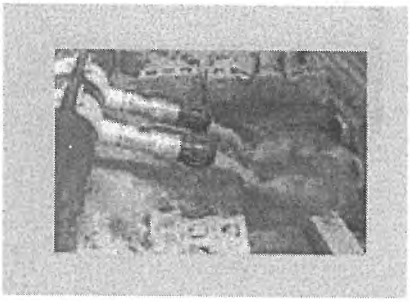

Fig. 2: Pictures of PVC pipeline and four inch turnout structures

3.6 Pipeline efficiency

During a period of four hours of irrigation, the average volume of irrigation water diverted by the farmers No. 4, 5 and 6, the estimated to be 642.8 m3. At the same period, a volume of 216 m3 was diverted by a down stream user. Therefore, the average delivered volume of irrigation water by all farmers on that day of irrigation totals to 858.8 m3. The conveyance efficiency of the concrete canal was obtained by the ratio of the volume of water delivered for irrigation to volume of water, 1065.6 m3, placed into the canal. That resulted to a canal conveyance efficiency of 81%, which agrees with averaged conveyance efficiency for the Al-Hassa irrigation project that estimated earlier by [1]. With the operation of the concrete canal, two out four farmers used to irrigate at a time, which resulted to a distribution efficiency of 50%. But, after the pipeline operation, three out four the farmers could irrigate together, which raised the distribution efficiency up to 75% (Table 2). Moreover, some farmers connected their turnouts with flexible pipes, in place of earthen ditches, to divert the irrigation water onto their farms. This may improve the conveying and application efficiency (Ea) on the farms. Improving these efficiencies will ultimately improve the overall efficiency of that irrigation system (E0=Es*Ec*EA*).

Over- irrigation is a common tendency among Al-Hassa farmers, because there is no control over the amount of irrigation water diverted by the farmers [2]. Figure 3 shows the effect of various scenarios of no control on hose siphons uses on the diverted amounts. The lowest rates of the diverted volume of irrigation water are shown when the correct hydraulic head and number of hoses were used.

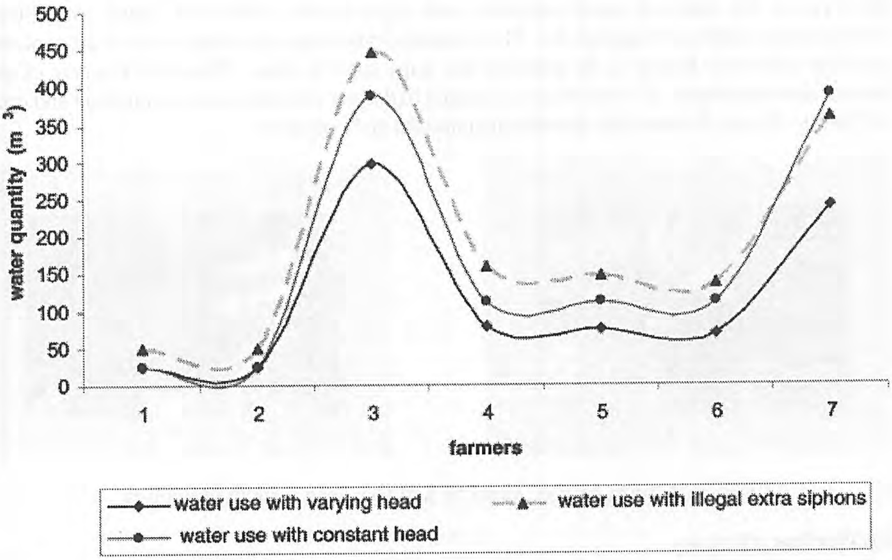

Fig.3: Diverted volume of water from P1H4 canal as farmers varying number and head of hose siphons

Table 2: comparison between distribution and conveyance efficiencies of the concrete P1H4 lateral canal and pvc pipeline

Pipeline	P1H4 lateral canal	
75%	50%	Distribution efficiency
91.7	81%	Conveyance efficiency

4. Conclusions

The availability of polyvinyl chloride (PVC) pipes and easy handling made it possible to convert a distribution system from a concrete canal and hose siphons to a PVC pipeline with turnout structures. Low pressure pipeline was designed, installed and operated effectively with 2.7 m energy head in place of a concrete canal at the Al-Hassa irrigation project. The least effective cost pipe diameter, 304 mm, was chosen after a diameter estimation using flow data of the concrete canal and the Poiseullie's law.

By having control on the farmers' diverted amounts, operation at full capacity of the pipeline was achievable only when the turnout amounts were equal or greater than the pipeline capacity. That allowed more farmers to irrigate at a time than used to be and improved the distribution efficiency from 50% to 75%. Also it can be concluded that introduction of the pipeline and the turnouts reduced the chances for upper streamers to exercise priority rights over irrigation practices. In addition to that, some farmers were motivated and decided to connect their turnout structures with flexible pipes in order to deliver the water into their farms. That was a very interesting and encouraging positive move, which in turn will lead in improving the application efficiency.

In this study, the conveyance efficiency of the replaced section (P1H4) of the Al-Hassa irrigation network increased by 10% due to pipeline elimination of the leakage and evaporation. In general, improving the application and the conveying efficiency of the Al-Hass irrigation network will lead an improvement of the over all efficiency.

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تحسين كفاءة نقل و توزيع المياه من خلال تحويل قناة الري الفرعية المكشوفة الى حط انابيب منخفض الضغط في مشروع الري بالاحساء في المملكة العربية السعودية

الملخص

تصف هذه الورقة الخطوات التصميمية و الاستخدام لانبوب (   ) شبه مفتوح و مدفون جزئيا في المنطقة الشرقية من المملكة العربية السعودية ، و مساهمة ذلك تطوير المحافظة على المياه . تم اختيار البولي فنايل كلورايد لانشاء خط انبوبي طوله ٣٦٢م في المنطقة التي يغطيها مشروع الري و الصرف بالاحساء . أن سهولة الصيانة و طول العمر الافتراضي و مرونة التحوير و انخفاض التكلفة ، كل ذلك يجعل من هذا النوع من الانابيب خيارا اقتصاديا واعدا لظروف المملكة العربية السعودية . و لقد تم احتساب ٢,٧ متر كضاغط طاقة لتقدير كمية المياه المارة عبر الانبوب و كذلك قطره الداخلي عن طريق معادلة بواسيل . ازيلت القناة الفرعية الخرسانية المغذية للمزارع الواقعة حولها بالتزامن مع تثبيت الانبوب ، حتى لا تتعرض المحاصيل المروية لأي اجهاد و دلت النتائج إلى ازدياد كفاءة النقل و التوزيع بنسبة ١٠% و ٢٥% على التوالي . مما يدل على انخفاض الفواقد المائية الناتجة عن التسرب و التبخر .

Calculation of discharge in compound channels

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CALCULATION OF DISCHARGE IN COMPOUND CHANNELS

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ABSTRACT

Calculating the discharge in open channels, especially in compound channels is a problem that encounters many engineers involved in the design and management of irrigation channels, rivers and drainage systems. This is because the discharge calculated by conventional methods is usually exaggerated. This consequently led to designing uneconomic channel sections with capacities more than that actually needed. Based on the hypothesis that when the flow in a compound channel is deep the compound channel approaches single unit behaviour a new method was developed. This method was tested using data from different sources, and then compared with some other methods. The result showed that this method is simpler and predicts more satisfactory discharge values than the others do, and it can be applied to symmetric or asymmetric channels with homogeneous or non-homogeneous roughness.

Key words: Compound Channels, Discharge, Overbank, Alluvial

INTRODUCTION

Calculating conveyance and roughness of channels when flow goes overbank is a major problem to engineers involved in design and management of irrigation channels, rivers and drainage systems. This is because of the interaction effect between fast flow of the main channel and the slow flow of the floodplains, which results in transfer of momentum from the main channel to the floodplains. Most conventional methods of discharge calculations, such as Chow (1986), attempt to account for the large variation in velocity across the channel section by dividing it into sub-sections, each of which is considered as hydraulically homogenous. The discharge in each sub-section is then calculated using one of the well-known formulae such as Chezy, Manning or Colebrook-White. However, such a traditional method fails to take this interaction into account, which results in poor estimation of discharge. The main objective of this study is to produce a simple and practical method for predicting the total discharge in compound channels.

THEORETICAL BACKGROUND

To tackle the poor estimation of discharge by the traditional method the interaction effect was taken into consideration and different one-dimensional (1D), two-dimensional (2D) and three-dimensional (3D) models have been developed. Examples of 1D empirical models based on apparent shear stress acting on a particular interface are those developed by Wormleaton, Allen & Hadjipanos (1982), Knight & Demetriou (1983) and Wormleaton & Merrett (1990). 2D models based on depth averaged parameters have been developed to give lateral distribution of both velocity and boundary shear stress. See for example, Keller & Rodi (1988) and Shiono & Knight (1990 & 1991). Examples of 3D turbulence models are those developed by Krishnapan & Lui (1986) and Tominaga & Nezu (1991). 2D and 3D models are more complicated and hence they are more frequently used in research. 1D models are less accurate, but due to their simplicity practical engineers favour them; such models are those produced by Ackers (1992, 1993) and Lambert and Myers (1998). These two models adopt different concepts in which the interaction effect is implicitly expressed as explained hereafter. The intensity of the interaction effect between the main channel and the floodplain flows has been related to the relative depth, H*, [H*= (H-h)/H] by all researchers dealing with compound channels. A typical compound channel and symbols used are shown in Figure 1.

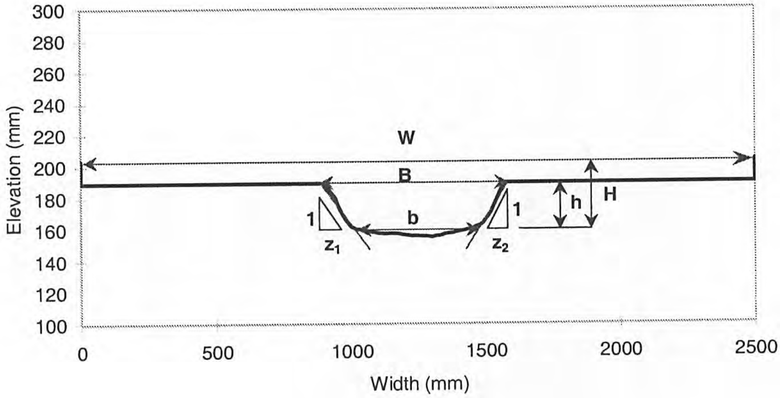

Figure 1: The applied average hydraulic parameters in a compound channel.

Ackers (1992, 1993) merged all the parameters that have influence on the interaction effect in one dimensionless parameter termed the channel Coherence (Coh), and came up with a practical 1D model known as the Coherence method. The coherence of a compound channel is defined as the ratio of the nominal conveyance calculated by treating a channel as a single unit to that calculated by summing the conveyances of the separate flow zones. When Manning equation is used, it is expressed as

$$Coh = \frac{\dfrac{\sum_{i=1}^{i=N} A_i^{5/3}}{\sum_{i=1}^{i=N}\left[n_i p_i^{2/3}\right]}}{\sum_{i=1}^{i=N}\left[\dfrac{A_i^{5/3}}{n_i p_i^{2/3}}\right]} \qquad \textbf{Eq.1}$$

where p_i , A_i and n_i are wetted perimeter, area and the Manning's roughness coefficient of i=1 to N flow zones respectively. Based on the Coherence concept, Ackers (1992, 1993) was the first who linked the intensity of the interaction effect to different regions of flow. In this method, first the nominal total conveyance of a channel is computed using the traditional divided channel method with a vertical interface, which is excluded from the wetted perimeter calculation. The nominal total conveyance is then adjusted for the interaction effect using different empirical equations via adjustment factors in each region of flow. Finally, a procedure has to be followed in order to define the applicable region of flow and hence selecting the appropriate total adjusted flow. This long procedure is well explained and documented in Ackers (1992, 1993), and thus for details it is advised to refer to.

Another 1D model of practical use is that developed by Lambert & Myers (1998). Lambert and Myers (1998) Weighted Divided Channel Method (WDCM) assumes there is an interface appropriate to account for the momentum transfer and this interface lies somewhere between the horizontal and vertical interfaces. Instead of determining the location of this interface explicitly, they suggested a weighting factor to allow a transition between the velocities determined by assuming a horizontal interface and a vertical interface. Though, they stated that this factor varies between 0 and 1, they gave a single weighting factor value of 0.5 for smooth channels and 0.2 for rough channels. The equation for calculating the total discharge is,

$$Q = A_{mc} (x\, U_{mc\text{-}V} + (1\text{-}\, x)\, U_{mc\text{-}H}) + \sum_{i=1}^{i=m} A_{fp} [\xi U_{fp-V} + (1-\xi) U_{fp-H}]$$

Eq.2

where,
$U_{mc\text{-}V}$ = the calculated velocity in the main channel using vertical division.
$U_{mc\text{-}H}$ = the calculated velocity in the main channel using horizontal division.
A_{mc} = area of the main channel.
$U_{fp\text{-}V}$ = the calculated velocity in the floodplain using vertical division.
$U_{fp\text{-}H}$ = the calculated velocity in the floodplain using horizontal division.
A_{fp} = area of the flood plain
Q = the corrected total (overall) discharge.
x = weighting factor

It is known that the intensity of the interaction effect varies with the relative depth and hence any correction factor used to account for this variation must be a function, or a series of constants to be introduced at different relative depths (or range of relative depths). Thus, due to the long procedure associated with the empirical nature of the Ackers (1992, 1993) Coherence method (COHM), and the use of a single empirical weighting factor by Lambert and Myers's (1998) Weighted Divided Channel Method (WDCM), most of the predicted discharges by the two methods are not close enough to the measured values. From the previous discussion it can be understood that there is a need for a simple formula close to the form of the WDCM, but which has variable adjustment factors making use of the relative depth and the Ackers Coherence concept.

METHOD DEVELOPMENT

In a compound channel it is known that the interaction between the main channel and the floodplain flows is maximum at lower relative depths, and then it decreases with increase in the relative depth until it become negligible at higher relative depths. When the relative depth of a compound channel approaches unity the channel coherence approaches unity (Ackers, 1992, 1993) and hence the divided channel method with vertical division and the single channel method calculations are approximately equal.

Thus, consider a compound channel where its main channel (bankfull flow channel) banks are gradually inundated until they reach a maximum level. Assume also that according to the channel coherence at each relative depth the total discharge in the compound channel is partially calculated by the single channel method and partially by the divided channel method with vertical division according to Equation 1,

$$Q = AF_1\, Q_{SC} + AF_2\, Q_{DC\text{-}v} \qquad \textbf{Eq.3}$$

where Q is the total adjusted flow, Q_{SC} is the flow calculated using the single channel method, $Q_{DC\text{-}v}$ is the flow calculated using the divided channel method with a vertical division, AF_1 and AF_2 are adjustment factors that are a function of (H*, Coh). To determine these adjustment factors two extreme boundary conditions were considered at which the channel can be treated as a single unit. These are, the compound channel is at the threshold of inundation (at relative depth = 0, i.e. bankfull channel) and the channel is inundated to a maximum level (the relative depth approaches unity).

At the first boundary condition, i.e. when H*=0, the flow can be calculated by the single channel method only if AF_2=j(H*)=0 and AF_1=j(H*, Coh)=1 or if AF_1=j(H*)=0 and AF_2=j(H*, Coh)=Q_{SC}/Q_{Dc-v}. The first assumption does not satisfy Eq.1, because at H*=0 the Coh '''1. The second assumption satisfies Eq.1 only if AF_2= (Coh-H*) = $Q_{SC}/Q_{Dc-v.}$ Based on this, Eq.1 can be rewritten as

$$Q= H^*Q_{SC} + (Coh-H^*)\, QDC_{-v}$$
Eq.4

Thus, at the first boundary condition, $H^*=0$ and $Coh\text{-}H^*= Q_{SC}/Q_{Dc-v}$ as $Coh=Q_{SC}/Q_{Dc-v}$ and hence $Q= Q_{SC}$. At the second boundary condition, *H* H" Coh H" 1.0*, $Coh\text{-}H^* = 0$ and hence $Q=Q_{SC.}$ As AF_1 and AF_2 are dimensionless factors, this method will be called the Dimensionless Total Flow Adjustment Method (DTFAM).

Between the two boundary conditions, i.e. when $0<H^*<1$ it was hypothesised that the total adjusted flow is contributed by both the single channel method and the divided channel method with ratios equal to their adjustment factors AF_1 and AF_2.

Results and discussion

Figure 2 shows a comparison in discharge calculation between all these methods, but in terms of the discharge discrepancy ratio ($Q_{predicted}/Q_{measured}$) using 58 rigid boundary tests chosen from the sources shown in Table 1. This data associated with detailed calculation of the discharge by the three methods (DTFAM, COHM and WDCM) are also shown in Tables 3 & 4. As shown in Figure 2, the new method underpredicts the total discharge at very low relative depths; however, this underprediction rarely exceeds 10%. With the same low relative depths the other two methods overpredict the discharge by more than 15 %, (15-32%); this is evident up to a relative depth of 0.3m. At relative depths higher than 0.3m they almost have the same behaviour, though the new method behaves even better as most of the data is located between the ± 10% range.

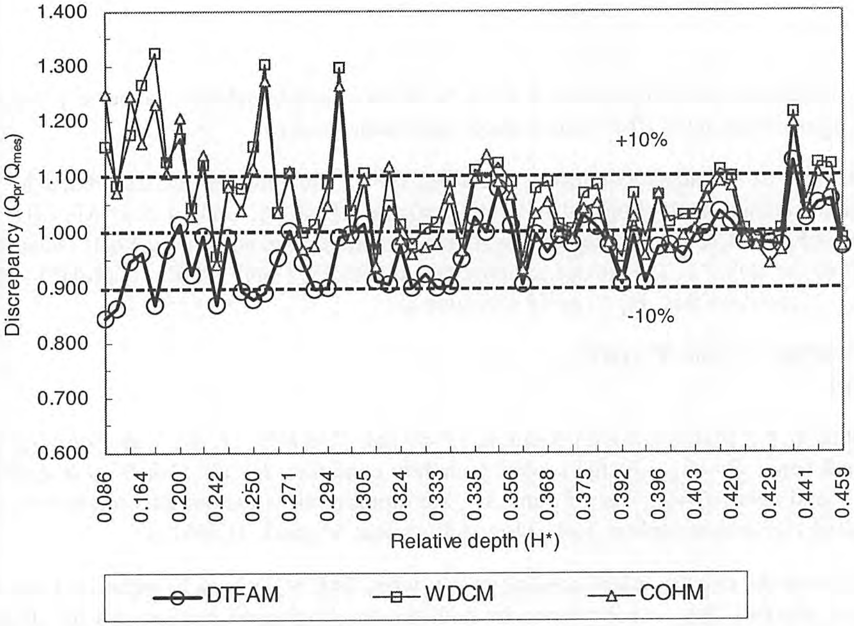

Figure 2: Comparison between the DTFAM, COHM and WDCM discharge prediction

Thus it can be concluded that the best predicted total discharge was obtained by the new method DTFAM, where most of the data is located within ± 10%, then the COHM comes next and finally the WDCM. This is also evident in Table 2 where with this new method the Maximum Absolute Error (MAE) is 5.13%, while the WDCM and the COHM have less accuracy at 6.91% and 7.35% respectively (see Table 2).

CONCLUSUIONS

In this study a new method for calculating the total discharge in compound channels was developed. This method is based on Ackers channel coherence concept and relies only on two dimensionless adjustment factors which vary with the relative depth. These adjustment factors are only functions of relative depth and channel coherence. In addition, it uses the well-known traditional divided channel method with vertical division and the single channel method, which makes the method very simple to apply. This method has improved features over the other 1D method, which are summarised as:

1. Simple and dimensionless this may make it of general use by practice engineers.
2. Can be applied to channels with homogenous or non- homogeneous roughness (refer to Table 2).
3. Can be applied to symmetric and asymmetric channels.
4. It predicts better discharge values compared to the Coherence and the Weighted Divided Channel methods.

Table 1: The experimental data, characteristics and data sources, which are used in the comparison

Data source	Channel type	NO.	Channel characteristics							
			$Q (ls^{-1})$	$S*10^{-3}$	$(H-h)/H$	n_{fp}/n_{mc}	W/B	B/h	b/h	z
Prinos et al. (1985)	Rigid symmetric	2	30.2,32.2	1.0	0.164	1.0-1.273	2.701	5.98	4.98	0.5
Knight and Demetrriou (1983)	symmetric	13	7.3-29.4	0.966	0.131-0.506	1.0	2.0-4.0	2.0	2.0	0.0
Wormleaton et al. (1982)	symmetric	17	15.0-48.0	0.430	0.250-0.429	1.308-1.963	4.175	2.417	2.417	0.0
Nousopoulos and Hadjipanos (1983)	symmetric	16	9.0-45.0	1.5	0.187-0.479	1.0	4.0-6.667	2.0	2.0	0.0
Myers (1978)	asymmetric	10	6.3-18.2	0.265	0.086-0.394	1.0	3.803	2.49	2.49	0.0
	Total	**58**								

Table 2: Mean Absolute Error (MAE), Maximum Error (Max.E) and Minimum Error (Min. E) for different methods in calculating total discharge in compound channels (Table 1).

Channel boundary	Error Type	DTFAM	COHM	WDCM
Rigid 58 tests	MAE(%)	**5.13**	**6.91**	**7.35**
	Max.E(%)	-15.40	27.13	32.31
	Min.E(%)	-0.17	-0.08	0.09

Table 3: Comparison between the Dimensionless Total Flow Adjustment Method (DTFAM) and the Ackers Coherence method (COHM)

Source	Test	H	H*	S×10³	Q_exp (l/s)	Calculated Q_bfcs	Calculated Q_bfcs/Q_exp	Calculated Error%	DTFAM Q_p (l/s)	DTFAM Disc.	DTFAM Error%	COHM Q_p (l/s)	COHM Disc	COHM Error%	nc	nf	n*	W*	B/h	b/h	z
Prinos et al. (1985) 381 610 381; var.(38,102); 920; material not known		122	0.164	1.000	32.500	44.269	1.362	36.211	31.307	0.963	-3.671	37.768	1.162	16.208	0.0110	0.0110	1.000	2.701	5.980	4.980	0.500
		122	0.164	1.000	30.200	43.606	1.444	44.371	26.307	0.871	-12.890	37.150	1.230	23.014	0.0110	0.0140	1.273	2.701	5.980	4.980	0.500
Knight & Demetriou (1983); variable (304,456,610); 76; 152; Perspex		100.3	0.242	0.966	7.300	7.313	1.002	0.181	6.339	0.868	-13.162	6.869	0.941	-5.893	0.0099	0.0099	1.000	2.000	2.000	2.000	Rect. z=0
		113.5	0.330	0.966	9.450	9.533	1.009	0.877	8.505	0.900	-9.997	9.089	0.962	-3.820	0.0099	0.0099	1.000	2.000	2.000	2.000	
		125.8	0.396	0.966	11.700	11.793	1.008	0.799	10.628	0.908	-9.162	11.201	0.957	-4.269	0.0099	0.0099	1.000	2.000	2.000	2.000	
		149.8	0.493	0.966	17.100	16.654	0.974	-2.607	15.003	0.877	-12.262	15.771	0.922	-7.769	0.0099	0.0099	1.000	2.000	2.000	2.000	
		87.5	0.131	0.966	5.000	5.710	1.142	14.209	4.319	0.864	-13.614	5.365	1.073	7.302	0.0099	0.0099	1.000	3.000	2.000	2.000	
		96.1	0.209	0.966	6.700	7.363	1.099	9.901	6.202	0.926	-7.429	6.903	1.030	3.032	0.0099	0.0099	1.000	3.000	2.000	2.000	
		112.1	0.322	0.966	11.150	11.099	0.995	-0.461	10.168	0.912	-8.805	10.478	0.940	-6.027	0.0099	0.0099	1.000	3.000	2.000	2.000	
		125	0.390	0.966	15.150	14.592	0.963	-3.683	13.742	0.907	-9.291	13.962	0.922	-7.837	0.0099	0.0099	1.000	3.000	2.000	2.000	
		149.2	0.491	0.966	23.400	22.056	0.943	-5.742	21.196	0.906	-9.419	21.447	0.917	-8.347	0.0099	0.0099	1.000	3.000	2.000	2.000	
		102.1	0.256	0.966	9.100	9.789	1.076	7.569	8.677	0.953	-4.652	9.420	1.035	3.519	0.0099	0.0099	1.000	4.000	2.000	2.000	
		114.2	0.336	0.966	13.500	13.722	1.016	1.643	12.855	0.952	-4.780	13.046	0.966	-3.367	0.0099	0.0099	1.000	4.000	2.000	2.000	
		127.4	0.403	0.966	18.000	18.601	1.033	3.337	17.906	0.995	-0.520	18.082	1.005	0.456	0.0099	0.0099	1.000	4.000	2.000	2.000	
		153.8	0.506	0.966	29.400	30.058	1.022	2.238	29.530	1.004	0.440	29.689	1.010	0.982	0.0099	0.0099	1.000	4.000	2.000	2.000	
Wormleaton, Allen & Hadjipanos (1982); 460 460; 120; 290; smooth concrete blocks	B2	160	0.250	0.430	20.500	23.760	1.159	15.902	18.316	0.893	-10.652	21.306	1.039	3.931	0.01070	0.01400	1.308	4.175	2.417	2.417	Rect. z=0
	B3	170	0.294	0.430	26.000	28.247	1.086	8.642	23.285	0.896	-10.441	25.342	0.975	-2.530	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B4	180	0.333	0.430	31.000	33.122	1.068	6.846	28.681	0.925	-7.479	30.132	0.972	-2.798	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B5	190	0.368	0.430	38.000	38.351	1.009	0.924	34.472	0.907	-9.284	35.419	0.932	-6.753	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B6	210	0.429	0.430	48.000	49.759	1.037	3.665	47.127	0.982	-1.819	46.853	0.976	-2.390	0.01070	0.01400	1.308	4.175	2.417	2.417	
	C3	160	0.250	0.430	18.000	22.686	1.261	26.038	15.806	0.878	-12.191	20.054	1.114	11.411	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C4	170	0.294	0.430	22.500	26.724	1.188	18.772	20.270	0.901	-9.909	23.559	1.047	4.707	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C5	180	0.333	0.430	28.000	31.085	1.110	11.018	25.161	0.899	-10.138	27.391	0.978	-2.175	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C6	190	0.368	0.430	30.500	35.750	1.172	17.212	30.448	0.998	-0.172	31.516	1.033	3.332	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C7	200	0.400	0.430	37.000	40.695	1.100	9.986	36.101	0.976	-2.429	35.907	0.970	-2.953	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C8	210	0.429	0.430	43.000	45.901	1.067	6.746	42.097	0.979	-2.099	40.548	0.943	-5.700	0.01070	0.01700	1.589	4.175	2.417	2.417	
	D3	160	0.250	0.430	15.000	21.750	1.450	44.999	13.361	0.891	-10.925	19.070	1.271	27.133	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D4	170	0.294	0.430	17.500	25.370	1.450	44.969	17.319	0.990	-1.035	22.118	1.264	26.387	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D6	180	0.333	0.430	24.000	29.273	1.220	21.972	21.698	0.904	-9.591	25.433	1.060	5.971	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D6	190	0.368	0.430	27.500	33.437	1.216	21.590	26.471	0.963	-3.744	28.989	1.054	5.416	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D7	200	0.400	0.430	33.000	37.842	1.147	14.673	31.609	0.958	-4.214	32.764	0.993	-0.715	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D8	210	0.429	0.430	38.000	42.471	1.118	11.766	37.090	0.976	-2.394	36.740	0.967	-3.317	0.01070	0.02100	1.963	4.175	2.417	2.417	
Noutsopoulos & Hadjipanos (1983); variable (1000,800,600); 75; 150; smooth concrete blocks	A11	92.25	0.197	1.500	9.000	10.595	1.176	17.609	8.722	0.969	-3.094	9.968	1.108	10.757	0.01000	0.01000	1.000	6.667	2.000	2.000	Rect. z=0
	A12	107.25	0.301	1.500	18.000	19.105	1.061	6.139	17.931	0.996	-0.386	17.818	0.990	-1.013	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A13	119.35	0.371	1.500	27.000	27.633	1.023	2.345	26.837	0.994	-0.604	27.028	1.001	0.103	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A14	122.25	0.387	1.500	30.000	29.971	0.999	-0.098	29.250	0.975	-2.498	29.428	0.981	-1.905	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A15	138.75	0.450	1.500	45.000	44.121	0.980	-1.953	43.720	0.972	-2.644	43.832	0.974	-2.595	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A21	105.00	0.286	1.500	15.000	15.437	1.029	2.880	14.190	0.946	-5.401	14.549	0.970	-3.009	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A22	111.00	0.324	1.500	18.000	18.595	1.033	3.306	17.545	0.975	-2.529	17.772	0.987	-1.269	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A23	119.25	0.371	1.500	23.000	23.373	1.016	1.621	22.532	0.980	-2.033	22.733	0.988	-1.169	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A24	129.75	0.422	1.500	30.000	30.084	1.003	0.280	29.447	0.982	-1.842	29.615	0.987	-1.285	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A25	139.75	0.468	1.500	36.000	36.338	1.009	0.938	35.832	0.995	-0.468	35.973	0.999	-0.075	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A26	141.00	0.468	1.500	37.000	37.968	1.026	2.615	37.488	1.013	1.322	37.624	1.017	1.686	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A41	98.75	0.248	1.500	10.000	11.289	1.127	12.890	9.914	0.991	-0.861	10.823	1.082	8.233	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A42	114.75	0.348	1.500	15.000	17.156	1.144	14.376	16.161	1.077	7.742	16.386	1.092	9.241	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A43	120.80	0.375	1.500	18.000	19.471	1.082	8.173	18.564	1.031	3.134	18.782	1.043	4.344	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A44	134.25	0.441	1.500	25.000	26.311	1.052	5.245	25.574	1.023	2.296	25.774	1.031	3.095	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A46	144.80	0.479	1.500	30.000	31.405	1.047	4.683	30.738	1.025	2.461	30.930	1.031	3.099	0.01000	0.01000	1.000	4.000	2.000	2.000	
Myers (1978); 396; 102; 254; Perspex	10	111.6	0.086	0.265	6.300	7.615	1.209	20.874	5.330	0.846	-15.397	7.874	1.250	24.977	0.00900	0.00900	1.000	3.803	2.490	2.490	Rect. z=0
	9	118.3	0.136	0.265	7.000	8.704	1.243	24.340	6.643	0.949	-5.105	8.721	1.246	24.580	0.00900	0.00900	1.000	3.803	2.490	2.490	
	8	127.5	0.200	0.265	8.400	10.394	1.237	23.733	8.516	1.014	1.375	10.141	1.207	20.724	0.00900	0.00900	1.000	3.803	2.490	2.490	
	7	133.3	0.236	0.265	9.800	11.557	1.179	17.926	9.753	0.995	-0.475	11.157	1.139	13.848	0.00900	0.00900	1.000	3.803	2.490	2.490	
	6	140	0.271	0.265	11.200	12.983	1.159	15.919	11.239	1.003	0.345	12.427	1.110	10.950	0.00900	0.00900	1.000	3.803	2.490	2.490	
	5	146.7	0.306	0.265	12.600	14.489	1.150	14.991	12.780	1.014	1.430	13.788	1.094	9.425	0.00900	0.00900	1.000	3.803	2.490	2.490	
	4	153.4	0.336	0.265	14.000	16.068	1.148	14.769	14.374	1.027	2.674	15.346	1.096	9.612	0.00900	0.00900	1.000	3.803	2.490	2.490	
	3	158.3	0.358	0.265	15.400	17.266	1.121	12.110	15.572	1.011	1.114	16.411	1.066	6.567	0.00900	0.00900	1.000	3.803	2.490	2.490	
	2	162.8	0.377	0.265	16.800	18.649	1.110	11.006	16.945	1.009	0.865	17.650	1.051	5.060	0.00900	0.00900	1.000	3.803	2.490	2.490	
	1	168.4	0.394	0.265	18.200	19.838	1.090	8.998	18.117	0.995	-0.455	18.719	1.029	2.850	0.00900	0.00900	1.000	3.803	2.490	2.490	

Table 4:Comparison between the Dimensionless Total Flow Adjustment method(DTFAM) and the Lambert-Myers Weighted Divided Channel Method (WDCM)

	Test	H	H*	S*10⁻³	Qmes(l/s)	Calculated Qcal	Qcal/Qmes	Error%	DTFAM Qd(l/s)	Disc.	Error%	WDCM Qd(l/s)	Disc.	Error%	nc	nf	n*	W*	B/h	b/h	z
Prinos et al (1985) 381 610 381 vw (30,152) 305 material not known		122	0.164	1.000	32.500	44.269	1.362	36.211	31.307	0.963	-3.671	41.184	1.267	26.720	0.0110	0.0110	1.000	2.701	5.986	4.990	0.500
		122	0.164	1.000	30.200	43.600	1.444	44.371	26.307	0.871	-12.890	38.959	1.323	32.315	0.0110	0.0140	1.273	2.701	5.986	4.986	0.500
Knight & Demetriou (1983)		100.3	0.242	0.966	7.300	7.313	1.002	0.181	6.338	0.868	-13.162	6.988	0.957	-4.280	0.0099	0.0099	1.000	2.000	2.000	2.000	
		113.5	0.330	0.966	9.450	9.533	1.009	0.877	8.506	0.900	-9.997	9.252	0.979	-2.097	0.0099	0.0099	1.000	2.000	2.000	2.000	
		125.8	0.396	0.966	11.700	11.793	1.008	0.796	10.629	0.908	-9.152	11.595	0.991	-0.894	0.0099	0.0099	1.000	2.000	2.000	2.000	
		140.8	0.493	0.966	17.100	16.654	0.974	-2.607	15.003	0.877	-12.262	16.639	0.973	-2.695	0.0099	0.0099	1.000	2.000	2.000	2.000	
		87.5	0.131	0.966	5.000	5.710	1.142	14.209	4.319	0.864	-13.614	5.425	1.085	8.511	0.0099	0.0099	1.000	3.000	2.000	2.000	Rect. z=0
		96.1	0.200	0.966	6.700	7.363	1.099	9.901	6.202	0.926	-7.429	7.013	1.047	4.668	0.0099	0.0099	1.000	3.000	2.000	2.000	
variable (304,456,610) 76 152		113.1	0.322	0.966	11.150	11.099	0.995	-0.461	10.160	0.912	-8.905	10.807	0.969	-3.076	0.0099	0.0099	1.000	3.000	2.000	2.000	
		125	0.392	0.966	15.150	14.582	0.963	-3.663	13.742	0.907	-9.291	14.454	0.954	-4.595	0.0099	0.0099	1.000	3.000	2.000	2.000	
		148.2	0.491	0.966	23.400	22.056	0.943	-5.742	21.196	0.906	-9.419	22.341	0.955	-4.527	0.0099	0.0099	1.000	3.000	2.000	2.000	
		102.1	0.256	0.966	9.100	9.799	1.076	7.669	8.677	0.950	-4.652	9.427	1.036	3.590	0.0099	0.0099	1.000	4.000	2.000	2.000	
		114.2	0.335	0.966	13.500	13.722	1.016	1.643	12.865	0.952	-4.700	13.453	0.996	-0.351	0.0099	0.0099	1.000	4.000	2.000	2.000	
Perspex		127.4	0.403	0.966	18.000	18.601	1.033	3.337	17.906	0.996	-0.520	18.534	1.030	2.966	0.0099	0.0099	1.000	4.000	2.000	2.000	
		153.8	0.506	0.966	29.400	30.058	1.022	2.238	29.532	1.004	0.448	30.608	1.041	4.108	0.0099	0.0099	1.000	4.000	2.000	2.000	
Wormleaton, Allen & Hadjipanos (1982)	B2	160	0.250	0.430	20.500	23.780	1.159	15.902	18.316	0.893	-10.652	22.129	1.079	7.945	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B3	170	0.294	0.430	26.000	28.247	1.086	8.642	23.285	0.896	-10.441	26.426	1.016	1.640	0.01070	0.01400	1.308	4.175	2.417	2.417	Rect. z=0
	B4	180	0.333	0.430	31.000	33.122	1.068	6.846	28.681	0.925	-7.479	31.165	1.005	0.532	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B5	190	0.368	0.430	38.000	36.351	1.009	0.924	34.472	0.907	-9.284	36.301	0.956	-4.471	0.01070	0.01400	1.308	4.175	2.417	2.417	
	B6	210	0.429	0.430	48.000	49.799	1.037	3.665	47.127	0.982	-1.819	47.627	0.992	-0.776	0.01070	0.01400	1.308	4.175	2.417	2.417	
	C3	160	0.250	0.430	18.000	22.690	1.261	26.055	15.806	0.878	-12.191	20.757	1.153	15.314	0.01070	0.01700	1.589	4.175	2.417	2.417	
460 460 120 230	C4	170	0.294	0.430	22.500	26.724	1.188	18.772	20.270	0.901	-9.908	24.462	1.087	8.720	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C5	180	0.333	0.430	28.000	31.065	1.110	11.016	25.161	0.898	-10.136	28.537	1.019	1.920	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C6	190	0.368	0.430	30.500	35.750	1.172	17.212	30.449	0.998	-0.172	32.947	1.080	8.022	0.01070	0.01700	1.589	4.175	2.417	2.417	
	C7	200	0.400	0.430	37.000	40.695	1.100	9.986	36.101	0.976	-2.429	37.660	1.018	1.768	0.01070	0.01700	1.589	4.175	2.417	2.417	
smooth concrete blocks	C8	210	0.429	0.430	43.000	45.901	1.067	6.746	42.097	0.979	-2.099	42.652	0.992	-0.809	0.01070	0.01700	1.589	4.175	2.417	2.417	
	D3	160	0.250	0.430	15.000	21.780	1.450	44.999	13.361	0.891	-10.925	19.537	1.302	30.246	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D4	170	0.294	0.430	17.500	25.370	1.450	44.969	17.319	0.990	-1.036	22.716	1.298	29.805	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D5	180	0.333	0.430	24.000	29.273	1.220	21.972	21.696	0.904	-9.601	26.202	1.092	9.175	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D6	190	0.368	0.430	27.500	33.437	1.216	21.590	26.471	0.963	-3.744	29.965	1.090	8.963	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D7	200	0.400	0.430	33.000	37.642	1.141	14.673	31.609	0.958	-4.214	33.981	1.030	2.973	0.01070	0.02100	1.963	4.175	2.417	2.417	
	D8	210	0.429	0.430	38.000	42.471	1.118	11.766	37.090	0.976	-2.394	38.230	1.006	0.606	0.01070	0.02100	1.963	4.175	2.417	2.417	
Noutsopoulos & Hadjipanos (1983)	A11	92.25	0.187	1.500	9.000	10.585	1.176	17.609	8.722	0.969	-3.094	10.153	1.128	12.816	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A12	107.25	0.301	1.500	18.000	19.105	1.061	6.139	17.931	0.996	-0.386	18.709	1.039	3.940	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A13	119.25	0.371	1.500	27.000	27.633	1.023	2.346	26.837	0.994	-0.604	27.441	1.016	1.633	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A14	122.25	0.387	1.500	30.000	29.971	0.999	-0.096	29.250	0.975	-2.498	29.849	0.995	-0.504	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A15	136.75	0.459	1.500	45.000	44.121	0.980	-1.953	43.720	0.972	-2.844	44.503	0.989	-1.104	0.01000	0.01000	1.000	6.667	2.000	2.000	
	A21	105.00	0.286	1.500	15.000	15.427	1.028	2.850	14.190	0.946	-5.401	15.013	1.001	0.088	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A22	111.00	0.324	1.500	18.000	18.595	1.033	3.306	17.545	0.975	-2.529	18.249	1.014	1.386	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A23	119.25	0.371	1.500	23.000	23.373	1.016	1.621	22.532	0.980	-2.033	23.173	1.008	0.753	0.01000	0.01000	1.000	5.333	2.000	2.000	
variable (1000,800,600)	A24	129.75	0.422	1.500	30.000	30.084	1.003	0.280	29.447	0.982	-1.842	30.143	1.005	0.478	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A25	138.75	0.459	1.500	36.000	36.339	1.009	0.938	35.832	0.995	-0.466	36.673	1.019	1.869	0.01000	0.01000	1.000	5.333	2.000	2.000	Rect. z=0
75 190	A26	141.00	0.469	1.500	37.000	37.968	1.026	2.615	37.489	1.013	1.322	38.378	1.037	3.725	0.01000	0.01000	1.000	5.333	2.000	2.000	
	A41	99.75	0.246	1.500	10.000	11.269	1.127	12.690	9.914	0.991	-0.861	10.835	1.084	8.350	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A42	114.75	0.346	1.500	15.000	17.156	1.144	14.376	16.161	1.077	7.742	16.866	1.124	12.439	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A43	120.00	0.375	1.500	18.000	19.471	1.082	8.173	18.564	1.031	3.134	19.273	1.071	7.070	0.01000	0.01000	1.000	4.000	2.000	2.000	
smooth concrete blocks	A44	134.25	0.441	1.500	25.000	26.311	1.052	5.246	25.574	1.023	2.296	26.442	1.058	5.768	0.01000	0.01000	1.000	4.000	2.000	2.000	
	A45	144.00	0.479	1.500	30.000	31.405	1.047	4.683	30.738	1.025	2.461	31.813	1.060	6.043	0.01000	0.01000	1.000	4.000	2.000	2.000	
Myers (1978)	10	111.6	0.098	0.266	6.300	7.615	1.209	20.874	5.330	0.846	-15.397	7.282	1.156	15.580	0.00980	0.00980	1.000	3.803	2.490	2.490	
	9	118.3	0.138	0.266	7.000	8.704	1.243	24.340	6.643	0.949	-5.105	8.240	1.177	17.712	0.00980	0.00980	1.000	3.803	2.490	2.490	
	8	127.5	0.200	0.266	8.400	10.384	1.237	23.733	8.516	1.014	1.375	9.834	1.171	17.065	0.00980	0.00980	1.000	3.803	2.490	2.490	
356	7	133.3	0.235	0.266	9.800	11.567	1.179	17.906	9.753	0.996	-0.475	10.974	1.120	11.976	0.00980	0.00980	1.000	3.803	2.490	2.490	
	6	140	0.271	0.266	11.200	12.983	1.159	15.919	11.239	1.003	0.345	12.401	1.107	10.725	0.00980	0.00980	1.000	3.803	2.490	2.490	
102	5	146.7	0.305	0.266	12.600	14.489	1.150	14.991	12.780	1.014	1.430	13.933	1.106	10.580	0.00980	0.00980	1.000	3.803	2.490	2.490	
254	4	153.4	0.335	0.266	14.000	16.069	1.148	14.789	14.374	1.027	2.674	15.556	1.111	11.125	0.00980	0.00980	1.000	3.803	2.490	2.490	Rect. z=0
	3	158.3	0.366	0.266	15.400	17.265	1.121	12.110	15.572	1.011	1.114	16.799	1.091	9.082	0.00980	0.00980	1.000	3.803	2.490	2.490	
Perspex	2	163.6	0.377	0.266	16.800	18.649	1.110	11.006	16.935	1.008	0.805	18.241	1.086	8.576	0.00980	0.00980	1.000	3.803	2.490	2.490	
	1	168.4	0.394	0.266	18.200	19.838	1.090	8.990	18.117	0.995	-0.455	19.484	1.071	7.053	0.00980	0.00980	1.000	3.803	2.490	2.490	

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Trader opinion about consumer acceptance of agricultural products irrigated with low quality water-case from Jordan

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TRADER OPINION ABOUT CONSUMER ACCEPTANCE OF AGRICULTURAL PRODUCTS IRRIGATED WITH LOW QUALITY WATER-CASE FROM JORDAN

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ABSTRACT

The interest in wastewater treatment as a reliable source of water has increased in Arab regions due to population growth, as wastewater treatment was considered as a solution to the environmental problems caused by the flow of the wastewater and concurrently as a solution for water availability. The use of treated wastewater in irrigation has different social relevance. If the re-use of water were economically and technically feasible, what would the consequences on the consumer side be and what is the consumer response and acceptance of the products that have been irrigated with this water? This research deals with consumer reaction to farm products produced with different types of water. The overall objective of this study is to investigate the final chain of using re-used water in agriculture focusing on the socio-economic aspects of the people's attitudes towards applying treated waste water and consumption of products produced with low quality water. The information in this study was collected from traders about consumer behaviour at markets by using structured questionnaires at four markets in Jordan in the region. The final results of this study can be used by other countries in Arab regions which have similar conditions of water scarcity and also for those countries that started to use treated water in order to avoid the negative social impact of using this water. The results show there are differences in consumer acceptance of products irrigated with low quality water when compared to the traders view. The effect of price is very high in some markets from the point of view of the trader regarding the level of income of the consumer at these markets. The quality of the products is also important in increasing the acceptance of these products by the consumer. However, the effect of these factors is different from urban to rural markets. In the markets where income is very low, the acceptance is very high if the price is lower than the other products and the acceptance increases at an increasing rate as the price of these products decreases. But in other markets, such as the West Amman Market, quality (high level of income) is more important than the price of the products.

Introduction

In the last few years, the shortage in fresh water in urban and rural areas in the Arab region has increased, since water, as a natural resource is limited. The demand for water has increased steadily over the years while the supply of water has become quite limited. The competitors for fresh water have different economic, social and political relevance. With increasing demand in domestic water use because of increasing population, tourism and individual needs, the domestic and tourism sector will require more water in the future. Intensification in agriculture is also based mainly on an increase in water use. The re-use of water could be a strategy to increase water based on the same natural water capacity. High quality water for irrigation should be limited and reallocated to the different sectors.

A more comprehensive view is needed to evaluate and assess the different strategies in re-used wastewater, where economic, social and environmental aspects have to be included. From the farmer point of view, there is a desire to decrease the cost of production and increase profits in order to improve living standards. What is economically viable for the farmer, and what investment can they afford? Better knowledge and a more comprehensive understanding are needed about people's perception of the re-use of wastewater, and their cultural, ethic and social reservations. If the re-use of water is economically and technically viable, what will be the consequence on the farmer and consumer side? What is the consumer response and acceptance of the products that have been irrigated with this water?

The overall objective of this study is to investigate the final chain of using re-used water in agriculture. This chain starts from the treating of water in a treatment plant, using treated wastewater on the farm and the consumption of the farm products. This research focuses on the socio-economic aspects of the people's attitudes towards applying wastewater and consumption of products produced with low quality water. The specific objectives are to investigate consumer knowledge regarding residues and reactions to farm products produced with different types of water and how far the price and the quality of products affects the increase of the acceptance of these products by the consumer according to trader experience in urban and rural markets. The advantage of this analysis is that the trader is considered here as a key person, which means one trader has his own experience with many consumers. His experience reflects the reality of the consumer reaction in purchasing different quality products with different prices. At the same time, the trader can estimate consumer willingness to buy products irrigated with low quality water.

The interviews with traders were done to evaluate consumer perception and preferences in purchasing products that have been irrigated with low quality water in different rural and urban markets. A minimum of six traders at each market were interviewed, a total of twenty-six traders. This analysis was in four different markets in Jordan; two are rural markets and the other two urban:

- Two main markets in Amman city were used as the urban markets; East and West Amman with different levels of income; high level income and low level of income. These markets are far from any treatment plant, and agricultural activities are not the main economic activity.
- The Deir-Alla market was used as a rural market as it is in an agricultural production area in the middle of the Jordan Valley.
- The Al-Hashemeia market was used as a rural market which is near the Al-Samra treatment plant.

The random selection approach of interviewed traders was attempted as much as possible. Restrictions with regards to the theoretical concept and requirements of random sampling, however, arose from the selection of traders. Not all of the selected traders agreed to interviews and replacement of previously selected traders occurred frequently. Here an uncontrollable bias may occur, but was judged to be minor enough to keep the results valid. The justification of this judgement is the fact that refusal to participate in the survey was not related to the information content of the interviews. Thus, the missing information due to total non-response can be regarded as "missing at random" and allows for bias-free replacement (LITTLE, 1987).

The situation of using non-conventional water in Arab regions

It is estimated that the current use of non-conventional water in the Arab Countries is about 7.5 billion cubic meters annually. It represents about 3% of the 247.5 billion m^3/ year, total Arab water resources.

Jordan water is moving toward the use of non-conventional water to meet the high water demand. Non-conventional water in Jordan includes the use of treated brackish water, and the use of sewage water which is gaining a high momentum in that country.

In Bahrain desalination is the main source of non-conventional water. It is being mixed with low saline water to meet high water requirements. It is estimated that the total irrigated area from sewage water in Bahrain is 665 hectares.

The reuse of treated sewage water is the main source of non-conventional water in Tunisia; its volume is estimated at about 150 million cubic meters annually.

In Algeria, desalination started in 1975. The present volume of the sewage and industrial drainage is expected to reach 1000 million by the year 2020.

The use of non-conventional water in Sudan is very limited. There is only one small desalination unit in Port Sudan, and there is no sewage system except in parts of the Capital Khartoum.

In Syria, the use of non-conventional water is increasing to meet the water shortage. Desalination is now in the planning stage. There is increasing demand for the reuse of wastewater and especially reuse of agriculture drainage water and there are many activities in that direction.

Saudi Arabia is a pioneer in the desalination of sea water, which is the main source of its potable water. Its use in irrigation is very limited. Regarding sewage, it used to be drained into open wells before the construction of collecting networks. It is treated before being used in limited irrigation.

Iraq's conventional water resources are very sensitive to the stand of other riparian countries who control the up stream reaches of the international rivers and for that reason non-conventional water is one of Iraq other possible options, including brackish water, desalination, and reuse of drained waste water.

In Palestine, Israel is controlling all conventional water sources and, this has led Palestine to give more attention to the use of non-conventional water. Sewage water is considered the main non-conventional source of water in Palestine. This type of water is on a continuous increase, but due to lack of adequate facilities and manpower, the use is not optimum.

Kuwait was the first Arab country to rely basically on desalination as a source of drinking water. The production raised from 4 million cubic meters per day in 1959 to 282 million in 1998. Some treated sewage water is being used in agriculture. The amount of treated sewage increased from 55 million gallons per day in 1989 to 90 million in 1999 and is expected to reach 140 million in 2015.

The total volume of Cairo sewage water is about 4 million cubic meters daily and one million in Alexandria, and Egypt's total volume of sewage water could reach 7.5 million if collecting networks are made in other cities. At present, part of this water is used to irrigate 150000 feddans in the desert, with no recorded bad effect on soils.

Egypt is the Arab pioneer in the reuse of agricultural drained water with an estimate at 4.7 billion cubic meters annually and it is expected to reach 7 billion. This practice is mainly in the delta region.

The estimated volume of desalination water in Morocco is about 8680 cubic meters daily, used mainly for drinking. Sewage water in Morocco is about 500 million cubic meters annually; only 60 million are used to irrigate 7000 hectares. The volume of this water is expected to reach 900 millions in 2021. There is no reuse of industrial drainage in agriculture in Morocco.

The main activities of the people in Mauritania are trade and ranching. Mauritania has a reliable source of conventional water from the Senegal River. Its present use of non-conventional water is limited, but there is a growing interest to develop non-conventional water sources. Sewage drained water in the Capital is being used to irrigate public green lawns.

In Yemen, agriculture is the backbone of the economy of the country and it employs 70% of the population. Due to the big gap between demand and the available conventional water, Yemen is increasing its interest in non-conventional water. It is the first Arab country to use desalinated water. The first Arab desalination station was constructed in Aden in 1869. There are no reliable records about industrial drained water but its volume is estimated to be 80 million m^3/year. Sewage water is estimated to be 74 million cubic meters annually, and it is expected to reach 155 million in 2010.

The Methodology of the research

The weight of the traders

The interviews were done with different traders. Some of them own a big shop and others small ones. The weight of the trader was considered high in this analysis if he had a large number of the customers. This study assumed that the opinion of the trader reflects the behaviour of customers who buy from him. According to this assumption, the weight index of each trader was calculated as follows:

$$WT = (NC/TC) * 100\% \qquad (1)$$

WT: Weight index of the trader.
NC: The average number of customers per day for each interviewed trader in each market.
TC: The total number of customers per day for all interviewed traders in each market.

Table 1 shows the number of interviewed traders in each market and the weight of each trader. According to this assumption, the weight of traders is between 5% and reaches 50%. The analysis of consumer behaviour in the following parts depends on the weight of the trader.

Table 1: The number of customers and the weight of each interviewed trader in each market, Jordan 2000

The no. of trader	Al-Hashemeia Market		East Amman Market		West Amman Market		Deir-Alla Market	
	Number of customers	Weight index of trader	Number of customers	Weight index of trader	Number of customers	Weight index of trader	Number of customers	Weight index of trader
Trader 1	150	41%	70	7%	120	12%	65	19%
Trader 2	80	22%	200	19%	50	5%	40	12%
Trader 3	20	5%	150	15%	150	15%	50	15%
Trader 4	30	8%	100	10%	500	50%	20	6%
Trader 5	60	16%	60	6%	100	10%	70	21%
Trader 6	30	8%	200	19%	80	8%	50	15%
Trader 7			250	24%			40	12%
Total	**370**	**100%**	**1030**	**100%**	**1000**	**100%**	**335**	**100%**

Trader opinion about consumer willingness to buy products irrigated with low quality water

The discussion now is concerned with trader opinion of the consumer acceptance of buying products irrigated with low quality water if he does not find substitute products at the market. The estimate of the trader regarding the percent of consumers who accept these products is different from one trader to the other. Some of them expect all the consumers would accept them, others expect 75% of consumers, while the others expect 25% or that none of the consumers would accept them. In this trader interview, the sum of the weighted traders for each expected percent of the consumers who would accept these products and the total acceptance were calculated as shown in Table 2.

The opinion of the trader is considered here as a summation of the weight of traders who had the same opinion of the percent of the consumers who would buy the products irrigated with low quality water. The total percent of the consumers in this case was calculated depending on the following formula:

$$TA = \Sigma(EC*OT) \qquad (2)$$

TA: The total expected percent of the consumers who would buy products irrigated with low quality water.
EC: The expected % of the consumers who would buy products irrigated with low quality water.
OT: The opinion of the traders about the percent of consumer acceptance of fruits and vegetables irrigated with low quality water.

The traders at different markets expected that there would be no difference in the acceptance of vegetables and fruits in each market if they were irrigated with low quality water, but it differed from one market to another as shown in Table 2. The acceptance in East Amman was more than West Amman because the difference in the level of income was high. In West Amman where the income is high compared with that in East Amman, if the consumers do not find the high quality of products they would buy from other markets even if the price was high. This result is the same in the consumer survey.

The acceptance in Al-Hashemeia Market was low and in Deir-Alla high although the results in the consumer survey were different. In this case, it was high in Al-Hashemeia compared to Deir-Alla. The reason behind that is that many of the consumers in these two areas were farmers. In Al-Hashemeia, they used treated wastewater while in Deir-Alla they did not, so when they were asked about their willingness to buy products, each group answered in a way that supported the quality of their products. However, the answer from the trader was from his own experience regarding the behaviour of the consumers.

In this section, the price of products was considered in the analysis of the behaviour of the consumer and to what extent the price influenced consumer acceptance of products irrigated with low quality water. The analysis used two cases:

- The quality of products irrigated with low quality water was the same as the others.
- The quality of products irrigated with low quality water was worse than the others.

The analyses for these two cases were in each market, and for each case in all the markets to compare the differences in the behaviour of the consumer in different markets.

Table 2: The opinion of the trader about the consumer acceptance of fruits and vegetables irrigated with low quality water in different markets, Jordan 2000

	Al-Hashemeia Market		**East Amman Market**		**West Amman Market**		**Deir-Alla Market**	
	Opinion of the trader (OP)		**Opinion of the trader (OP)**		**Opinion of the trader (OP)**		**Opinion of the trader (OP)**	
Expected the % of consumer (EC)	**Fruit**	**Vegetable**	**Fruit**	**Vegetable**	**Fruit**	**Vegetable**	**Fruit**	**Vegetable**
100%	0%	0%	45%	30%	20%	20%	52%	52%
75%	16%	16%	19%	53%	0%	0%	33%	33%
50%	0%	0%	29%	0%	15%	15%	0%	15%
25%	57%	57%	7%	17%	15%	15%	15%	0%
0%	27%	27%	0%	0%	50%	50%	0%	0%
The total acceptanc e (TA)	**26%**	**26%**	**75%**	**74%**	**31%**	**31%**	**81%**	**84%**

Trader opinion of the effect of price on the consumer acceptance of products irrigated with low quality water

West Amman Market

The opinion of the traders in the West Amman Market is that consumer acceptance of products irrigated with low quality water is low. Not more than 35% of consumers would accept them, even if the quality of these products was the same as the other products and the price was 50% less as shown in Figure 1. Not more than 15% of consumers would accept these products if the quality were worse than the other products even if the price was 50% less.

The percent of consumers who would accept these products was between 0%-15% if the quality were worse, and 20%-35% if the quality were the same as the other products. The difference between the ranges in each case is 15%, but the difference between the two cases in each price is about 20%. This means that the quality of products is more important than the price at this market. The improvement of the product quality was more important to increase consumer acceptance of these products than the decrease in price.

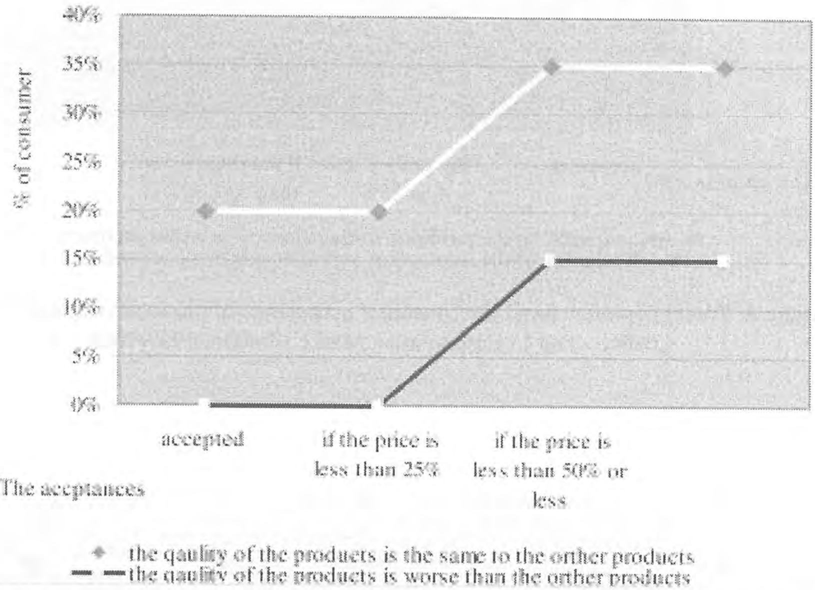

Figure 1: Trader opinion of consumer acceptance of products irrigated with low quality water, in West Amman Market, Jordan, 1999/2000

East Amman Market

The acceptance of these products in this market reached 100% if the difference in price between these products and the others was high (50% or more). In this market, the most important factor influencing consumer acceptance of products irrigated with low quality water was the price, which is more important than the quality of products (Figure 2).

Al-Hashemeia Market

This market is near the As-Samra Treatment Plant and is a rural market. At this market, the acceptance of products irrigated with low quality water was high if the price was lower than the other products. It reached 100% of the consumers according to the opinion of the trader. The effect of the quality of the products was very low on consumer acceptance of these products as shown in Figure 3. The difference in the percent of consumers who accepted these products if the quality were better but there was no difference in the price was about 20%. The acceptance of these products, if the price was less than the other products and the quality was worse, is the same as if the price were less and the quality were the same as other products.

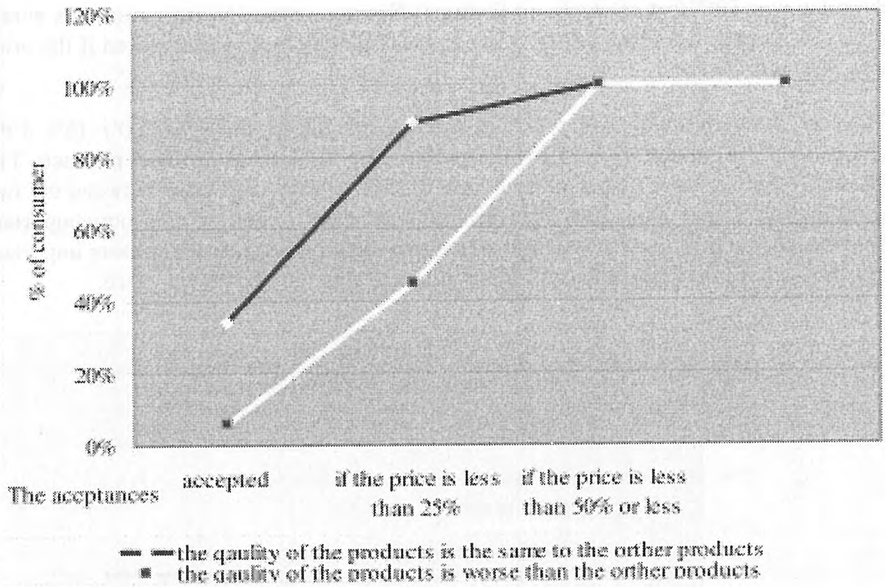

Figure 2: Traders opinion about the consumer acceptance of products irrigated with low quality water in East Amman Market, Jordan, 1999/2000

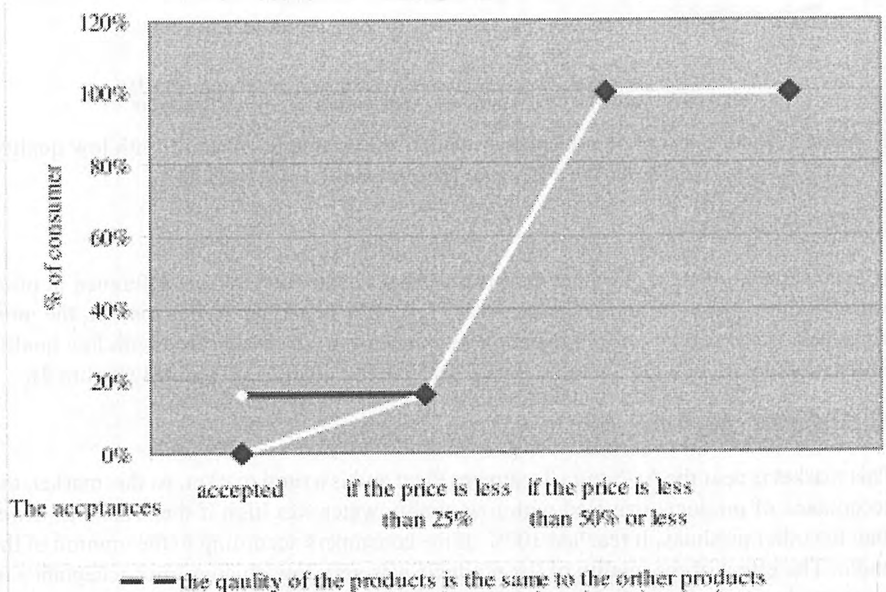

Figure 3: Traders opinion regarding the consumer acceptance of products irrigated with low quality water in Al-Hashemeia Market, Jordan, 1999/2000

Deir-Alla Market

This market is a rural market in an agricultural area. The effect of the quality of the products is very high in this market if there is no difference in the price of different products. As shown in Figure 4, at this market the quality and the price of the products are both important in influencing consumers acceptance of products irrigated with low quality water. As the price became lower the acceptance became greater. The percent of consumers who accepted these products is 10% but if the difference in the price were 50% less than the other products, then the percent of consumers would be about 70%. The percent of consumers would be 80% if the quality of these products were the same as the other products even if the prices are the same.

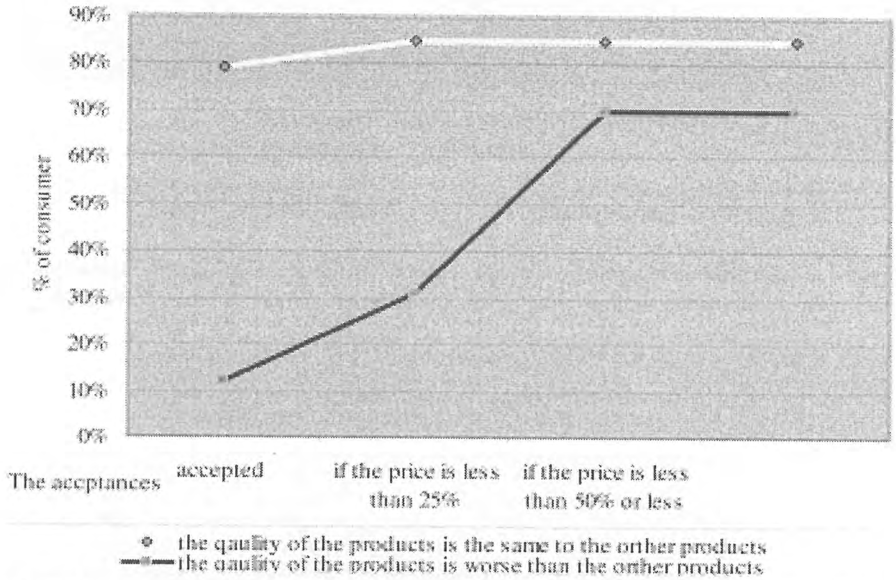

Figure 4: Trader opinion about the consumer acceptance of products irrigated with low quality water in Deir-Alla Market, Jordan, 1999/2000

The difference in consumer acceptance at different markets

Figure 5 shows the opinion of the traders regarding consumer acceptance of products irrigated with low quality water at different markets if the quality of these products were the same as the other products. The West Amman Market has the lowest percent of the consumers who accept these products compared to others. In the other three markets, the acceptance is very high if the price of these products were 50% less than the other products. In this case, the highest acceptance is in Deir-Alla Market even if there is no difference in the price of different products.

Figure 6 shows the percent of consumers who would accept these products if the quality of these products were worse than the others with different levels of price. In this case, the lowest acceptance is in West Amman Market and the highest percent in East Amman Market. In the other two markets, the acceptance is very low if there is no difference in the price of different products, this percent increased at an increasing rate in these two markets as the difference in the price of different products increased.

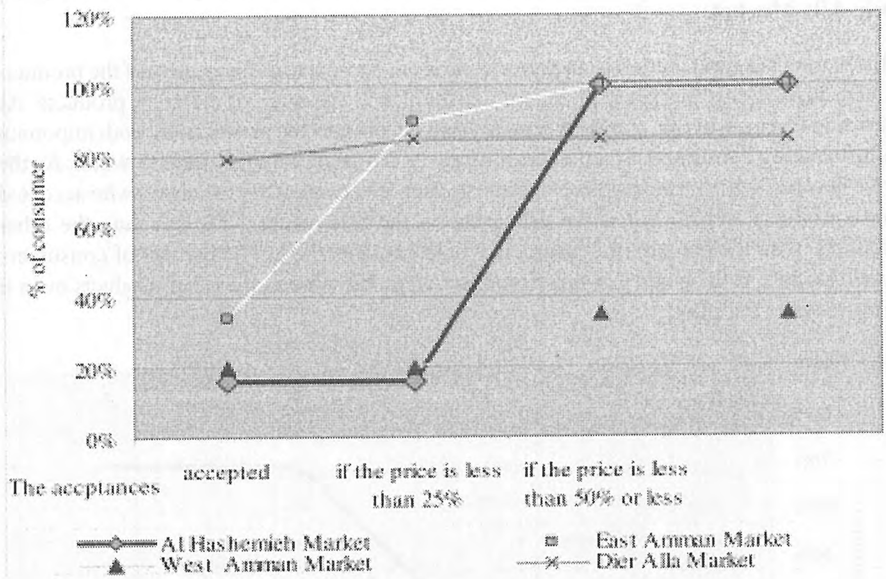

Figure 5: Trader opinion regarding consumer acceptance of products irrigated with low quality water if the quality is the same to other product in different markets, Jordan, 1999/2000

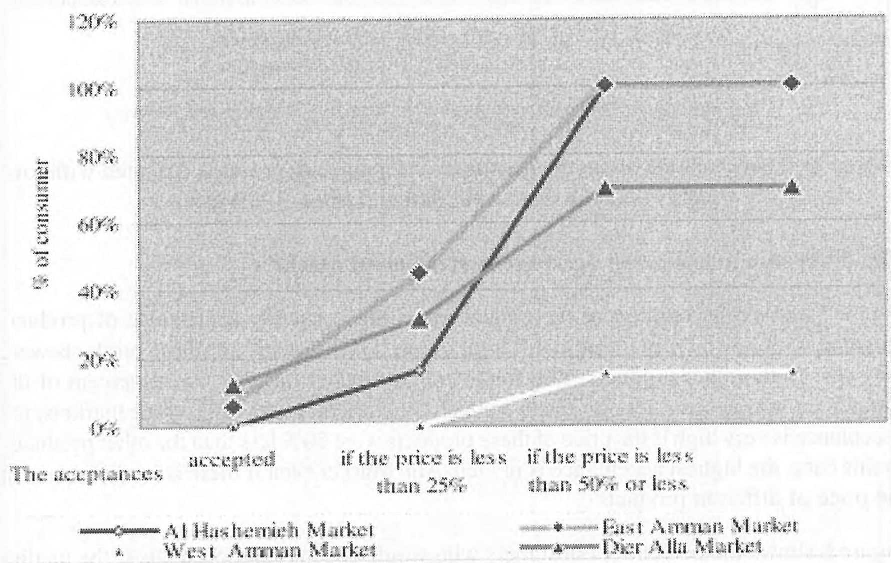

Figure 6: Trader opinion regarding the consumers' acceptance of products irrigated with low quality water if the quality were worse than the other products in different markets, Jordan, 1999/2000.

Results and conclusions

The major findings of this study can be summarized as follows:

- Results show that consumer acceptance and knowledge about low quality water for irrigation and its effect on the products differ between markets and with regard to products.
- The consumer has the flexibility to buy these products. This flexibility and the acceptance are different from one product to another and from one market to another.
- The relation between acceptance and price shows the effect that, with decreasing prices, products produced with low quality water are increasingly accepted. The high price elasticity of low income classes is not reflected when income is high. Here, higher prices are accepted for better quality.
- The quality of products affect consumer opinion in accepting the products irrigated with low quality water. Still, however, efforts are needed to increase awareness about this water and to create trust that it is safe when used for irrigation.
- Which products and where they can be used should be taken into consideration before applying any policy related to using low quality water in agriculture.

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7. المنظمة العربية للتنمية الزراعية ٢٠٠١، دراسة تقويم الآثار المترتبة على سوء استخدام الموارد المائية غير التقليدية على البيئة الزراعية العربية

الملخص

نتيجة للتزايد السكاني في الوطن العربي ومحدودية الموارد المائية فيه فقد تزايد الاهتمام بمياه الصرف الصحي المعالجة كمصدر للمياه يمكن أن يعتمد عليه في أقطار الوطن العربي وبخاصة في القطاع الزراعي وذلك لحل مشكلة المياه من جهة والمشاكل البيئية الناتجة من مياه الصرف الصحي من جهة أخرى. لكن استخدام هذه المياه قد يكون له آثار اجتماعية، فإذا كانت هذه المياه اقتصادياً مقبولة لدى المزارع وفنياً يمكن تطبيقها فيجب معرفة مدى تقبلها اجتماعياً وبيئياً، فالحلقة الأخيرة للسلعة الزراعية هي المستهلك، فما هي وجهة نظر المستهلك في شراء المنتجات الزراعية التي تم ريها بالمياه المعالجة؟ وما مدى استعداده لشراء هذه المنتجات؟ وما هي العوامل التي قد تزيد من تقبله لهذه المنتجات؟ لأنه إذا رفض شراء هذه المنتجات فهذا يعني انه لا جدوى من معالجة هذه المياه بهدف استخدامها للزراعة. لذلك يأتي الهدف العام لهذا البحث هو معرفة وجهة نظر المستهلك في قبول المنتجات التي تم ريها بمياه الصرف الصحي المعالجة، وما هي العوامل التي قد تزيد من تقبله لها، وتم هذا البحث من خلال مقابلة تجار وتعبئة استمارات منهم في عدة أسواق مختلفة في الأردن، ونتائج هذه الدراسة ممكن أن تعتمد ويستفاد منها في دول المنطقة والتي تعاني من مشكلة المياه وتلك الدول التي بدأت باستخدام معالجة مياه الصرف الصحي. وقد أظهرت نتائج هذه الدراسة أن تقبل المزارع لهذه المنتجات الزراعية المروية بالمياه المكررة تختلف حسب السوق إن كان في منطقة ريفية أو منطقة حضرية، كذلك فنوعية المحصول وسعره يلعبان دوراً مهماً في تقليل تلك المنتجات وتأثير هذه العوامل يختلف كذلك حسب السوق ومستوى الدخل للمستهلك. فسعر المنتجات الزراعية المروية بمياه الصرف الزراعي المعالجة له أثر كبير في تقبل شرائها من قبل المستهلك وخاصة ذوي الدخل المنخفض، بينما أظهرت نتائج هذه الدراسة أن نوعية المنتج لها الأثر الأكبر في تقبل هذه المنتجات من قبل المستهلك مقارنة بسعرها إذا كان من ذوي الدخل المرتفع.

Georeferencing irrigation and drainage networks using advanced spatial technology

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GEOREFERENCING IRRIGATION AND DRAINAGE NETWORKS USING ADVANCED SPATIAL TECHNOLOGY

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ABSTRACT

The locations of irrigation and drainage canals are, often, determined by visual estimations obtained from a base map for the project area. Occasionally, the water manager may need to use a map that contains little or no coordinate information. This is a problem if he intends to project this map with another in a multi-layer database. In this study we used the coordinates of selected ground control points (GCPs) obtained with a Trimble GPS unit to transform (georeferencing) the network base map (7 sheets dated 1964; 1:10000-scale; unknown coordinate system) digitized in units of the base map into a metric UTM system. Registration was done using affine transformation in Arc/Info GIS. The accuracy of this procedure was analyzed by comparing the GPS positions of 40.72 km one-way-roads with their corresponding GIS transformed positions. A 40-m buffer was required for at least 96% of the GIS roads to occupy the same GPS road positions. The georeferenced network map was successfully updated by map-to-map registration and involved a residential area map developed from a recent SPOT image and a soil salinity map developed using variogram and kriging analysis with a scale of 1:40000 as an upper limit. Errors in the GPS measurements, digitizing, marking GCPs on the base map, and the transformation numerical method represented 3, 5, 25, and 67%, respectively, of the total RMS error (20 m) introduced in the estimation of network positions.

Keywords: Geographical information system; Global position system; SPOT imagery; Map registration; Soil salinity

INTRODUCTION

Modern management of irrigation and drainage projects, network analysis, and conservation of resources require monitoring the soil-plant-atmosphere continuum. The resulting spatial and temporal data could be the input for a geographical information system (GIS) with a network capability (Burrough, 1987; Zhang et al., 1996; zhou et al., 1996). Occasionally, the water manager may need to use a map for the irrigation and drainage networks that contains little or no coordinate information. This is a problem if he intends to register this map with another in a georeferenced multi-layer database. Georeferencing refers to the process of assigning a map coordinate system to map data. Map-to-map registration involves georeferencing only if the reference map is already georeferenced (ERDAS, 1982). In this case the global positioning system (GPS) (Hofmann-Wellenhof et al., 1994) together with geographical information system (GIS) could be used to estimate a coordinate system for an irrigation and drainage project area without coordinate information (Rogowski, 1995; Alsaeedi and Elprince, 2000). GPS positions can be used to verify the accuracy of GIS products by comparing the GPS data to the GIS product in question (Trimble, 1996; ESRI 1994). The earth ellipsoid chosen for the GPS is GRS-80 ellipsoid. The datum incorporating this ellipsoid and used by GPS is called the World Geodetic System 1984 (WGS-84). The formulas that take longitude and latitude (on the spheroid) and convert to X and Y (on a flat surface touching the globe) are called map projection. One common projection is the Universal Transverse Mercator, UTM. Arc/Info supports a total of 46 projections (ESRI, 1994).

The objectives of this study were: (i) to evaluate the uncertainty introduced by georeferencing surface irrigation and drainage networks into a metric UTM system using a digitized base map with an unknown coordinate system, GPS position measurements for selected GCPs, and coordinate transformation in GIS; and (ii) to update the georeferenced network map by map-to-map registration by involving a residential area map developed from recent SPOT image and a soil salinity map developed using variogram and kriging techniques.

MATERIALS AND METHODS

The project area

Al Hassa irrigation and drainage project in Saudi Arabia, is situated some 60 km inland of the Gulf coast between 25° 37' 12'' and 25° 20' 24'' N Lat and 49° 32' 24'' and 49° 46' 48'' E Long, and covers an area of approximately 20,000 ha. Since 1971, the project has been in full operation and is managed by Al-Hassa Irrigation and Drainage Authority (HIDA). The project area is L-shaped and sloped with a very low gradient towards the Gulf coastal plain. Lithostratigraphic sequence at the project area is in Al-Sayari and Zotl (1978). The aridity of the climate is characterized by potential evapotranspiration, which far exceeds precipitation (<100 mm/year) in the region (Hoyningen-Huene, 1979). The soils of the project were divided into six great groups, namely Salorthids, Gypsiorthids, Calciorthids, Psammaquants, Haplargids, and Torripsaments (Elprince, 1985). The total cultivated land within the project is 8000 ha, 92% of which is planted with date palms (Phoenix dactylifera L.) (Elprince et al., 1982). The water used in the project (wells) discharges (13.62 m^3 s^{-1}) from a karistified neogen aquifer belonging to the Umm-er-Raduma formation which occurs at a depth of 280 m with a thickness of approximately 320 meters (Nejm, 1994).

The irrigation network consists of open reinforced concrete main, sub-main, and lateral canals. The laterals are led 150 m apart in such a way that irrigation is done on one side only to minimize land leveling. Water is conveyed from lateral canals to adjacent farms via flexible hose siphons, that can be inserted anywhere. Water is released into canals twice a week in the summer and once a week in the winter. The drainage network is divided into main, sub-main, and lateral canals. The drainage water of the main canals is conveyed to two drainage lakes outside the project area.

The base map, digitizer, GIS, and GPS

The base map for the project area was made of 7 MELAR sheets at 1:10000-scale, dated 1964, and signed by WAKUTI Consultant Engineers, W. Germany. The map showed the roads, railway, drainage and irrigation canals (main, sub-main, and laterals), and other related features (bridges, weirs, bottom falls, sag pipes, elevated reservoir, pump stations, springs,..). The coordinate system could not be identified. The method we had selected for creating the spatial data was digitizing. The digitizer was Calcomp 34480 (size A0; active area: 914 x 1219 mm; outside dimensions: 1276 x 1499 mm). The software used was AUTOCAD (version 14) and the resulting layers (DXF files) were exported to Arc/Info.

GIS used in this study was ARC/INFO (version 7.1.1) and ARCVIEW (Version 3.3). The Trimble GPS system hardware used in this study was made of an 8-channel GPS/MSK Beacon Pro XR receiver, a TDC1 data logger, an Integrated GPS/Beacon Antenna, and Camcorder Batteries (Trimble, 1996). The MSK Beacon configuration contained a fully automatic dual channel MSK radio beacon receiver for receiving DGPS (Differential GPS) broadcasts conforming to the IALA (International Association of Lighthouse Authorities) standard. Our GPS/MSK beacon antenna was in the range of Al-Bahrain DGPS radio beacon transmitter. Al-Bahrain station calculated and broadcasted through radio signals the correction for each satellite as it received the data. This correction was received by the rover (in-Al-Hassa) and applied to the position it was calculating. As a result, the position displayed by the controlling software (Asset Surveyor) and logged to the data file was a real-time differentially corrected position with a typical accuracy of 1-10 meters in real-time (Trimble, 1996). The Trimble GPS system software was: (1) TDC1 Asset Surveyor software (version 3.30) used to navigate and collect GPS field data and (2) Pathfinder Office software used to view, edit, plot and export data to GIS.

GPS measurements

We used selected sites (roads intersections, water reservoirs, springs, and bridges) as ground control points (GCPs), the locations of which were marked directly on the 1:10000 base map. The GPS coordinates of the GCPs were obtained in the field with the Trimble GPS system. Furthermore, GPS road survey field data were collected at a rate of one position every 5 seconds. The 3D real-time radio link differentially corrected data were transformed from the data logger to our office computer using the Data Transfer Utility of Pathfinder Office software. Field data maps were displayed in UTM coordinates, zone North 39, WGS 84 datum and were exported in an Arcview GIS file format (.shp). The 3D real-time DGPS horizontal measurements obtained in this study were under the following conditions: (1) number of satellites used ^{3} 5; (2) PDOP (Position Dilution of Precision) < 6; (3) GPS Signal to Noise Ratio (SNR) > 6; (4) Satellite Elevation Mask: ^{3} 15°; and (5) the DGPS beacon station was in Al-Bahrain at a distance of about 100 km from the study area.

Georeferencing

We used the GPS coordinates of selected GCPs to transform the project map digitized in units of the base map of an unknown coordinate system into the metric UTM system. Transformation was done using affine transformation in ARC/INFO. The affine equations are:

$$X = Ax + By + C \qquad (1)$$

and

$$Y = Dx + Ey + F \qquad (2)$$

Where X and Y are the GPS coordinates and x and y are the digitized base map coordinates. The geometric interpretation of the six affine coefficients includes translation, scale changes, skew and rotation (Hofmann-Wellenhof et al., 1994; ESRI, 1994). A transformation procedure uses the resulting affine coefficients to transform position estimates in base map units into position estimates in decimal degrees using the ground control points as tic points. Decimal degree coverages then are projected into metric UTM (Alsaeedi and Elprince, 2000).

Map to map registration

Spot image acquired in 1998 was used in this study and was supplied to us by the Saudi Center for Remote Sensing, King Abdulaziz City for Science and Technology, Riyadh. Residential areas were digitized using AUTOCAD14. Registration of the residential area map to the georeferenced network map was done using GPS coordinates of seven GCPs (road intersections) and affine-transformation in Arc/Info.

The soil salinity map used in this study was developed from soil salinity data (electrical conductivity of 1:3 soil extracts, $EC_{1:3}$, dS/m) for soil samples (0-20 cm depth) collected at a rate of approximately one composite soil sample per 19 ha. Ordinary kriging seemed the "best" kriging approach when using 12 neighboring points and an exponential variogram model (Elprince et al, 2004). Registration of the soil salinity map to the georeferenced network map was done using GPS coordinates of 20 GCPs and affine-transformation in Arc/Info.

RESULTS AND DISCUSSION

Network statistics

A statistical summary of canal lengths is presented in Table 1. The irrigation network is made of 19 main, 170 sub-main, and 2003 lateral canals; the total lengths of which are 140,841 m, 251,567 m, and 1,024,413 m, respectively. The length of a main irrigation canal (L_{mi}) varies from 2,086 m to 19,885 m. Values of the length of a sub-main irrigation canal (L_{smi}) vary from 133 m to 4,463 m and seem more accurately described as log-normally than normally distributed (Fig.1). As seen from the fractile diagrams in Fig.1, the probability units (U) are linearly related more to ln (L_{smi}) than to L_{smi} and hence the values of L_{smi} seem more accurately described as log-normally than normally distributed. This result is supported by the approximate Wilk-Shapiro correlation coefficient square values 0.8403 for L_{smi} vs U and 0.9887 for ln (L_{smi}) vs U (Table 1). Furthermore, a Kolmogorov-Smirov test (Analytical Software, 1996) showed that the two distributions are indeed different. In contrast with sub-main irrigation canals, the length of a lateral irrigation canal (range: 65m to 1,852 m) is normally distributed (Table 1).

The drainage network is made of 15 main, 170 sub-main, and 1761 lateral canals; the total lengths of which are 119,273 m, 177,058 m, and 927,274 m, respectively (Table 1). In agreement with the irrigation network, while values of the length of a sub-main drain are more accurately described as log-normally than normally distributed, values of the length of a lateral drain are normally distributed (Table 1).

Field estimation of GCPs position

Average position of each GCP has been estimated from a number of individual measurements by the GPS Trimble unit. Longer readings generally decrease standard deviation values, i.e. improved accuracy. On the average we took about 1 minute readings. This gives site location as a median of 6 to 53 observations (Table 2). The recorded positions (point feature) are of sub-meter accuracy (Table 2) and seemed unaffected by most the error sources reported by Rogowski (1995), namely ephemeris, receiver, atmospheric / ionospheric, worst case S/A, and PDOP.

Comparison of GPS measured and GIS affine-transformed roads

For error evaluation we have applied buffers of different sizes for one-way roads (total length: 40.70 km) which were GPS measured and GIS affine transformed. A 40-m buffer is required for at least 96% of the GIS roads to occupy the same GPS road positions (Table 3). Thus any resulting coverage map should be at a lower scale than the base map with a scale of 1:40000 as an upper limit. Furthermore, the RMS total error introduced in the base map due to the GPS position measurements, digitizing and registration procedure (affine transformation using 3D real-time DGPS) is estimated equal to 20 m (= 40/2).

Table 1: Statistics for the irrigation and drainage canals of Al-Hassa project.

		Irrigation canals			**Drainage canals**		
Statistics	**Units**	**Main**	**Sub-main**	**Lateral**	**Main**	**Sub-main**	**Lateral**
N		19	218	2003	15	170	1761
Sum	m	140841	251567	1024413	119273	177058	927274
Mean	m	8285	1154	511	7951	1041	627
SD	m	6311	942	197	10051	777	199
CV	%	76	82	39	126	75	38
Minimum	m	2086	133	65	947	124	85
Median	m	6182	867	504	4152	851	519
Maximum	m	19885	4463	1853	35281	4186	1666
R_w^2 {L vs U}*			0.8403^a	0.9807^a		0.8677^a	0.9765^a
R_w^2 {ln(L) vs U}*			0.9887^b	0.9696^a		0.9878^b	0.9685^a

* If superscript letters are different in a column, the two distributions are different. If letters are in a column, the two distributions are not different according to a Kolmagorove-Smirov test.

Error analysis

Table 4 summarizes the error RMS budget. As previously shown, our GPS position measurements are of sub-meter accuracy (< 0.599 m from Table 2). Thus the GPS-measurement's error represented less than 3% of the total error (Table 4) and GPS could be used with confidence to identify positions inside the irrigation and drainage networks of Al-Hassa project. Furthermore, digitizing introduces an error into the estimation of position. The digitizing RMS error value depends on the nature of the data, the scale of the base map, and the material from which the data is digitized. This study used MYLAR maps, thus we may need to accept an RMS value of 0.1 mm (ESRI, 1994), which represents

1 m on the 1:10 000 scale base map and 5% of total error (Table 4). Another source of error is the visual sitting of GCPs on the base map. Marking a GCP on a map introduces an error into the estimation of position. A point 1-mm in diameter on a 1: 10 000 map represents a circular field area 10-m across and thus a 25% of the total error (Table 4). The rest of the error (67% of total error) seems due to the affine transformation numerical method and any other undefined sources of error (Table 4).

Map-to-map registration

The georeferenced network map has been successfully updated by map-to-map registration and involved residential areas (digitized fro SPOT image) and soil salinity (kriged) maps. For illustration, Fig.2 shows the irrigation network (laterals and labels are deleted for simplicity) together with updated residential areas and soil salinity classes at a scale of 1: 150 000. On the resulting map (Fig.2), $EC_{1:3}$ values were grouped into five classes; namely very low (<0.73 dS/m), low (0.73-1.21 dS/m), medium (1.21-2.44 dS/m), high (2.44-6.63), and very high (>6.63 dS/m). Soils with an $EC_{1:3} > 1.21$ dS/m can, approximately, be considered saline (Bui et al., 1996). Soil salinity distribution seems in agreement with the oasis model (Elprince, 1985; Alsaeedi and Elprince, 1999). Figure 3 shows the irrigation network east of Al-Hofuf city including the mains, sub-mains and laterals at the upper limit scale 1:40 000.

In summary, a geo-referenced multi-coverage database could be created for an irrigation and drainage network area having a map of no coordinate information by employing the GPS/GIS procedure demonstrated above. Error analysis indicated that the procedure accuracy could be greatly improved by improving the coordinate transformation numerical method and reducing the error due to marking the GCPs on the base map. The resulted network map could be updated by map-to-map registration using satellite images and environmental survey data.

Table 2: The ground control points, GCPs and their coordinates on the base map and by GPS.

No.	Ground control point locations	Base map		GPS, Decimal degrees		Query feature	
		x	y	Long.(X)	Lat.(Y)	Positions sec/pos.	St.d.
1	ALHARRA (Water Reser.)	210600	584060	25.41151581	49.60211497	16 (5)	0.114
2	AIN BAHLA	211810	582560	25.39800883	49.61442269	10 (5)	0.132
3	D1.7 (ROAD INT.	212330	580190	25.37641169	49.61985644	10 (5)	0.565
4	D2BD (ROAD INT.)	214075	580290	25.37723994	49.63735911	8 (5)	0.208
5	D1P4M (ROAD INT.)	211200	581970	25.39307086	49.60859947	6 (5)	0.599
6	P4G (ROAD &CANAL INT.	211740	582420	25.39685297	49.61364175	10 (5)	0.137
7	BANI MAN BRIDGE	213380	581950	25.39224128	49.63000589	10 (5)	0.151
8	D1D5 (ROAD INT.)	213650	588530	25.45223419	49.63298308	9 (5)	0.196
9	F2.D1.5 (ROAD INT.)	212960	587530	25.44257789	49.62615958	24 (5)	0.161
10	F1AH (ROAD INT.)	221015	580240	25.37626111	49.70543958	35 (1)	0.118
11	ALAKAR_ROAD INT.	220490	580270	25.37649458	49.70020081	37 (1)	0.202
12	ALMARAH BRIDGE)	207530	605250	25.60302611	49.57264975	53 (1)	0.304
13	ALUYON BRIDGE	207570	603490	25.58705869	49.57267269	50 (1)	0.097
14	D1.D1.4 (ROAD INT.)	211980	593540	25.49672083	49.61695175	43 (1)	0.162
15	F2.D1 (ROAD INT.)	212490	592920	25.49130456	49.62207025	50 (1)	0.095
16	ALK-SHEBAH-D1BRIDGE	212860	590560	25.46977872	49.62548336	50 (1)	0.177
17	D2.D2.3 (ROAD INT.)	223230	584830	25.41792047	49.72797606	51 (1)	0.32
18	D2.D2.2 (ROAD INT.)	225020	584980	25.41960842	49.74581086	50 (1)	0.179
19	D2.2 (ROAD INT)	225040	584960	25.41912311	49.74626058	50 (1)	0.183
20	ER-P1 (WATER RES.)	212920	579475	25.37064281	49.62636356	Manual	

Table 3: Percentage of GPS roads intersected with GIS roads within buffers.

No. GCPs	**Buffer size**		
	30 m	**35 m**	**40 m**
5	77	83	91
9	87	92	97
14	86	92	96
19	86	91	96

Table 4: Error RMS[a] budget of the output map

Source	**Magnitude, m**	**% of RMS error**
GPS position measurements	0.6	3
Digitizing	1	5
Marking GCPs on base map	5	25
Affine transformation and others	13	67
Total	**20**	**100**

[a] RMS is 68% of the time and 2xRMS is 95% of the time

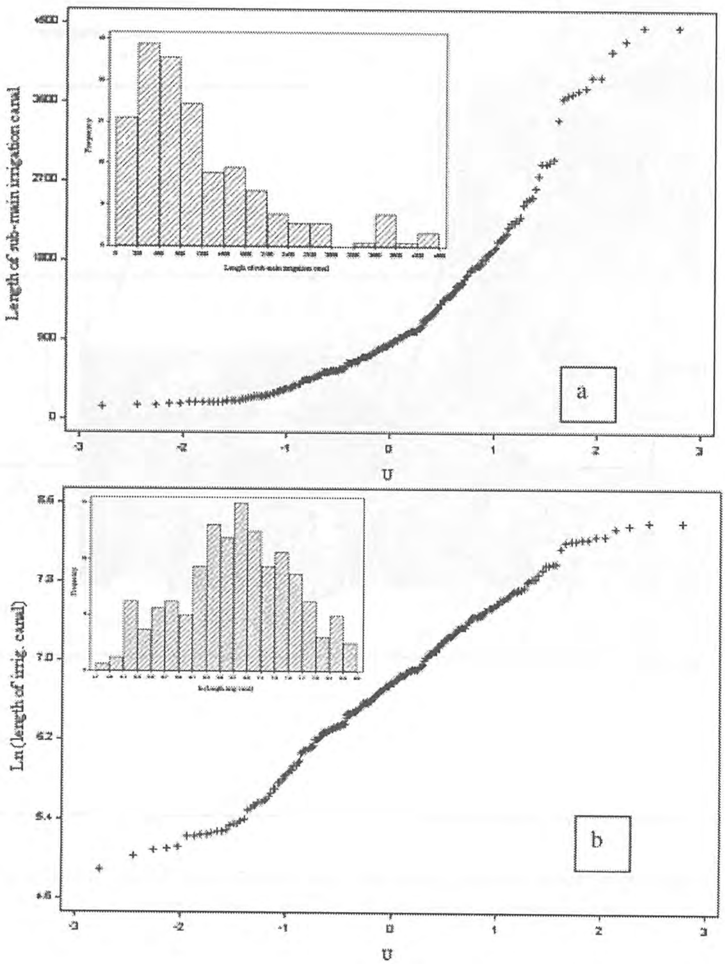

Fig. 1: Histograms and fractile diagrams for the length of sub-main irrigation canals (L_{smi}) of Al-Hassa project: (a) L_{smi} vs U and (b) ln (L_{smi}) vs U.

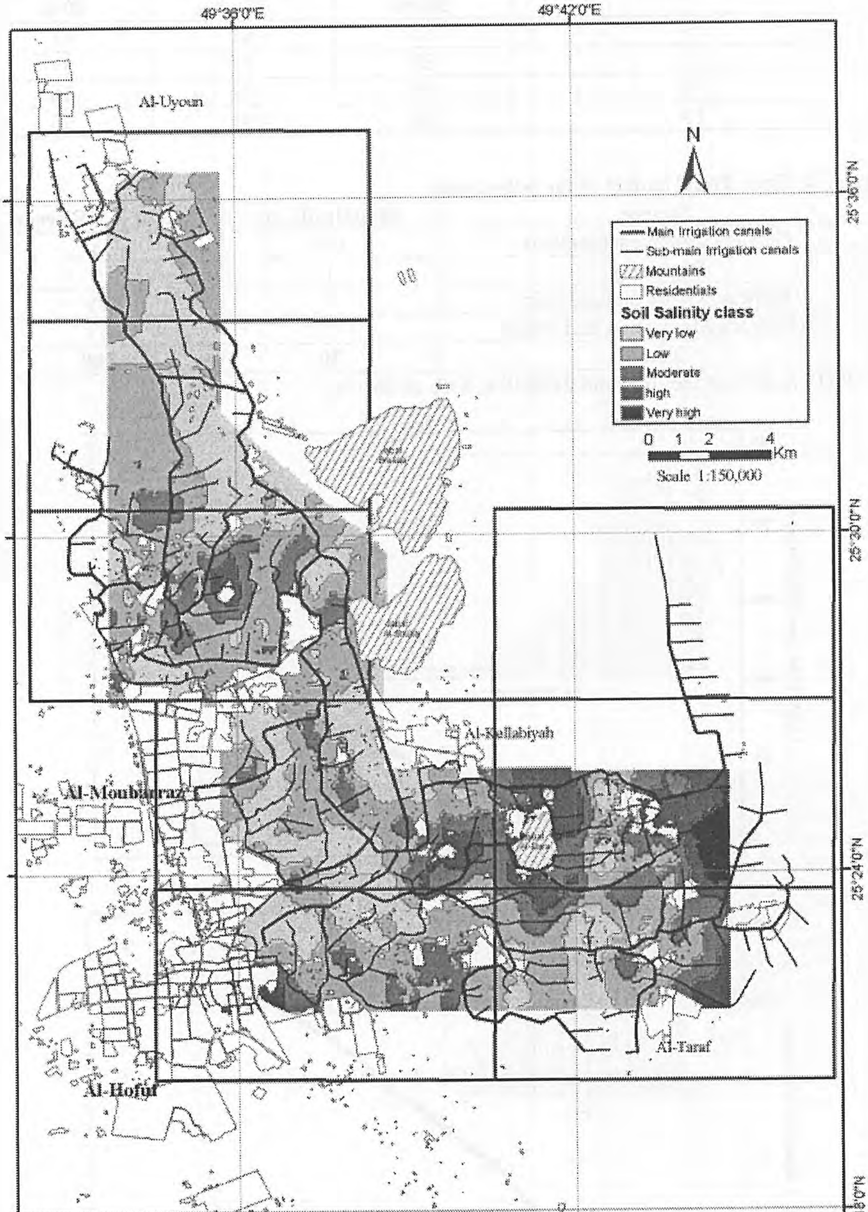

Figure 2: Irrigation network (lateral and labels are not shown), SPOT residential areas, and kriged soil salinity. Al-Hassa, Saudi Arabia.

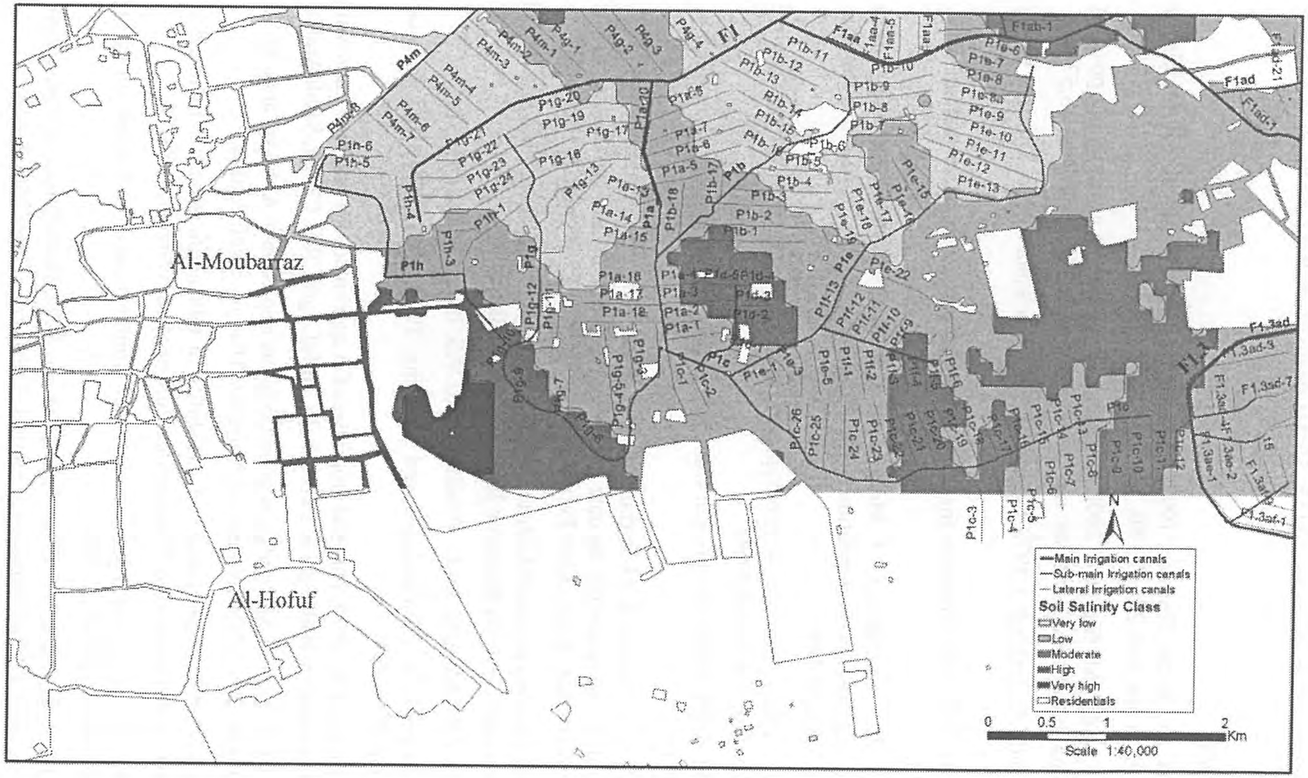

Fig. 3: Irrigation network east of Al-Hofuf (see Fig.2)

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Monitoring Vegetation Change in Abu Dhabi Emirate from 1996 to 2000 and 2004 using Landsat Satellite Imagery

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MONITORING VEGETATION CHANGE IN ABU DHABI EMIRATE FROM 1996 TO 2000 AND 2004 USING LANDSAT SATELLITE IMAGERY

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ABSTRACT

The Groundwater Research Program (GWRP) was established in 1988 and is a cooperative effort between the National Drilling Company of Abu Dhabi and the U.S. Geological Survey (USGS). The primary mission of the GWRP is to assess the ground-water resources of the Emirate of Abu Dhabi. In the fall of 2001, a study was initiated to investigate vegetation changes in the Emirate. The vast majority of vegetation present in the region is irrigated, and an analysis of vegetation change will support groundwater investigations in the region by indicating areas of increased groundwater use. Satellite-based imaging systems provide a good source of data for such an analysis. The recent analysis was completed between February and November 2002 using Landsat 5 Thematic Mapper satellite imagery acquired in 1996 and Landsat 7 Enhanced Thematic Mapper Plus imagery acquired in 2000. These assessments were augmented in 2004 with the study of Landsat 7 imagery acquired in early 2004. The total area of vegetation for each of seven study areas was calculated using the Normalized Difference Vegetation Index (NDVI) technique. Multi-band image classification was used to differentiate general vegetation types. Change analysis consisted of simple NDVI image differencing and post-classification change matrices. Measurements of total vegetation area for the Abu Dhabi Emirate indicate an increase from 77,200 hectares in 1996 to 162,700 hectares in 2000. Based on comparison with manual interpretation of satellite imagery, the amount of under-reporting of irrigated land is estimated at about 15 percent of the actual area. From the assessment of the 2004 Landsat imagery, it was found that the growth of irrigated vegetation in most areas of the Emirate had stabilized and had actually slightly decreased in some cases. The decreases are probably due to variability in the measurement technique and not due to actual decreases in area of vegetation.

Key words: vegetation monitoring, satellite imagery, Landsat 5, Abu Dhabi, normalized.

INTRODUCTION

The Groundwater Research Program (GWRP) is a cooperative program between the National Drilling Company (NDC) and the U.S. Geological Survey (USGS). The GWRP is managed and staffed by the Abu Dhabi NDC and is advised by USGS senior scientists. The mission of the GWRP is to assess the groundwater resources of the Emirate of Abu Dhabi, provide training related to water-resources investigations, and conduct research on the hydrology of the arid environment (Al Bady, 2002).

Remotely sensed imagery can be used effectively in hydrologic studies, such as where ground water is being used for irrigation. By knowing the location and type of new areas of irrigated vegetation, hydrologists can better understand how the ground water may be affected in that area. Different crop types use different kinds and amounts of groundwater. Acacia forests can tolerate more brackish water and use relatively less amounts of water. Grass fields require larger amounts of fresh water. An investigation in the use of satellite imagery to determine areas of irrigated vegetation and rates of change was conducted in the Abu Dhabi Emirate in 1997, using Landsat 5 Thematic Mapper satellite imagery acquired in 1987 and 1996 (Sohl, 1999). The 1997 study focused on examining ways to monitor vegetation change and suggested the most applicable change detection technique to be used by the GWRP staff.

The goal for this study was to extend the vegetation change analysis to the 1996 to 2000 and 2004 timeframe, and train the NDC staff in using remote-sensing techniques to perform this analysis in future applications. Landsat 7 Enhanced Thematic Mapper Plus (ETM+) satellite imagery was acquired for the winter 2000 timeframe and analysis was conducted to delineate areas of vegetation to compare with the 1996 results. Further analysis was performed, where possible, to describe the kinds of vegetation present, according to a simple land-cover classification scheme.

DESCRIPTION OF THE STUDY AREAS

With approximately 59,200 km² (square kilometers) of land area, the Abu Dhabi Emirate is the largest of the seven emirates that make up the United Arab Emirates. Located on the Arabian Peninsula, the United Arab Emirates is bordered by the Kingdom of Saudi Arabia to the west and south, the Sultanate of Oman to the east, and the Arabian Gulf to the north.

Most of the Abu Dhabi Emirate is underlain by nearly flat-lying sequences of sedimentary rocks, with the uplifted Oman Mountains and associated thrust zone in the east of the Emirate. Alluvial deposits eroded from the Oman Mountains are found on flood plains radiating from the mountain front. More recent deposits of wind-blown sand cover much of the current land surface (Moreland, 1988). The climate is arid, with average annual rainfall less than 10 cm. No perennial streams are present in the Emirate. Appreciable amounts of groundwater can be found, however, in dune sand deposits, alluvial deposits, and shallow sedimentary formations. Groundwater quality differs with location and depth from fresh, to brackish, to saline.

For the purposes of this study, the vegetative cover was categorized into five main classes: (1) Forests - primarily acacia trees planted in uniform plots; (2) Date palms - dense, well-established groves of palm trees; (3) Vegetable fields - small fields laid out in a patchwork fashion, containing a wide variety of vegetable plants; (4) Grass/

fodder fields - both small (4 hectares) and large (800 hectares) fields growing grasses typically used for animal feed; and (5) Dry grass - not a true vegetation type, but this refers to fields of cut grass laid out to dry. These fields of drying grass were distinctive in the multi-spectral imagery. Mixed classes, such as date palms/grass, trees/grass, and scattered acacia, also were used during some of the image classifications. Irrigation of all these vegetation types is supplied by groundwater.

Areas of irrigated vegetation are located primarily east of the city of Abu Dhabi, in a large area around the city of Al Ain, and in the Liwa region. Three Landsat scenes cover nearly the entire area of irrigated vegetation. To improve the results of the image classification and to allow for easier handling of the data sets, the scenes were divided into seven study areas. Each study area covers a separate region of irrigated vegetation in the Abu Dhabi Emirate. The imagery for the Abu Dhabi, Al Hayer, Al Khazna, and Al Ain study areas was contained in Landsat scene 160/43. The imagery for the Ghayathi study area was contained in scene 161/44. The locations of the study areas within the Abu Dhabi Emirate are presented in Figure 1.

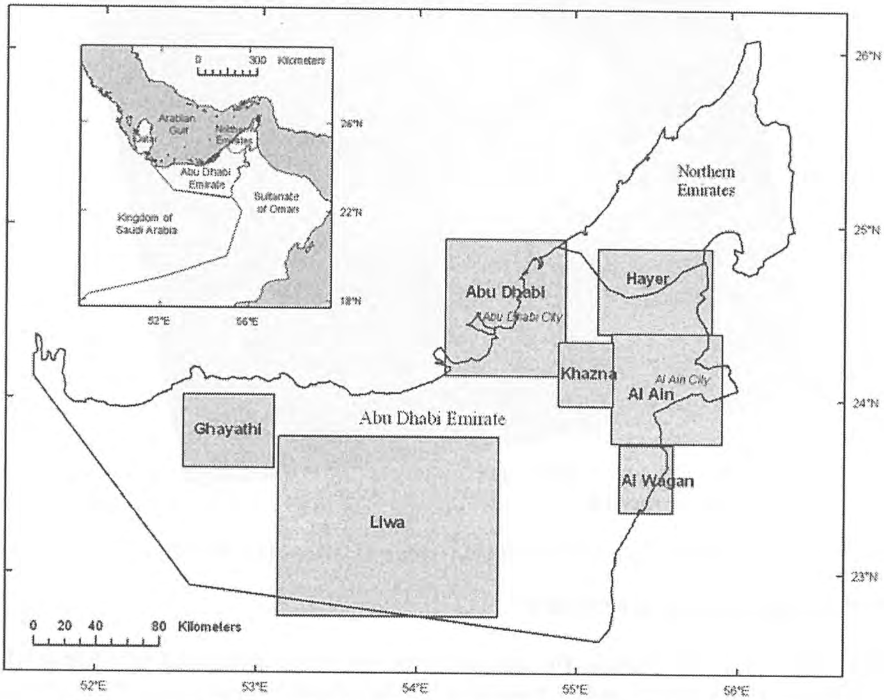

Figure 1: Study areas in Abu Dhabi Emirate.

DATA SOURCES

Landsat Scenes

The Landsat 7 ETM+ scenes were ordered from the USGS EROS Data Center in Sioux Falls, South Dakota. Each scene was georeferenced to the UTM projection/coordinate system, zone 40, was terrain corrected, and was in GEOTIFF format. Each scene covers an area approximately 180 km on a side, or 32,000 km². The data sets were loaded onto local computer disks at the GWRP and were converted into ERDAS Imagine files and reprojected into the Transverse Mercator projection. The 1996 Landsat 5 imagery had been acquired earlier for the 1987-96 change analysis (Sohl, 1999) and already had been transformed into the appropriate projection/coordinate system. A browse image of a full Landsat 7 ETM+ scene (path 160, row 43), in natural color (bands 3,2,1 – red, green, blue), which contains the area from the city of Abu Dhabi in the west, to Al Ain in the east, and Dubai to the north is presented in Figure 2.

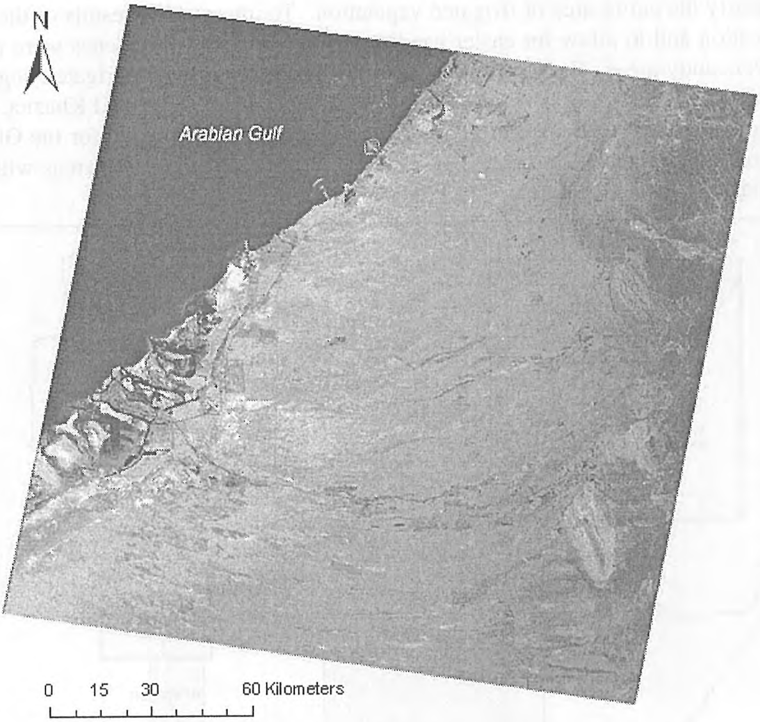

Figure 2: Landsat 7 ETM+ scene 160-43, May 30, 2004

METHODS AND PROCEDURES

The following sections describe the methods and procedures used for determining vegetation change using Landsat imagery in the Abu Dhabi Emirate, from 1996, 2000 and 2004.

Overview of Multi-spectral Imagery

Remote sensing essentially is the act of measuring a characteristic without physically encountering the subject. Remote sensing of the earth using satellite imagery measures electromagnetic energy (typically the light from the sun) after it has been reflected from the earth's surface. Different materials on the earth's surface interact with sunlight in different ways by reflecting or absorbing wavelengths of light in different intensities. These differences allow drawing conclusions about what type of material was present.

Vegetation reflects light the strongest in the green and infrared ranges of the electromagnetic spectrum. In other words, vegetation absorbs light mostly in the blue and red wavelengths. Because of this result, vegetation typically is seen as green. How a given material reflects the range of wavelengths of light is called its spectral reflectance curve, or its spectral signature. Multi-spectral sensors were designed to measure the intensities of reflected light in multiple wavelengths. Landsat 7 ETM+, for example, detects reflected light energy in six different wavelengths, (plus one panchromatic band and a thermal band) (Jensen, 2000). Having reflectance values for a given surface type in six different wavelengths allows the plotting of the reflectance curve with six data points. More data points (more image bands) allow the curve to be more precisely defined. Many variables can affect the way light is reflected or absorbed. Sun angle, moisture levels, topography, atmospheric conditions, surface heterogeneity, and many other factors can change the intensity of light reflected by a given surface type. Because of this characteristic, satellite remote sensing only can give indicators of surface type.

Data Preprocessing

Within ERDAS Imagine, the import utility was used to convert the .tif files into .img files. For each scene, bands 1, 2, 3, 4, 5, and 7 were stacked into a single .img file using the layerstack utility. This stacking allowed for easier file handling. The original data are stored as unsigned 8-bit data, meaning that each pixel can have a value up to 8 bits in size (2 to the 8^{th} power = 256 possible values). Bands 6(1) and 6(2) are thermal bands (the detectors were sensitive to wavelengths of light in the thermal infrared range) and generally are not useful in these applications. Band 8 is a high-resolution panchromatic image. The detectors have a higher spatial resolution (15 m as compared to 30 m for the other bands and 60 m for the thermal), and a broad spectral resolution (sensitive to a wide range of spectral wavelengths). As the working projection used in the GWRP is Transverse Mercator, the imagery was reprojected from UTM. This reprojection was done using Imagine's reproject utility. The output projection was defined as: Transverse Mercator, Clarke 1880 spheroid, Nahrwan UAE datum, scale factor 0.9996, central meridian of 54 degrees longitude, origin of latitude of 0 degrees, and 500,000 m false easting. A second-order polynomial equation provided sufficiently accurate results (sub-pixel RMSE). Nearest neighbor resampling was used as it retains the original pixel values without any averaging.

Determination of Vegetated Areas

The first step in analyzing the vegetated areas in the Abu Dhabi Emirate was to create Normalized Difference Vegetation Index (NDVI) images for each study area. The index is created by subtracting band 3 from band 4, and normalizing that value. Because healthy vegetation reflects light strongly in the infrared frequency (band 4), and not strongly in the red (band 3), the presence of high values in the difference image usually indicates green vegetation. ERDAS Imagine software has a built-in function for the creation of NDVI data sets. The year 2000 NDVI greyscale image for the Al Khazna study area is shown in Figure 3.

The NDVI images created were 8 bit, with 256 possible pixel values. These images were converted to thematic images (as opposed to continuous images), and displayed in pseudocolor mode. Using the inquire cursor and the raster attributes display, pixel values were chosen that best defined the threshold between vegetation and not vegetation. The image was converted into a binary file where pixels can have one of only two possible values, representing vegetation or not vegetation. For example, it may be decided that original pixel values lower than 81 do not represent vegetation, and pixels with values 82 and above do represent vegetation. This determination is made after viewing the image at four or five threshold levels. The display color of the pixels was changed from grey-level values to either black or white, with white representing vegetation. There was a threshold value, which varied from image to image, and included all the vegetation in the image, but did not include too many "noise" pixels. Noise pixels, or pixels incorrectly classed as being vegetation, were kept to a minimum. This was difficult in some images, as some of the faint vegetation, such as widely spaced acacia trees, would not be classed as vegetation before large areas of noise would start to be included. For the most part, however, a reasonable distinction could be made between areas of vegetation and no vegetation.

Figure 3: Normalized Difference Vegetation Index greyscale image of the Al Khazna study area, 2000.

After a determination of a threshold value had been made, the NDVI image was recoded using the raster recode utility in the Imagine software. Values below the threshold were recoded to 1 and values above the threshold were recoded to 2 (a background value of 0 was retained). To make the resulting images easier to interpret, a filtering algorithm was run over the image. This filtering does not make the image any more accurate (it may reduce slightly the accuracy), but it allows the user to more easily read the map results by eliminating small, isolated pixels that appear as noise. A neighborhood function filter was used, with a 3-by-3 majority window. This function applies a 3-by-3 window over the image and evaluates each pixel. The pixel value in the center of the window is replaced by the value of the majority of the pixels within the window. This process has the effect of eliminating small, isolated pixels of either category. Using the Raster Attribute Editor, an attribute column of area was added (in hectares).

One limitation of the NDVI analysis technique is that it only is sensitive to appreciable green plant-surface cover and cannot indicate those areas where the land has been prepared for irrigation (substrate preparation, laying of irrigation piping, and others) but plants were not yet visible. To be held above the NDVI threshold in this analysis, the amount of plant cover had to be appreciably greater than the background reflectance contributed by the soil/sand substrate. This result will tend to underestimate areas of irrigation as compared to amounts based on actual irrigated land, regardless of plant maturity.

IMAGE CLASSIFICATION

Supervised and Unsupervised Classification

Image classification is the assignment of thematic information (usually land cover type) to pixels based on an analysis of the reflectance values across the various spectral bands. Different surface materials interact with light in different ways by absorbing some wavelengths and reflecting others. By detecting the intensity of light reflected across a number of wavelengths, an idea of the reflected signature of the material can be made. The reflected light from a surface material can be affected by a large number of variables, including atmospheric conditions, sun angle, and moisture content. These characteristics change over time and with location, so the prediction of a given surface type based on measured light reflectance almost can never be made with 100 percent certainty.

There are two basic approaches to image classification. Supervised classification normally is used when the investigator has a good knowledge of the surface types present in the image area. The investigator points at examples of the various surface types in the image (usually drawing polygons around them) and these examples are used in computer analysis to find pixels throughout the image with similar spectral characteristics. Supervised classification techniques were used only for the Al Ain study area in 2000, because of the high level of user knowledge in this area. The second approach is called "Unsupervised Classification", and is used most often when the investigator does not have good knowledge of all the surface types present in the image. This approach was used for all the study areas in the Abu Dhabi Emirate. The spectral signatures of the pixels are examined with computer analysis and are grouped or "clustered" based on an iterative statistical analysis. The user specifies how many categories will result. Generally, it is best to specify as many as three times the number of suspected surface types. 60 classes were used in the study areas. After classifying the image into categories with the software, the user examines the resulting thematic image and tries to name each category. In ERDAS Imagine, the displayed color of the various categories can be changed using the raster attributes editor and the inquire cursor. Categories may be combined with others if they appear to define the same or similar surface type. This combination can be performed with the recode utility. In an attempt to create a better classification of images, the filtered NDVI image was used as a mask on the multi-band imagery prior to classification. This masking eliminated the non-vegetated areas in the unsupervised classification analysis.

Land-Cover Classification Scheme

Because the NDVI mask was used to exclude non-vegetated areas, only those areas representing some kind of vegetation were categorized. Land cover such as sand, bare rock, or urban areas all were combined into one category, non-vegetation. Attempts were made to discriminate vegetated areas into the following categories: forests, vegetable fields, dry grass, date palms, and grass/fodder fields. Mixed categories were also used in some cases.

Change-Detection Analysis

The change-detection analysis was conducted in two ways. The first way was the direct comparison of thresholded NDVI images. By recoding one of the output images, the 1996 and the 2000 NDVI images could be added to obtain a new image showing four possible outcomes:1—non-vegetation in both years; 2—vegetation only in 1996; 3—vegetation only in 2000; and 4—vegetation in both years. The second way was the comparison of the post-classification images. After satisfaction with the assignment of categories to the classified thematic images, an image addition was performed, similar to the NDVI change analysis. After recoding one of the images (1996 or 2000) to allow for unique outcomes, the two images were added together. The results can be displayed in a change matrix, showing the area falling into each new category. This kind of analysis gives more detail as to what kind of change is taking place than the first technique. The tables are presented as a matrix of the 1996 categories on the left and the resulting 2000 categories across the top. Therefore, for any given category in 1996, how much of that category either stayed the same or changed to a new land-cover category as in 2000, can be seen.

VEGETATION CHANGE

Total Vegetated Area for Study Areas

The results of the area calculations from the filtered NDVI image analysis are presented in Table 1. An increase in measured vegetation from 1996 to 2000 is seen in all study areas. Total area of vegetation for the Abu Dhabi Emirate as measured with the change-detection analysis shows an increase from 77,200 hectares in 1996 to 162,100 hectares in 2000 (Fig. 4). Slight decreases in vegetation for most study areas are seen in 2004.

Table 1: Total vegetation for the seven study areas, 1996, 2000, and 2004, (hectares).

Study Area	1996	2000	2004
Liwa	13,000	28,700	27,500
Al Ain	24,900	43,200	42,500
Al Khazna	8,300	14,700	12,500
Al Hayer	6,900	14,200	13,600
Abu Dhabi	14,900	41,300	38,700
Ghayathi	7,900	12,400	9,100
Al Wagan	1,600	7,600	7,800
Total	77,500	162,100	152,000

Classification by Study Area

For the Al Ain area, the dense groves of palm trees in the oasis presented a unique signature. This category does not seem to be present in the Liwa study area, or is not easily distinguishable from other categories. The spacing of palms is important to its signature, and outside of densely planted oases, the palms tend to appear more similar to other tree types. Also, the fields of drying, cut grass do not stand out in the Liwa area. While these fields were observed during a visit in March 2001, either they were not present when the Landsat scene was acquired (Dec. 27, 2000) or they are not easily separated from the other categories in this area. The results of a supervised classification for the Al Ain study area in 2000 is presented in Figure 5.

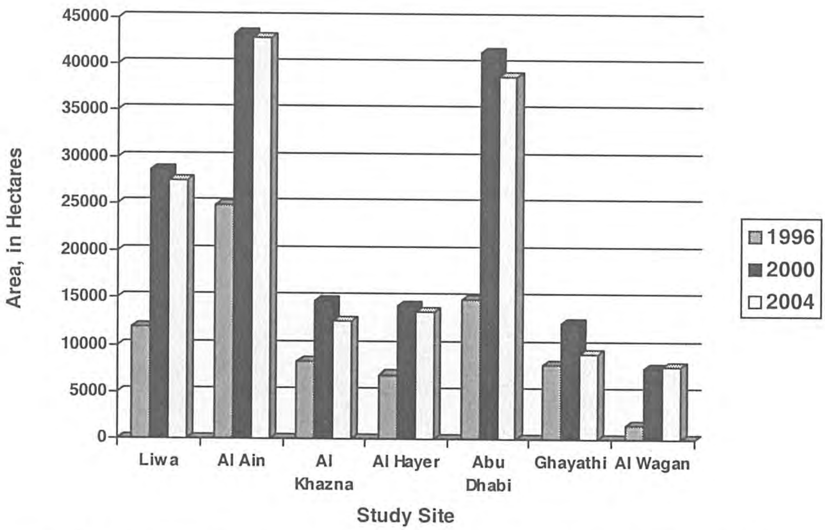

Figure 4: Area of total vegetation for each study area, 1996, 2000, and 2004.

Differences by Land-Cover Category

A matrix of the 1996 and 2000 categories from the unsupervised classification of the Al Ain study area are presented in Table 2. The largest change is non-vegetation to forest.

Table 2: Change matrix of the 1996 and 2000 classification categories for the Al Ain study area. (Hectares)

		Categories in 2000				
		Non-veg	**Palms**	**Grass/veg**	**Dry grass**	**Forest**
Categories in 1996	**Non-veg**	448,400	100	3,700	300	21,900
	Palms	50	400	50	0	200
	Grass/veg	900	700	800	200	2,800
	Dry grass	200	0	100	150	700
	Forest	6,100	300	1,600	150	9,300

Total Vegetation Differences

Total vegetation differences for the Al Ain study area are presented in Figure 6 and Table 3. The thematic image has four possible values: (1) was non-vegetation in 2000 or 2004, (2) was vegetation only in 2000, (3) was vegetation only in 2004, or (4) was vegetation in both 2000 and 2004.

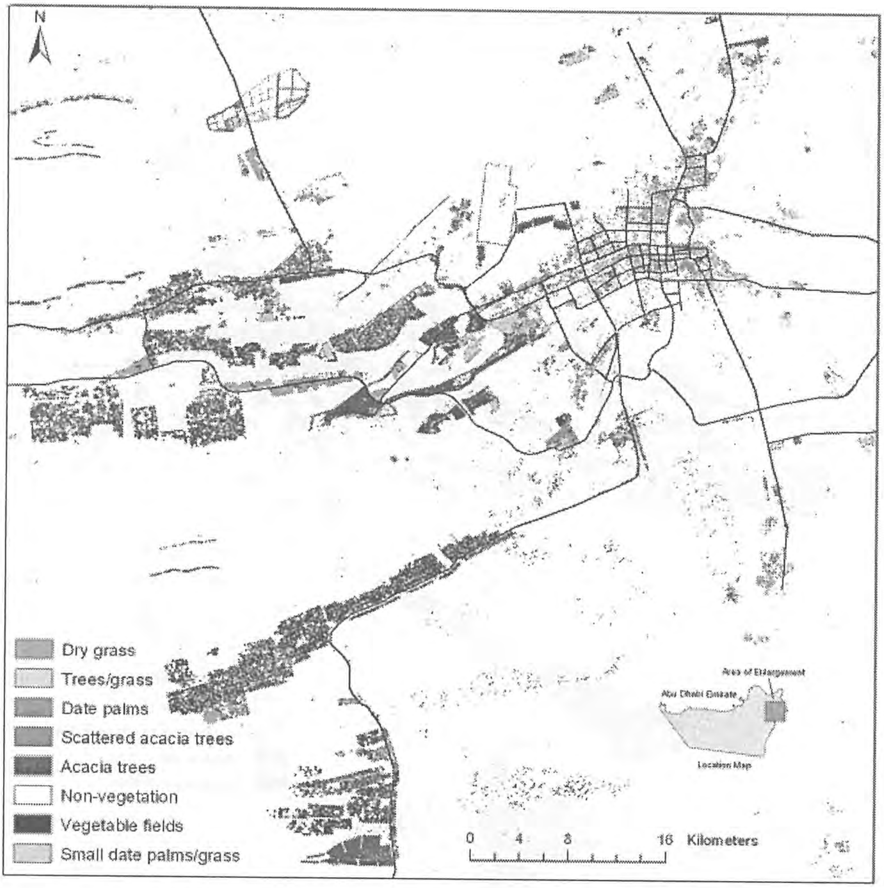

Figure 5: Supervised classification for the Al Ain study area, 2000.

ACCURACY OF THE CHANGE DETECTION ANALYSIS

To gain an understanding of the accuracy of the unsupervised classifications, a set of 80 random control points was generated for the Al Ain study area and the classification values were compared to field or photo-interpreted observations. ERDAS Imagine was used to generate the randomly placed points over the vegetation classification.

An error matrix resulting from analyzing control points for the Al Ain unsupervised classification is presented in Table 4. Field observations (reference data) along the X-axis, and classification results (map data) along the Y-axis are presented in the table. The values indicate the number of points that fall into each result/reference relation. For example, 14 of the 80 points were classified with the unsupervised classification process as category 4 (Acacia trees) and they also were classified as category 4 with the reference evaluation. These 14 points would be seen as correctly classified. Alternatively, three points were classified as category 6 (Vegetable fields) by the unsupervised classification technique, but were described as category 4 (Acacia trees) from the field evaluation. This result is seen as a misclassification. Categories 2 (Scattered trees) and 3 (Grass) had appreciable overlap and it was decided to collapse them into a single category. Overall, of the 80 control points, 51 were classified the same as the reference, resulting an overall 64 percent accuracy rate.

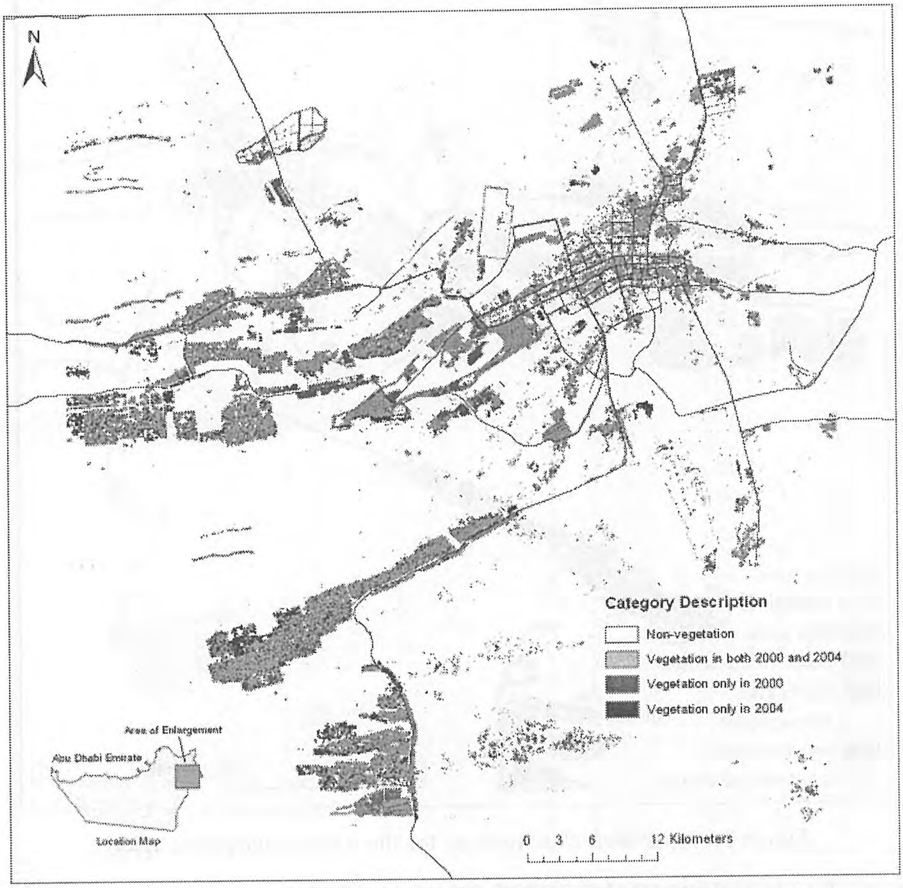

Figure 6: Vegetation difference in the Al Ain study area, 2000 to 2004.

Table 3: Vegetation differences (in hectares) for the Al Ain study area, 2000 to 2004.

Category	Description	Area (hectares)
1	Non-vegetation in 2000 and 2004	446,500
2	Vegetation in both 2000 and 2004	32,000
3	Vegetation only in 2000	11,000
4	Vegetation only in 2004	10,500

Table 4: Error matrix for the supervised classification for the Al Ain study area, 2000.

		Reference data (field observations)							
		Category description and number							
		Date palms (1)	Trees/ grass (2/3)	Acacia (4)	Scattered acacia (5)	Vegetable fields (6)	Dry grass (7)	Total	User's Accuracy (percent)
Map data (classification results)	Category description and number								
	Date palms (1)	4	2	-	-	-	-	6	67
	Trees/grass (2/3)	1	13	2	1	4	-	21	62
	Acacia (4)	-	3	14	2	3	-	22	64
	Scattered acacia (5)	-	-	1	4	2	-	7	57
	Vegetable fields (6)	-	3	3	1	10	-	17	59
	Dry grass (7)	-	1	-	-	-	6	7	86
	Total	5	22	20	8	19	6	80	na
	Producer's Accuracy (percent)	80	59	70	50	53	100	na	64

A study in 1990 (Hamid and Hassan, 1993; Maddy, 1993) found that the agriculture area measured by field verification was about 50 percent greater than that measured from satellite imagery. For the Al Hayer study area, a small study was performed where the vegetation was manually interpreted from the higher resolution Landsat band 7. The resulting polygons representing manually interpreted vegetation totaled 16,600 hectares. The NDVI analysis for this study area indicates 14,200 hectares of vegetation, or 14 percent less than that measured by manual interpretation. While this is less than the 50 percent under reporting indicated by Hamid, the results of the algorithm-based satellite assessment are clearly not correctly classifying all the vegetation present in the study area.

Factors affecting the determination of accuracy include that the land cover may have changed since the imagery was acquired in 2000. This result may be true of vegetable and grass fields, but tree farms and palm oasis probably will remain stable over many years. It is speculated that the primary effect to the accuracy of measurements of total vegetation area determined during this study is the density and maturity of the plants. When relatively young, the trees are small and widely spaced. From satellite imagery, the surrounding sand has a much stronger affect on the measured reflectance than the actual tree. When performing an NDVI analysis, these areas of small trees will fall below the threshold of vegetation, causing an under reporting of vegetation in the study area.

Based on field observations, farm fields are relatively small compared with acacia forests, with a variety of trees and palms planted along the edges. At the 30 m resolution, the pixels usually contain a mixture of land cover types, making the distinction between crops and trees difficult. The typical farm field is about 190 meters square, containing 3.5 hectares. Approximately 36 pixels would cover a typical farm field. With edge effects, and the general lack of homogeneity within most fields, clear distinction between crops and trees is difficult. This difficulty is probably the primary cause of confusion between the vegetable plot class and the other categories. The small field size and heterogeneity of the vegetation types in combination with small errors in horizontal registration could also effect the correct assessment of the resulting classification.

There is clear textural and shape distinction between farm areas and forests. The farm areas show a rectangular, patterned texture, whereas the planted forests usually appear as continuous blocks of trees with sharp outside boundaries. It is possible that the use of additional rule sets and ancillary data could refine the classification of these categories.

SUMMARY AND CONCLUSIONS

Using Landsat 7 ETM+ satellite imagery, vegetation for the Abu Dhabi Emirate has been mapped in 1996, 2000, and 2004 and the results compared to indicate the amount and location of change. The vegetation was measured in terms of actual green vegetation growing at the time of the imagery acquisition to give total area in hectares for each study area.

Measures of actual area of vegetation in the Emirate indicate an increase from 77,500 ha in 1996 to 162,100 ha in 2000 (an increase of 109 percent). The largest increases in area of vegetation have been measured in the Abu Dhabi, Al Ain, and Liwa study areas. Area of measured vegetation is less than the area of irrigated land, because of the inability of the satellite sensor to detect small, widely spaced, or immature plants. Based on comparison with manual interpretation of satellite imagery, the amount of under reporting of irrigated lands is estimated at about 15 percent of the actual area. This amount of under reporting is much lower than that found in earlier assessments. Measurements taken from the 2004 imagery indicate that growth of new vegetated areas has slowed in most areas. Based on visual interpretation of the imagery, the small decreases in vegetated area are probably due to an inability of the measurement technique to sense certain patches of vegetation, as the extent of the fields appears to be the same as previous years. This may be due to differences in the time of year of image acquisition.

Multi-band classifications were performed successfully on the Al Ain and Liwa study areas. By area, the vegetation type most prevalent is forest cover. These areas consist primarily of acacia trees that have been planted successfully in large tracts throughout the Emirate. Category change is predominantly non-vegetated land to forest cover. Based on a comparison with 80 test points, the classification of the Al Ain study area showed an overall accuracy rate of 64 percent. Factors contributing to this relatively low accuracy rate may include the relative small size of the vegetable plots (usually about 4 hectares) and the high heterogeneity of the fields. Small discrepancies in horizontal registration could also have an effect on the accuracy of indicated land-cover categories.

Overall, irrigated vegetation has increased greatly in the Emirate, (more than 100 percent increase in 4 years), with addition of new fields slowing in recent years. By using satellite imagery to monitor changes in vegetation in the Emirate, scientists will have an efficient and valuable data source to aid them in ensuring the proper management of the groundwater resources in the Abu Dhabi Emirate.

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WATER MANAGEMENT IN THE OIL INDUSTRY

Biological treatment of oil process water and use in biosaline agriculture

B. A. Hasbini, Y. M. Al-Lawati, S. Ismail

BIOLOGICAL TREATMENT OF OIL PROCESS WATER AND USE IN BIOSALINE AGRICULTURE

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ABSTRACT

Oil production is often associated with production of significant volumes of saline water. This water contains oil residues along with some heavy metals and disposal has always been problematic from economic and environmental perspectives. The common practice is to dispose of this water in deep aquifers through injection wells. This process is expensive and has pollution hazards. Petroleum Development Oman (PDO) is one such company where on a daily basis, over 500,000 cubic meters of produced water must be disposed of through deep water injection. Alternatives to disposal of process water include low-cost biological treatment through wetlands and re-use of saline effluent in biosaline agriculture. In this context, pilot wetland treatment systems based on reeds (*Phragmitis australis*) were constructed in southern Oman in April 2000 to investigate treatment and disposal of process water through a combination of biological treatment systems and volume reduction through evapotranspiration. The International Center for Biosaline Agriculture (ICBA) was given the task of designing a suitable biosaline agriculture scheme for re-use of treated saline water. As wetlands were not functioning as intended, ICBA was also requested to review the treatment process and identify key parameters affecting the proper functioning of wetlands. Following extensive chemical and physical analyses, ICBA scientists recommended a series of operation modifications which improved the performance of wetlands significantly. About 79 percent of the readings between August 2001 and August 2002 were within treatment objectives and the average oil-in-water concentration (OIW) of the effluent dropped from 14.9 ppm to 7.0 ppm. These same principles were also implemented in a new wetland system. Early analyses demonstrated that the new wetland is treating oil process water to unsurpassed levels. A number of grasses, shrubs and tree species were selected based on their tolerance to salts present in the effluent. In general, trees and shrubs were more successful than grasses which did not survive the elevated Boron concentration of the treated effluent.

Keywords: Biosaline agriculture, oil process water, wetlands, biological treatment systems, re-use of saline water

INTRODUCTION

Natural wetland treatment systems have been used for wastewater disposal for as long as wastewater has been collected, with documented discharges dating back to 1912 [2]. They normally consist of aquatic plants growing in waterlogged fields irrigated with partially treated effluent water. Combinations of physical and microbiological mechanisms are responsible for the removal of pollutants. Constructed wetland wastewater treatment systems are relatively easy to operate and maintain and are extremely energy efficient particularly when compared to conventional mechanical systems.

Over the years, wetlands have provided wastewater treatment for pollutants from sources ranging from rainfall runoff to intentional discharges of community sewage. Constructed wetlands may range from the creation of a marsh in a natural setting to extensive construction involving considerable earthmoving and creation of impermeable barriers.

Constructed wetlands are of two types: free water surface systems (also called surface flow systems) and vegetated submerged systems or subsurface flow systems. The difference between the two systems is related to the flow of water whether vertically or horizontally through the root zone. Free-water systems are typically inundated with water while vegetated submerged systems have the water surface below the soil surface. Each system is different in terms of design; however, subsurface flow systems are more efficient in terms of space but are more expensive to construct.

Many plant species are available for wetland treatment systems. The two principal groups are free floating plants such as water hyacinths and duckweeds and rooted plants such as cattails and reeds. Operationally, the main difference between the two groups is that the floating plants require regular harvesting to maintain effectiveness. The availability of nutrients affects the diversity and distribution of emergent plant species, the growth rate, and structural allocation of growth.

Few systems were constructed around the world to treat waters containing oil and grease residues. Success stories were reported by the Amoco's Mandan refinery in North Dakota, Chevron refinery in Richmond, California, and the Jingling Petrochemical Company in Nanjing, China, as well as several others where oil and grease residues from industry effluents have been reduced significantly [10]. Most studies researching treatment processes have concentrated on identifying the treatment mechanisms rather than developing standards for design and operation. For this reason a pilot-scale project was always recommended prior to construction of a large-scale wetland treatment system.

Pilot wetland treatment systems have been constructed in Rahab and Nimr regions in southern Oman. The objective was to develop an economical and environment- friendly system for treating oil process water, as an alternative to injection in shallow and deep aquifers. These systems were subjected to close monitoring in an attempt to refine and evaluate the entire technology. This paper presents an overview of this technology and the results achieved.

BACKGROUND

Petroleum Development Oman (PDO) produces 550,000 m3/day of saline process water along with its oil production of 135,000 m3/day. This process water is not readily usable as it contains oil and heavy metal contaminants and is being disposed as follows [18]:

1. Injection back to the reservoir for pressure maintenance
2. Disposal in deep water aquifers (DWD)
3. Disposal in shallow water aquifers (SWD)
4. Disposal at Mina Al Fahal into the sea
5. Use in various water flood/water injection pilots

At present, approximately 40 percent of produced water is injected back into the reservoir for pressure maintenance. Most of the remaining volume of 330,000 m3/day is disposed by SWD and DWD. In the Nimr area, about 70 percent of process water is disposed in shallow and deep-water aquifers [18].

Due to environmental constraints, a decision was made to phase out SWD by 2003. Additionally, DWD is a costly and energy intensive process. For these reasons, PDO decided to investigate the use of inexpensive biological treatment systems for developing an environment-friendly solution to water disposal.

Following a series of investigative studies, an 800 m2 pilot project was established in the Rahab region in southern Oman in 1998 to evaluate the use of indigenous hydrophytes in treating process water. Three reed species were tested namely *Phragmites australis*, *Typha latifolia* sp. and *Bolema* sp. The design treatment capacity of Rahab was 20 m3/day. Evaluation studies conducted in 1999 and 2000 revealed that over 98 % of hydrocarbons in process water were removed [3]. Encouraged by these results, a larger pilot project with a reed bed area of 3 hectares was constructed in Nimr in early 2000. The design treatment capacity of this site was 2,000 m3/day. The design of the Nimr reed bed site differed from the Rahab site in several aspects in an attempt to optimize the treatment process.

A rigorous monitoring program was established to monitor the performance of the beds in terms of operation and treatment. On a daily basis, inflow and outflow information is recorded. On a weekly basis, samples are analyzed for their oil-in-water (OIW) content, chloride concentration and total dissolved solids. On a monthly basis a more comprehensive analysis is performed whereby the concentrations of most pollutants are analyzed and evaluated.

Physical Setup of the System in Nimr

Four reed beds were established in series for the purpose of water treatment and volume reduction. These beds were termed B1 to B4. Reed bed B1 is a treatment bed with a soil depth of 1.25 m. Water discharged from B1 is then delivered by gravity to the next reed bed, B2. Beds B2, B3, and B4, have a combined area of 1.13 ha and a depth of 0.5 m, and are used to reduce water volume through evapotranspiration.

Discharge from the fourth bed is delivered to an evaporation pond of 1.7 ha connected to a spray evaporation system. A significant amount of the effluent water is thus evaporated by spraying through nozzles at a pressure of 22 bars. The remaining water evaporates in the evaporation pond, leaving the salts behind. Figure 1 illustrates the B-train configuration.

Process water entering Reed Bed 1 has a salinity of ~ 6,000 ppm and when discharged from this bed the salinity is 7,000 ppm. Discharged water enters sequentially to beds B2, B3 and B4 where the salinity of the produced water increases to 8,000, 9,000 and 10,000 ppm, respectively. As per the estimated figures, the salinity of water leaving the 'spray evaporation pond' would be 180,000 ppm and that in the 'open evaporation ponds' would be 380,000 ppm.

Based on these figures, salinity of water discharged from the B-train is not a limiting factor in using this water for biosaline agriculture. Salt tolerant crop plants and halophytes are capable of utilizing this water for biomass, forage, fodder, and grain production.

An integrated water bioremediation and salt-tolerant crop production system is feasible and can be environmentally friendly if water is treated properly in the reed beds and the principles and practices of biosaline agriculture are adhered to. Moreover, the socio-economic and ecological benefits are expected to outweigh any risks associated with the use of saline water in agricultural production, if any.

Preliminary Results

Monitoring results for February 2001 and until June 2001 revealed that the Nimr reed beds are not operating satisfactorily [18]. Firstly, the actual capacity of the system was much smaller than perceived in the design. The application of flows larger than 250 m3/day over the reed bed area of 3,750 m2 (75x50 m2) always led to flooding. Actual drainage discharge was much smaller than perceived. Secondly, OIW content in the reed beds effluent exceeded acceptable limits set at 10 ppm in the outflow, reflecting inadequate treatment of process water. Thirdly, evaporation of water through spraying did not prove practical. As a result, large volumes of saline water had to be disposed somehow.

A scientific delegation from ICBA visited the Nimr Reed Beds in July 2001. Activities of this delegation involved extensive measurements and samplings of soil, water and plants. The scope of activities was later enlarged to incorporate the recommendations of the assessment in a new reed bed system design that would be implemented in Bed A1. The key findings presented in an evaluation report were as follows [7]:

1. Reed bed B1 is actually filtering process water rather than degrading oil residues. An oil slick was found at about 80 cm below soil surface.
2. The capacity of reed bed B1 is at best 250 m3/day. This capacity will increase to 1,000 m3/day if all four beds B1 to B4 are operated in parallel.
3. Average OIW of process water entering the B1 reed bed is about 150 ppm. Technical problems at CPI separator will result in OIW of process water exceeding 700 ppm. The maximum OIW concentration of process water should not exceed 70-80 ppm. A settling tank must be used prior to the application of process water onto reed beds. Alternately, the capacity of the bed must be reduced to about 100 m3/day.
4. Reed plants were not growing uniformly. A periodic cutting and fertilizer program is needed to rejuvenate the plants.
5. The soil type used in B1 is not appropriate for proper water circulation. The infiltration rate is less than 1 mm per hour. As a result, waterlogging and subsequently anaerobic conditions are prevalent. A soil with an infiltration rate of 10 mm/hr would be more appropriate.

6. The drainage system is not performing as designed. A better design is needed. In particular, a bottom slope in the direction of flow in the bed would be appropriate.

Following these recommendations, The Nimr reed operator reduced the system capacity and implemented a physical program to re-open drain lines. In addition, he implemented a cutting and fertilizer program for reeds and used existing tanks in Nimr to settle process water prior to application onto the beds. By October 2001, OIW in the effluent of B1 dropped to below 10 ppm. As illustrated in Figure 2 Reeds were inflorescent and looked much healthier.

Figure 2: Inflorescent reeds in Nimr in October 2001

Encouraged by these results, PDO decided to adopt a design prepared by ICBA and transform a separate bed into a reed bed treating process water. This bed was termed bed A1. It was further decided to establish a biosaline agriculture demonstration project irrigated using treated process water. Contracting activities for A1 started in December 2001 and were completed in April 2002. Transplanted reeds were irrigated using treated sewage water until fully established. In June 2002, process water was applied. In July 2002, early observations illustrated in Figure 3 suggested that the newly established reed bed A1 is treating process water to unsurpassed levels.

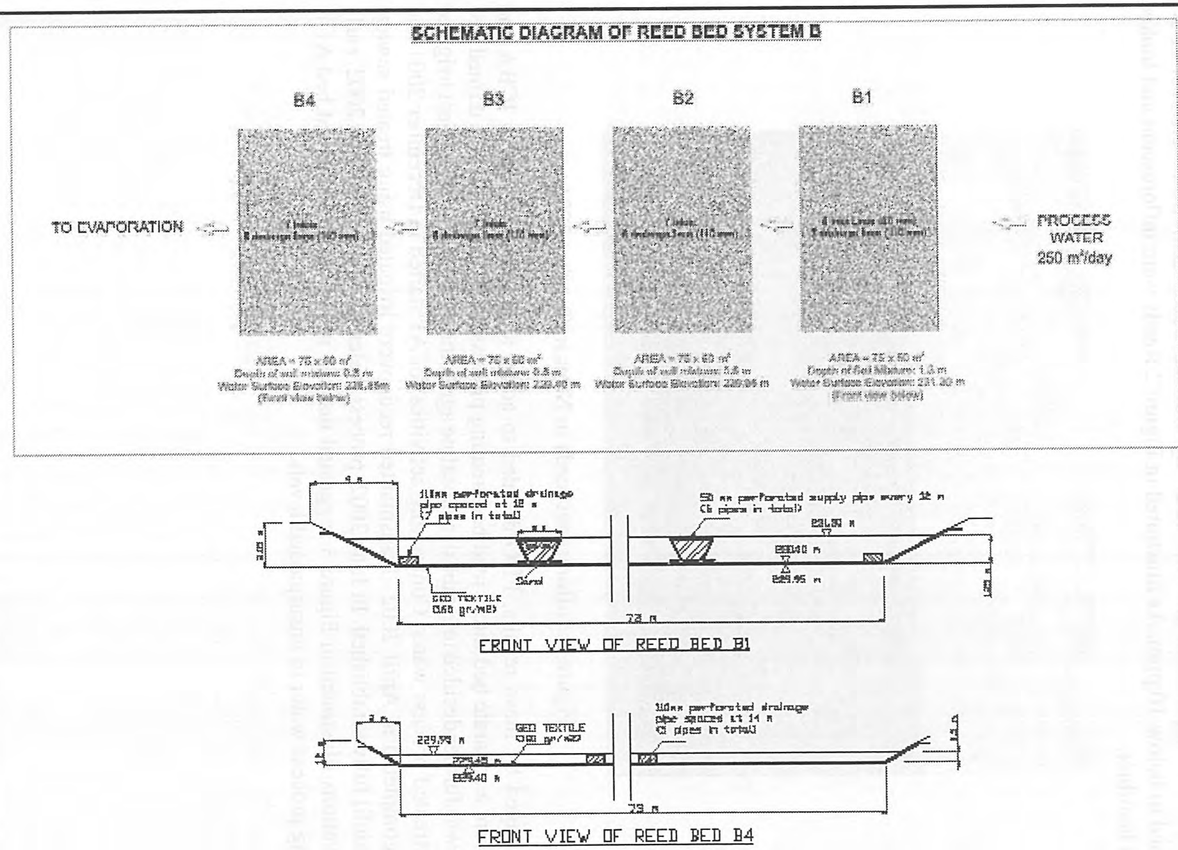


Figure 1. Layout of Reed Beds "B"

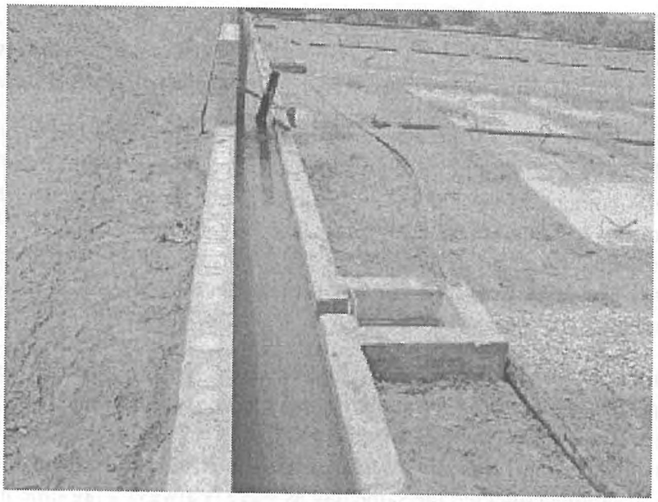


Figure 3: Treated effluent from reed bed A1

REED BED OPERATION

The operation of reed beds is a significant factor affecting the overall treatment process. Applying a large water volume when OIW concentration in the inflow is excessive will hamper the treatment for a considerable time. For this reason, determining the bed capacity was critical for this analysis. The ultimate objective is to operate the bed at or near capacity.

Reed bed treatment capacity is affected by several factors. These include bed size, soil intake rate, hydraulic loading rate, quality of the influent, in addition to other parameters. Analyses conducted revealed that OIW in the influent affects the treatment process in two aspects: firstly, reeds are killed as large volumes of oil are detrimental and secondly, the infiltration rate is reduced due to the clogging of soil pores with raw oil particles. Figures 6 and 7 illustrate patches of oil that practically killed reeds in B1 and raw oil in the soil.

Figures 6 and 7: Reeds affected by oil in B1 and soils containing raw oil

Design performance parameters for any reed bed systems include the, Hydraulic Loading Rate, and the Hydraulic Retention Time. The Hydraulic Loading Rate is the ratio of the inflow over the bed area. The Hydraulic Retention Time (*HRT*) is the ratio of the useable water volume of the bed over the inflow rate, and provides an estimate of the time available for microbiological reactions in rhizosphere.

The top surface area of reed bed B1 is 4,272 m2 while that of A1 is 4,256 m2. The design infiltration rate for A1 is 20 mm per hour. Consequently, the maximum inflow in A1 is 85 m3/hour or 2,043 m3 per day. Inflows larger than these figures will certainly cause flooding.

The hydraulic loading rate (*q*) for was designed at 6 cm/day, which corresponds to an inflow of 250 m3/day. However, this figure does not account for the quality of the influent in terms of OIW content. At low OIW in the influent, reed beds must be capable of handling higher volumes of process water, particularly that the infiltration rate is not limiting. To account for the quality of the influent, *q* is better expressed in terms of liters of oil per square meter of bed surface per day. As such, the maximum permissible inflow is variable and is dependent on OIW content.

The next challenge is to determine the maximum loading rate for which process treatment is still adequate. Due to the periodic simultaneous sampling of inflow and outflow, available data did not yield corresponding inflow/outflow OIW figures whereby a credible correlation could be established. Outflow OIW figures were not related to inflow OIW measured from an inflow sample collected the same day as there is always a lag time involved. For this reason, probability analysis was used [21].

The approach adopted in the following discussion is identical to that used in basic hydrology for analyzing flood events and sizing related civil structures [13]. The underlying assumption is that outflow OIW is strongly correlated to inflow OIW.

Based on collected data, an outflow OIW concentration of less than 10 ppm has a probability of 79 percent as illustrated in Figure 5. Similarly, an outflow OIW concentration of less than 5 ppm has a probability of 58 percent.

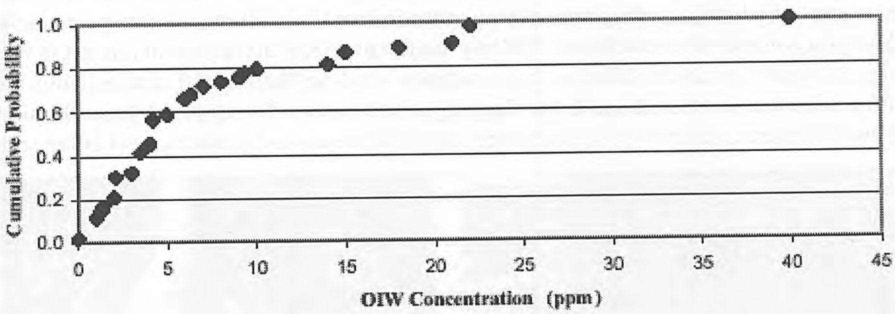


Figure 5: Probability plot for OIW in the Outflow of B1

The volume of oil entering the reed bed was calculated as the product of OIW concentration with the inflow for all weekly readings from August 11, 2001 to August 10 2002. A probability analysis was prepared for the volume of oil. Figure 6 illustrates this probability plot.

Combining this probability plot with the earlier probability plot prepared for OIW and isolating values at 79 percent (*P*79) and 58 percent (*P*58) would yield oil volumes of 32.4 liters and 12.4 liters respectively. These two volumes correspond to an outflow with maximum OIW concentrations of less than 10 ppm and 5 ppm respectively. The maximum loading rates per unit area under the two operation scenarios are thus calculated at 7.6 milliliters of oil per square meter of bed surface and 2.9 milliliters of oil per square meter of bed surface respectively.

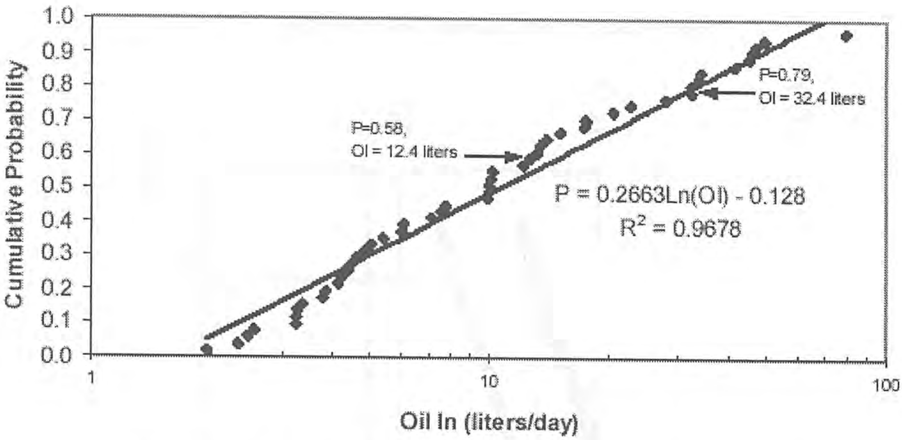


Figure 6: Probability Plot for Oil Entering Reed Bed B1 (OI)

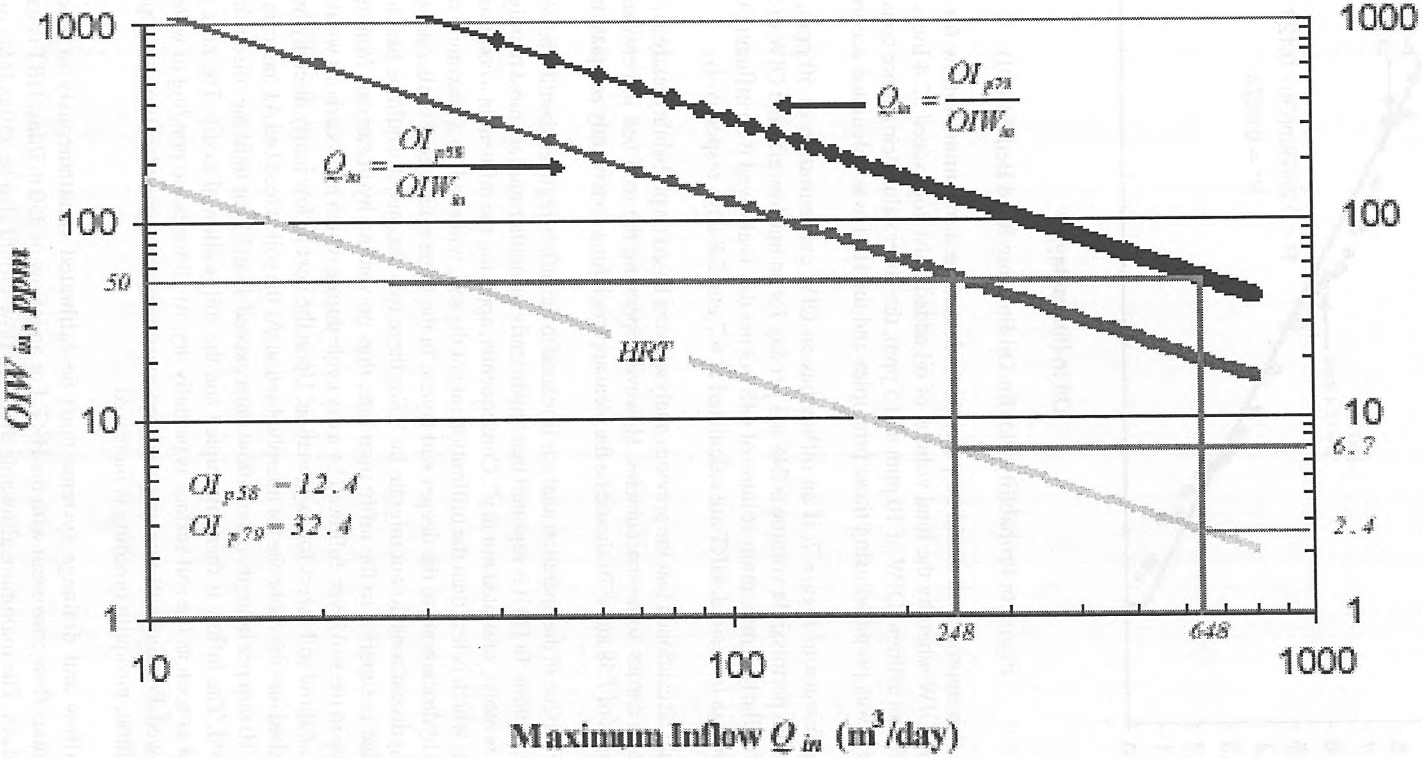
Using this approach it would be possible to determine a maximum inflow for a certain influent OIW whereby the total volume of oil added must not exceed 32.4 liters and 12.4 liters for an effluent OIW of 10 ppm and 5 ppm, the latter value being more conservative. Figure 7 was prepared using these principles and the HRT was calculated accordingly.

With reference to Figure 7, if an inflow has an OIW concentration of 50 ppm, then the maximum permissible volume is 248 m3 per day for an outflow effluent OIW of 5 ppm or less. Similarly, a maximum volume of 648 m3 per day is allowed if an effluent OIW of 10 ppm or less is desired. HRT are calculated at 6.7 and 2.8 days respectively.

It should be cautioned that the previous analyses were based on probability analysis, which by definition carries some inaccuracies. However, operating the reed bed in between the two envelopes of *P*58 and *P*79 should be the ideal approach for a consistently adequate treatment.

The next issue in the operation analysis is related to the inflow regime: should it be continuous or intermittent. In [1] it is reported that "hydrocarbons neither migrate most rapidly in a soil which is moist, saturated nor dry". Consequently, applying the influent in a continuous flow regime, which is less than the infiltration rate of the soil, will result in a maximum migration of raw hydrocarbons to the deeper soil layers. In the same reference it is also stated "in dry soils, hydrocarbons do not migrate far". For these two reasons, it would be best to apply a flow that is superior to the infiltration rate, thus preventing hydrocarbons from migrating too deep in the soil layer, followed by a dry cycle during which hydrocarbons would deposit on the shallow soil layers for oxygenation. Upon the next inflow partly dried hydrocarbons are flushed into the root zone for microbial action. As the soil in reed bed A1 has an infiltration rate of 10 mm per hour, then the inflow must exceed 43 m3/hour until the soil surface is just saturated. The inflow is then interrupted and the soil is allowed to dry. The next inflow is applied as soon as the soil surface is partially dry. At all times, no ponding of water should be allowed. Ponding will encourage oxygen consuming algae growth causing eutrophication. In addition, mosquito breeding is increased.

The inflow and drainage systems must be calibrated simultaneously to discharge a continuous flow consistent with the HRT. For an inflow of 350 m3/day, HRT is calculated at 4.3 days. The maximum allowable drainage flow should also be 350 m3/day minus any evapotranspration.



OIW in, ppm
Hydraulic Retention Time, days
Maximum Inflow Q in (m3/day)
Q in = OI p79 / OIW in
Q in = OI p58 / OIW in
HRT
OI p58 = 12.4
OI p79 = 32.4
50
6.7
2.4
248
648
1000
100
10
1

SIZING OF REED BEDS

Proper sizing of reed beds can be performed based on the results of the monitoring program and the analyses presented herein. Determining the optimal bed size depends largely on the OIW content and volume of the influent. The proposed approach is a slightly modified civil structures sizing approach based on probability analysis. To illustrate this approach, a minimum design process water inflow of 5,000 m3/day will be used.

Cumulative probability distribution of OIW of Nimr process water is presented in Figure 8. An OIW probability of excedence of once per year, i.e. $P=1/365$ means that OIW in process water will exceed the bed design capacity and flow once per year. Consequently 99.73 percent of OIW measurements are below the corresponding threshold OIW. Obviously, construction costs and, available budget may impose different design values but for illustrating this approach, this probability will be used.

The corresponding OIW for a probability of exceedence of 1/365 is obtained from Figure 10 or calculated using the correlation equation at 360 ppm. At an inflow of 5,000 m3/day, the maximum bed loading is calculated at 1.8 cubic meters of oil per day.

Figure 8 also illustrates the distribution of OIW values of the influent. About 81 percent of the OIW of process water are less than 150 ppm. An alternative to using a maximum OIW of 360 ppm would be to use this lower OIW threshold value. The maximum bed loading is thus calculated at 750 liters of oil per day. The bed can be designed for this lower loading rate but it will not be able to handle the design inflow of 5,000 when OIW exceeds 150 ppm.

For a total loading of 1.8 m3/day, the calculated area of the bed ranges between 23.7 hectares and 62.07 hectares for loading rates of 7.6 ml/m2 and 2.9 ml/m2 respectively. The latter rate incurs a larger cost but is more conservative. An area of 36 hectares is thus selected from the above range and the loading rate is calculated at 5.0 ml/m2.

It must be cautioned that the above wetland area is designed for a maximum daily flow of 5,000 m3 with an OIW of 360 ppm. Should the OIW in process water drop to 50 ppm, the same wetland can handle up to 36,000 m3 of process water per day. The maximum flow is limited by the infiltration rate of the soil. From the experience acquired, the recommended soil must have an infiltration rate of 10 mm/hr. As such, the maximum inflow for a wetland area of 36 hectares is 86,400 m3/day. Figure 9 illustrates the relation between OIW and maximum volume of process water per day.

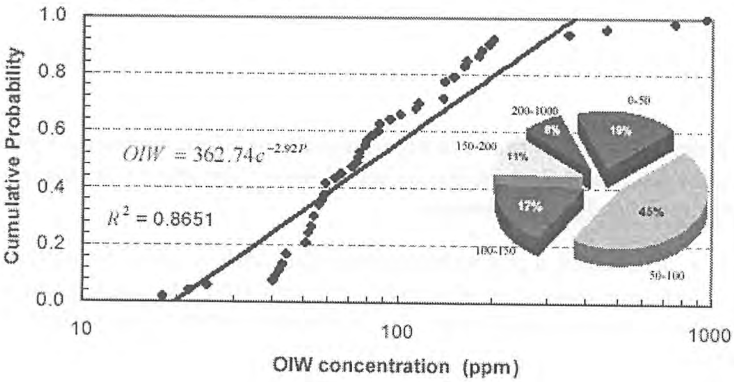


Figure 8: Probability and Frequency Distribution of OIW in Process Water

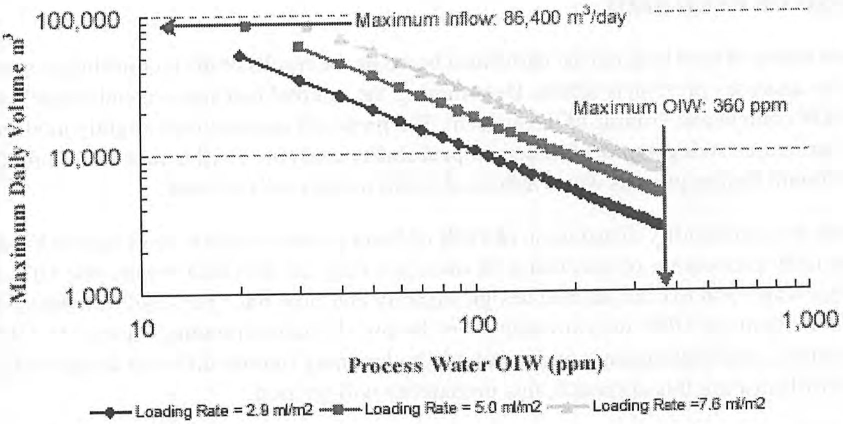


Figure 9: Maximum Daily Effluent for three Different Loading Rates

Monitoring Requirements

As mentioned earlier, the monitoring program implemented for B1 included daily inflow/outflow measurements, weekly OIW, TDS and Chloride measurements for both inflow and outflow, and comprehensive analyses performed on a monthly basis. Using probability analyses, various operational parameters were determined.

Inflow and outflow measurements must be performed using water meters as the current practice of a one-time sampling during the day is leading to inconsistent measurements. Ideally, water meters fitted with data loggers should be installed both at the inlets and outlets of each bed. A closer estimate of evapotranspiration can thus be obtained.

Evaluating the effectiveness of the treatment process involves comparison of corresponding inflow and outflow parameters, the least of which is OIW concentration. However, corresponding inflow/outflow figures cannot be compared without proper establishment of actual Hydraulic Retention Times (HRT). HRT can be determined using a continuous list of daily OIW measurements preferably over a 2-month period, and changes in inflow characteristics can then be matched to changes in outflow characteristics using the proper time lag or HRT. Alternatively, HRT is estimated using the following equation:

$$HRT = \frac{\mu V}{Q_{in}} \qquad (1)$$

Where μ is the soil porosity measured experimentally or obtained from the literature, *V* is the total soil volume and Q_{in} is the bed inflow. However, the above equation is not useful if gaps exist in the daily measurements.

When HRT is determined, a comprehensive chemical analysis of inflow/outflow must be performed for sampling intervals consistant with HRT. As such, changes in the concentration for individual pollutants can be assessed and the quality of the outflow is determined.

Soil samples must be collected at the same time and chelate extracts and saturated paste extracts must be analyzed concurrently for all constituents. The ability of the soil to retain heavy metals, measured as the difference between the chelate extraction and saturated paste extraction results, can then be extrapolated to determine the lifespan of the bed.

Lifespan of Reed Beds

Measurements performed on reed beds in Nimr are insufficient to accurately quantify their lifespan. Reed beds are considered unusable when the soil is no longer able to retain heavy metals and significant quantities are simultaneously present in the soil matrix and discharged in the effluent.

Determining the lifespan of a reed bed requires at least six measurements of heavy metal concentrations using chelate and saturated paste extraction techniques. These measurements should be performed on six-monthly bases for three years of uniform reed bed operation. As long as the concentration of heavy metals discharged in the effluent are within acceptable limits, then the reed bed is performing properly. The lifespan of the reed bed can then be extrapolated from analyses of variations in the l heavy metals content f the soil.

USE OF TREATED PROCESS WATER

Treated process water was used to irrigate a biosaline agriculture pilot project established in 2003. This work involved a multiple-cropping system where grasses, shrubs and trees were grown in an integrated approach. Grass species were primarily used for forage production, whereas, trees and shrubs were used for forage, wood and agroforestry demonstration.

Some of the grass species (*Paspalum vaginatum* and *Sporobolus arabicus*) were established successfully under harsh climatic conditions and with poor quality water. However, prevailing wind velocity and high Boron concentration in the outflow proved to be a critical factor for their further growth and productivity.

Shrubs (different species of *Atriplex*) established with almost 100% success with a very rapid growth rate. Plant volume increased significantly among the woody species, with *Atriplex lentiformis* and *Atriplex halimus* showing maximum shoot volume. Tree species (*Acacia ampliceps* and *Conocarpus lancifolius*) also exhibited a high survival rate and rapid growth rate, compared to other tree species tested (Figure 10).

Soil salinity measurements have been conducted periodically in reference to the different plant species and production systems. As mentioned earlier for the grass species plot, high wind velocity prevented uniform application of irrigation. This resulted in lower leaching of salts from the top soil surface, where the soil salinity at 0-20 cm was more than twice the salinity of irrigation water. Tree species in forestry and agroforestry blocks did not show any significant accumulation of salts up to 40 cm soil depth. Halophytic shrubs (*Atriplex* spp) also did not exhibit any accumulation of salts in the rhizosphere which were irrigated with bubbler.

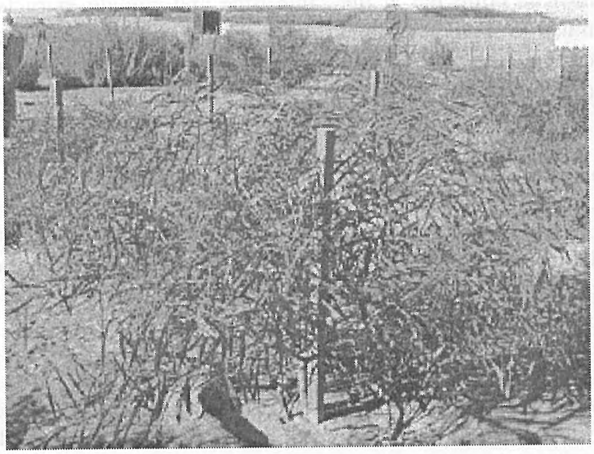


Figure 10: Acacia and Atriplex species thriving on treated process water.

CONCLUSION

The work described in this paper summarizes the work performed over three years aiming at re-using polluted oil process water in biosaline agriculture. This would substitute for the expensive process of injecting this water back into deep aquifers. Them design and sizing of wetland treatment systems growing reeds (Phragmitis australis) was based on probability analysis of inflow and outflow data. Estimates of loading rates were determined as such and were verified in a newly constructed reed based system. Treated water was then used to grow different plant species that tolerate saline water. These species include grasses, trees and shrubs which can be used for fodder and wood production.

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ABSTRACT

This paper intends to discuss the problem of produced/effluent water associated with oil production in Kuwait Oil Company (KOC) and to explain the company's approach in meeting the targets of its 2020 strategy, where the management of produced/effluent water should aim for more environmentally-friendly processes. The administration of Kuwait Oil Company commits itself to fully implement its HSE policy which ensures the health and safety of its people and to protect the environment through strict adherence to its own policies and procedures and by complying with all local regulations and international oil industry guidelines. Produced/effluent water is by far the largest single waste type in KOC operations. It is complex in its composition varying from salts, dispersed oil, suspended solids, traces of chemicals added during oil production and separation processes, dissolved mercury, heavy metals, hydrocarbons, sulfur (SOx) and nitrous oxides (NOx) and heavy metals. In addition to volumes of produced water from the reservoir, there is also an effluent water stream from desalter systems, where wash water is used to reduce the salt content of the crude oil. Almost all of this effluent is disposed into surface evaporation pits which represent potential risks to ground water supplies. Moreover, this is considered a safety hazard and may cause pollution to the surrounding environment where the emission could be a concern. The expected effluent water discharge to evaporation pits for 2003/04 is over 300 MBWPD. One of the appropriate and recommended solutions for managing the produced water is through the "underground injection" of the appropriately treated effluent water. In addition to that, the "processed sea water injection" for pressure maintenance in the oil reservoirs. Environmental Impact Assessment studies for most of the injection well projects have been carried out as part of HSE environmental management practices in compliance with the Kuwaiti Environmental Protection Agency (EPA) regulations. As an objective of the sustainable development in the company, all feasibility studies will be carried on for some other alternatives for management of the evaporation pits.

Keywords: Kuwait Oil Company - Produced Effluent Water - Environmentally Friendly Management - HSE Policy - Disposal Evaporation Pits - Underground Injection – Environmental Impact Assessment

Introduction

The volume of produced water typically increases with oil and gas fields' age. In that context, the worldwide volume of produced water was approximately 1.1 times that of hydrocarbon production in 2003 in more than 29 oil and gas companies in the world [1]. Produced water is a salty wastewater that is brought to the surface during the production of natural oil and gas, and it is one of the major issues for Kuwait Oil Company (KOC) to manage. It is not only a concern for safe disposal but also an issue of conservation and recycling and reusing. The approach of present water management development is one of the most important targets of KOC and GCC countries. The success of this development depends on the adoption and implementation of comprehensive policies and strategies for management in all its sides and targets by exchanges of experiences.

Kuwait, as one of the world's major oil producers, goes by KOC as an Oil and Gas Exploration and Production Company. KOC is producing a good amount of wet crude and thus faces a wastewater problem that had been handled until recently through the only available mean of disposal-evaporation pits. Accordingly this paper will discuss the effluent and produced water treatment and management practices in KOC in the past, current status, and the future vision. The approach will be through identifying and investigating problems associated with the current solution, potential HSE impact, technical views, KOC strategy and detailed feasibility study of the entire water project.

Background

Produced water, formation water or brine as it is sometimes referred to, is comprised of water containing residual hydrocarbons, heavy metals, radionuclides, numerous inorganic species, suspended solids and chemicals used in treatment and hydrocarbon extraction. It is the accompanying product of the cleaning process of raw crude from the well head. Figure 1 shows the typical process layout. The issue of the effluent water has become very significant after 1990 where the devastation of the oil field was the result of the Iraqi Regime invasion on Kuwait.

Raw production, from the wellhead, comes typically in the form of a mixture of free water, oil/water emulsion, oil and solids. This combination is also referred to as BSandW (basil sand and water) and oil. The raw production from the wellhead is then piped to gathering points. From there it is piped to the production facility.

This raw production goes through separation where the free water and loose solids are separated from the remaining oil/oil-water emulsion and stored in the produced water tanks. The remaining oil/oil-water emulsion then proceeds to the desalter vessels where a combination of heat and chemicals (emulsion breakers) are used to break the emulsion and produce clean oil and produced water and solids. Clean oil then proceeds to storage and/or shipping. The produced water from the desalter is transferred to tanks to hold for disposal to pits in later stages. Depending on residence time in the tanks, some of the solids may settle out of the water and residual oil in the water floats to the surface. This oil layer is skimmed off the top and recycled through the plant to recover this additional oil. After final oil separation, the water is discharged to disposal evaporation pits or re-injected down a hole (evaporation pit is a typical hole surrounded by sand shield/bund walls, a typical size of the pit is approximately 200 by 200 meters in length and 3 to 5 meters in depth). Technically as oil wells production mature with time the ratio of water to oil increases. Water becomes a significant byproduct of oil and gas production. The produced water is disposed of by either re-injection or by discharge into the environment.

The history of water production in KOC is illustrated in Figure 2 which indicates that the produced water increases rapidly while there was a sharp fall in the produced water quantity in 2003 (106,554,868). On other hand, year 2004 showed a remarkable increase in the quantity of produced water [2].

Produced Water Quality and Impacts

The produced water occurs with approximately 34,528 ppm salt content and a high level of suspended solids. This highly saline, hence corrosive water goes from the process facilities into wastewater pits for disposal via surface evaporation. Since 1982 about 50 evaporation pits have been constructed in KOC fields mostly around Gathering Centers (GCs))a Gathering Center is an oil production facility with different functions such as serving as a central collection gathering location of crude oil, reducing the crude oil pressure to zero (atmospheric) and processing crude to separate gas, water and salt to accommodate the effluent water). The basic idea for the evaporation pits is collecting effluent water in pits and leaving it for evaporation in hot summer conditions. This idea has some difficulties where evaporation rate is hindered by the presence of oil on the surface and which present on the top of evaporation pits. The oil from the surface is sometimes skimmed off. The other negative aspects of disposal pits are related to safety and ecological hazards. These pits are a danger to humans, birds and wildlife considering that the pits have expanded well beyond their boundary fences. Moreover, these pits are potential fire hazards as the case in the recent GC-4 accident, in South-East Kuwait. Other problems could occur when produced water exceeds pit design capacity or size, especially during adverse climatic conditions of low temperature and high relative humidity. Another crucial environmental impact is that when disposal pits are not lined, seepage of effluent water to the subsurface occurs. In addition, the pits bund walls are usually sprayed with heavy crude or bitumen to generate a semi impermeable layer to assist with bund stabilization. There is some concern that other dangerous volatiles, dissolved oil (benzene), injected chemicals, etc. would also be evaporated along with the effluent water potentially causing health problems in the vicinity of the pits. Figures (3a, 3b) show the Health Safety and Environment (HSE) impact of disposal pits in the south fields in Kuwait [3].

The probability of groundwater contamination is another concern, considering that Kuwait has limited fresh water reserves. Evaporation of the lighter fractions of oil from these pits is a potential source for air quality contaminants, which is expected to considerably increase the hydrocarbon concentration in the atmosphere. In this regard and due to all the direct and indirect impacts and potential hazards, it was decided to gradually eliminate such pits, effluent water disposal should be through underground injection wells and implement recommended remediation measures through a research project.

Produced Water Management Solutions

The injection wells in KOC fields are located in the southern and eastern parts of Kuwait, those in the west and north Kuwait are defined as "dead wells" and have been used for the disposal of the produced water. The KOC projects include the construction of two effluent water disposal plants in the SEK Fields, to manage effluent water collected from more than 13 gathering centers. The collected effluent water to be injected shall be treated for Total Suspended Solids (TSS) and oil removal, and ultimate disposal into allocated disposal wells. All effluent water disposal plant locations have been carefully selected so as not to impact the fresh water aquifer, thus avoiding any possible short or long term contamination.

Effluent Injection Techniques

The main scope of this technique is to use injection wells (deep wells into which water or pressurized gas is pumped in order to push petroleum resources out of underground reservoirs toward production wells, so as to increase their yields) to dispose effluent waters through injection beneath the lowermost water aquifers. Injection occurs into deep, isolated rock formations that are separated from the lowest underground water aquifer by layers of impermeable clay and rock. Therefore, as an HSE policy it is essential that we evaluate the technique and assess its effectiveness in details, to ensure its feasibility, safety and validity as an innovative environmental management approach [4, 5].

Technique Principles

Water injection is typically used as a water management technology for two main reasons:

1) Disposal of water recovered from the raw crude oil/water emulsion separation process.

2) Enhancement of oil recovery by maintaining formation pressure and displacing the crude oil in the reservoir.

Subsurface injection is the primary method for the disposal of produced water for land-based oil and gas operations. Produced water may be re-injected for disposal to shallower saltwater formations, or re-injected to older, depleted producing formations. By injecting the water into the producing formation, (Water Flood) well pressure and product flow is maintained by displacing the produced oil.

In this technique, wastewater is injected into brine-saturated formations thousands of feet below the land surface, where they are likely to remain confined. The geological formation, into which the wastewater is injected known as the injection zone, is sufficiently porous and permeable so that the wastewater can enter the rock formation without an excessive build up of pressure. The injection zone is overlain by a relatively non permeable layer of rock, known as the confining zone, which will hold injected fluids in place and restrict them from moving vertically towards the underground water aquifer. Effluent injection wells, with their sophisticated construction have many redundant safety features. The well's casing prevents the borehole from caving in and contains the tubing constructed of corrosion-resistant materials such as fiber glass reinforced plastic. The casing consists of an outer surface casing, which extends the entire depth of the well, and an inner string casing that extends from the surface to or through the injection zone. The innermost layer of the well, the injection tubing conducts injected sea wastewater from the surface to the injection zone. Normally, trained operators are responsible for day-to-day injection well operation, maintenance, monitoring and testing.

However the purpose of some of these wells in the west and north Kuwait is for increasing reservoir pressure such as (Managish Water Injection Plant) and seawater injection plant where injection of processed seawater is one of the techniques used for oil reservoir development; such water in some cases does not meet the specifications for injection and therefore, is disposed of in specially prepared pits. Due to leakage from these pits and overflow there is a potential of pollution of the fresh groundwater by the off-specification seawater. Accordingly, KOC requested KISR (Kuwait Institute for Scientific Research) to investigate the potential problem and to recommend remediation measures [6]. From the analysis results, it was clear that the seawater in the pits had significantly reached groundwater in some areas identified with depression and during the first few months the seawater went deeper in the aquifer. However, after stopping the seawater discharge in the pit as a decision from KOC management, the quality of the groundwater started to recover in general. The number and location of injection wells are illustrated in Table 1 within the south, east, west and north Kuwait fields. It is also explains the different types of rock formations and depth of the wells, the table also shows the commencement date of the wells.

One of the major tasks in the injection system is conducting complete analysis including chemical and physical properties for all water samples prior to injection which is a critical piece of data for the design. Moreover, water quality and injection performance are monitored on a continual basis as a routine part of operations. The analyses are conducted at KOC, the Gathering Centers and at the Managish Water Injection Plant in KOC which have laboratory facilities for sampling and analysis of the water, as well as using the Kuwait Institute for Scientific Research (KISR) facilities, personnel and laboratories for additional analysis through service contracts. Table 2 illustrates an example of effluent water and Table 3 illustrates an example of effluent water injected to wells analyses in year 2003.

Further to that, and as part of legal compliance Environmental Impact Assessment (EIA) studies have been carried out for most of the injection well projects in order to identify short and long term potential impacts and to recommend all mitigation measures side by side with feasibility cost benefit measures [5]. All these activities fulfill the Environmental Management System (EMS) which is a major milestone of the KOC HSEMS (Health, Safety and Environment Management System). The system assures compliance to Kuwaiti Environmental Protection Authority (KEPA) and Ministry of Energy (MOE) regulations and assists in the avoidance of long term liabilities [4].

The administration of KOC is committed -as a part of its HSE 2020 Strategy- to remediate all such evaporation pits by the year 2012 [4]. However, the laws in Kuwait grant the KEPA an ultimate control authority to define environmental goals, and approve proposed means to attain these goals.

Strategically, the disposal of produced water through injection wells are gradually increasing every year as illustrated in Figure 2 and Table 4, where in the year 2000 disposal to injection well was only 18%, in 2001 it was 19%, and in 2002 it was 20%. According to the action plan at KOC the disposed water to injection wells is significantly increased in years 2003 and 2004 where it reached 32% and 61% respectively. On the other hand, the disposal to evaporation pits had significantly decreased from 82% in the year 2000 to 36% in 2004. With the same rate of progress we can achieve our goal of zero discharge to evaporation pits according to the KOC 2020 strategy scenario [4].

Effluent Water Management Economics

The economic data of most injection plant projects was collected in 2000, where the capital cost reported for installation of seven effluent water injection pilot plants was around US $3,670,000. Divided as following: south Kuwait was reported to run $2,056,667 for four Gathering Centers; north Kuwait was reported $306,667 for two Gathering Centers and west Kuwait was reported US $306,667 for one Gathering Center. On other hand the capital cost reported for installation of the Managish Water Injection Plant which is located in west Kuwait was US $115,663,333 for the plant and field system and $44,083,333 for capacity expansion for increased effluent water separation, transfer and upgrade. Adding to that the capital cost amount of US $442,643,333 for installation of effluent water disposal plants and effluent disposal wells in south and east Kuwait field's phases 1 and 2. As a conclusion the front money investment for such strategy is an expensive step costs KOC more than US $606,060,000 in order to protect the environment and to enhance the ecosystem in its operation's fields.

Strategy Forward

Underground disposal of effluent water is one aspect of effluent water management; however by this technology we are not conserving the water. Their are many aspects in which KOC is working to conserve this effluent water, and reduce the amount of produced water as a whole or minimizing source by studying the geological/ecological/ economical feasibility of the technology targeting for the underground separation of the water from the oil and pumping of the produced water down hole, without bringing it to the surfaces. On other hand, other technologies of water management are worth considering such as bio-saline irrigation.

Conclusions

Produced water is the highest volume liquid waste generated and discharged during the production of oil and gas. Discharge of produced water is regulated in most countries. Regulations usually vary from region to another. Differences in guidelines and regulations of protection of the underground and on the surface reflect environmental concerns. For example, TSS can be an important aspect where discharge goes to injection wells. These factors are less important for evaporation pits discharge.

In Kuwait Oil Company HSE guidelines environmental issues are on the top priority list of the business guidelines aiming to become a leader in the region in the implementation of the HSE practices. Accordingly, there are several long term solutions/techniques available in industry to dispose wastewater, some of which are expensive and time constraining but will resolve disposal problems permanently, and minimize environmental and ecological hazards.

The most appropriate and recommended solution for produced water management is underground injection after it is exposed to primary treatment. From the KOC investment records, KOC had spent more than US $606,060,000 toward implementing these technologies for empowerment of the Environmental Performance which is reflecting the reputation and image of KOC among International Oil Companies (IOCs). Till the point of completion of those projects, a short-term solution was adopted to utilize lined-controlled evaporation pits, where produced water after separation from oil is released into these pits. Some of the short and long term impacts are significantly reported, such as fatal attraction to surrounding ecosystems, sever pollution to landscape and potential adverse impact to the underground. After all, minimizing these pits and all related consequent potential impacts will be the major effort towards compliancy to KEPA and MOE Regulations.

All these problems are of great concern and definitely cannot be ignored. They need an immediate solution before the matter complicates any further. Some significant solutions that have been developed worldwide include:

- Technological enhancement- Improved technology like the dual pump system, which separates the oil underground, thus, avoiding the need to bring up the water to the surface in the first place.
- Better injection systems, which are used for enhanced oil recovery.
- Existing pits are to be operated judiciously with efficient lining and control systems, oil separation techniques, and regular skimming off of all existing oil. Treatment of the oil contaminated soils is another concern for such strategy.
- Beneficial use of treated produced water for other purposes, like irrigation, etc., along with periodic removal of settled solids.

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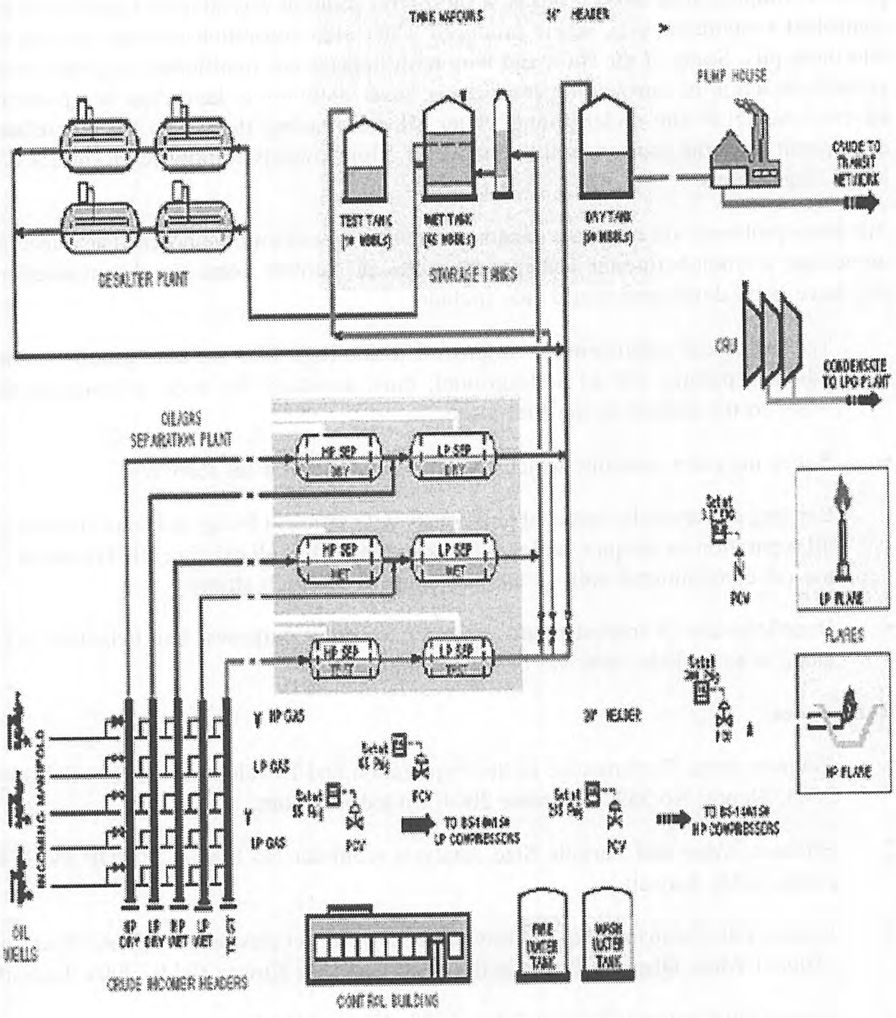

Figure 1: KOC Gathering Centre Schematic

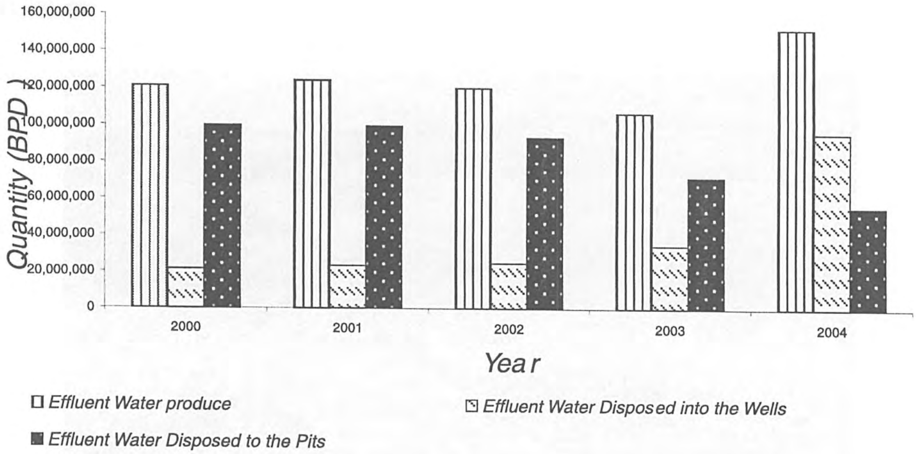

Figure 2: Total Produced and Disposed Effluent Water in all KOC Area

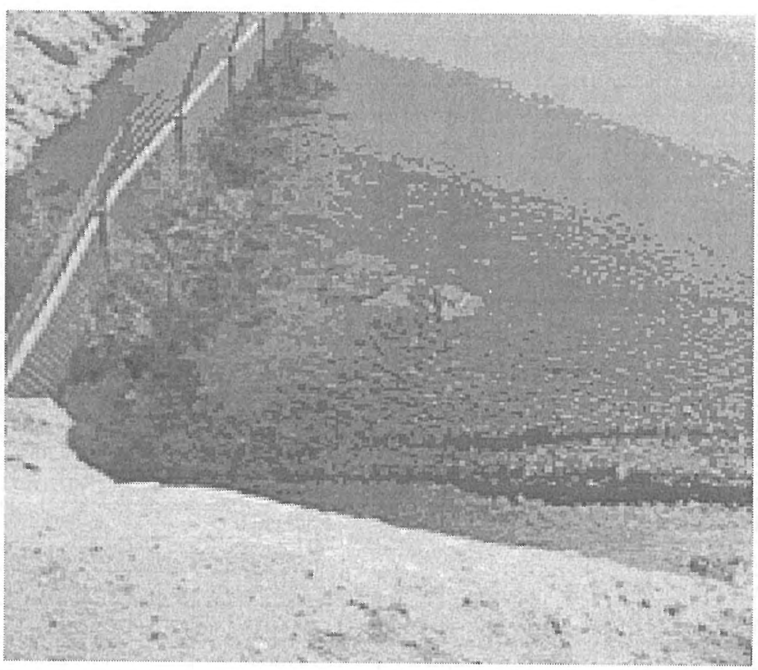

Figure 3a: HSE Impact of Disposal Pits in South Kuwait

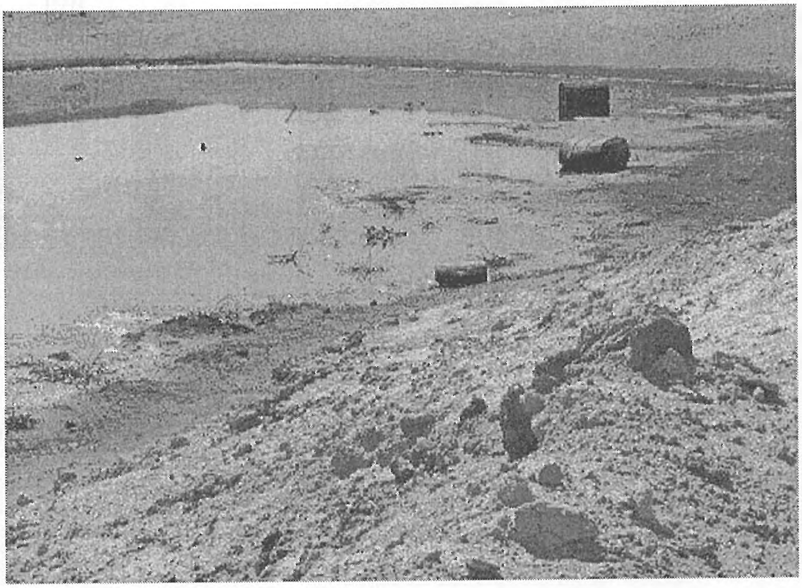

Figure 3b: HSE Impact of Disposal Pits in South Kuwait

Table 1: KOC Injecting Wells Information

Well No.	GC. No.	Depth (Ft.)	Formation	Commencement Date injection to Well
BG-502 (T)	2	5,775	Shuaiba	7/5/2003
BG-S6		2,900	Radhuma	12/19/1994
BG-502	6	5,775	Shuaiba	9/2/1996
BG-501	21	2,950	Radhuma	8/15/1998
MG-179	10	2,960	Radhuma	9/5/1995
UG-37	17	6,700	Shuaiba	1/15/2000
UG-76		6,676	Shuaiba	9/5/1995
UG-120		6,700	Shuaiba	1/5/2000
MWIP				7/3/2003
UG-123	27		Shuaiba	3/23/2002
UG-124			Shuaiba	3/23/2002
WW-215	15	3,630	Radhuma	10/13/1998
SA-96	23	4,600	Tayarat	4/22/1995
SA-175		0	-	0
SA-185		0	-	0
RA-65	25	5,000	Tayarat	11/12/2000
RA-130		4,920	Tayarat	9/10/2000
WW-215		3,630	Radhuma	

Table 2: Effluent Water Analysis

Element	Unit	GC1	GC2	GC3	CG4	GCe
Date	mg/l	28-Sep' 03	29-Sep	29-Sep 03	01-Oct 03	28-Sep 03
Na	mg/l	38929	35444	29301	35645	33321
Ca	mg/l	8801	8045	6589	7958	7670
Mg	mg/l	2001	1838	1598	1915	1782
K	mg/l	1880	1610	1367	1681	1612
Sr	mg/l	226	199	167	212	194
Ba	mg/l	1.9	1.55	1.16	1.87	1.47
Fe	mg/l	1.3	5.10	1.30	1.1	2.20
Mn	mg/l	1.2	1.10	1.20	1.9	1.10
Li	mg/l	3.82	3.61	2.65	4.24	1.47
Al	mg/l	2.3	2.45	2.48	2.50	2.40
Si	mg/l	5.07	5.28	5.40	3.85	4.9
B	mg/l	31.61	28.68	23.15	33.86	25.72
Si02	mg/l	10.80	11.25	11.50	8.20	10.44
Cl	mg/l	76480	69550	56425	70565	68215
Fluoride	mg/l	5.2	4.8	4.9	4	3.9
C03	mg/l	<0.1	<0.1	<0.1	<0.1	<0.1
HC03	mg/l	268	195	287	179	231
OH	mg/l	<0.1	<0.1	<0.1	<0.1	<0.1
S04	mg/l	150	220	370	170	260
P04	mg/l	0.84	0.1	1.01	0.6	0.1
IIN02+N03)	mg/l	0.2	0.2	0.2	0.2	0.1
NH3-N	mg/l	48	46	44	43	46
Oxygen		<0.01	<0.01	<0.01	<0.01	<0.01
CO2	mg/l	171	197	183	114	147
H2S	mg/l	<0.1	<0.1	<0.1	<0.1	<0.1
pH		6.65	6.43	6.75	7.151	6.47
Temperature .C	•C	34.5	37.2	38.00	32.9	37.8
Specific gravity I@ 60.F		1.09228	1.0837	1.06961	1.08515	1.08095
Viscosity @70 F	cP	1.09774	1.00286	0.89355	1.05802	1.05068
TDS (calculated)	gm/l	128.80	117.16	96.16	118.39	113.34
Conductivity mS/cm	mS/cm	166.9	168.7	135.2	141.1	149.3
Turbidity	NTU	19.6	9.06	14	112	113
Oil/Water	mg/l	53	119	15	647	12
TSS, (0.45um) Total	mg/l	18.21	8.35	14.31	98.36	30.38
TSS, (0.45um), Oil free	mg/l	4.37	2.20	3.58	1.60	9.01
TSS, (3.0um) Oil free	mg/l	4.16	1.81	4.35	N/A	3.82

Table 3: Effluent Water Injected to Wells Analysis Compared to Standards

S.NO.	Parameter	Units	Upper Limit for injection	Date / Time	8/9/03 09.50 Hrs.	8/10/03 09.15 Hrs.
1	**pH**	At 25 C	NOT SPECIFIED		6.28	6.53
2	**Temperature**	Deg C	NOT SPECIFIED		53.5	54
3	**Conductivity**	Micro Siemens/Cm	NOT SPECIFIED		278000	328400
4	**Turbidity**	NTU	NOT SPECIFIED		4.67	10.7
5	**Total suspended solids**	ppm	5		6.91	13.5
6	**Total Dissolved Solids**	ppm	250000		194600	229880
7	**Oil in Water**	ppm	10		7.94	146
8	**Total iron**	ppm	30		0.1	0.1
9	**Chloride**	ppm	NOT SPECIFIED		Nan	NAN
10	**Carbon-di-Oxide**	ppm	NOT SPECIFIED		295	240
11	**Sulphide (as S)**	ppm	NOT SPECIFIED		30	20
12	**Dissolved Oxygen**	ppb	5		0	0
13	**Specific gravity**	-	NOT SPECIFIED		-	NAN
14	**Particle Size Distribution in 0.1 ml**					
1	**Above 2 Microns**	Numbers	NOT SPECIFIED		18542	9995
2	**Above 3 Microns**	Numbers	NOT SPECIFIED		6102	4171
3	**Above 5 Microns**	Numbers	200		778	743
4	**Above 8 Microns**	Numbers	75		129	77
5	**Above 10 Microns**	Numbers	25		39	15
6	**Above 12 Microns**	Numbers	20		6	2
7	**Above 15 Microns**	Numbers	10		1	1
8	**Above 20 Microns**	Numbers	0		0	0

Table 4: Quantity of Produced and Disposed to Injection Wells Water 2000 - 2004

Year	Effluent Water produced Bbls	Effluent Water Disposed into the Wells Bbls
2000	120,898,582	21,379,770
2001	123,866,776	23,285,914
2002	119,780,601	24,539,318
2003	106,554,868	34,198,060
2004	151,948,995	95,754,199

PDO Water Management Approach in Treating and Reusing Production Water Using Reed Beds and Water Shut-off Technologies

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PDO WATER MANAGEMENT APPROACH IN TREATING AND REUSING PRODUCTION WATER USING REED BEDS AND WATER SHUT-OFF TECHNOLOGIES

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ABSTRACT

Petroleum Development Oman (PDO) is the largest oil producing company in the Sultanate of Oman. In the early stages of oil production oil fields produce very little or no associated water. With time however increasingly more water is produced along with the oil. Currently PDO produces more than 650,000 m^3/day of water along with its oil production. Disposing of such large quantities of water poses a great challenge in that, initially water needs to be treated and subsequently disposed of in accordance with environmental regulations. An estimated 40% of the water is injected back into the oil producing reservoirs for pressure maintenance. The remainder of the water is largely disposed into deeper reservoirs. The main priorities in managing the water are to minimize the produced volumes and to reuse the water in a beneficial cost-effective manner. In addition to injecting water for reservoir pressure maintenance PDO has embarked on novel technologies like mechanical and chemical shut-offs to reduce the amount of water produced from wells. PDO also experimented reed bed technology and bio-saline agriculture.

Keywords: Pressure maintenance, water disposal, producing reservoirs, dehydration water, environmental challenge, environmental impact, reed bed technology, water shut-off technology.

1. Introduction

Currently PDO produces more than 650,000 m^3/d of water along with its oil production. An estimated 40% of the water is injected back into oil producing reservoirs mainly in oil fields in the north in order to maintain reservoir pressure and boost oil production. Recently, the practice has been extended to various other fields. There is however surplus dehydration water which is largely disposed into deeper reservoirs.

Water and oil exist in the subsurface as separate entities. Oil normally floats on water due to differences in density. Oil wells are completed in the oil leg above the water horizon. Subsequent production however results in pressure depletion of the oil reservoir, resulting in the up-coning of water from the water leg to rise and invariably mix with the oil. Continuous production in this mode causes the water percentage to increase and eventually overtake the oil, until such a time comes when oil production is no longer economical, at which point wells are shut-in or completely abandoned. There are however means to avoid arriving at this end point. Nowadays many methods are available to minimize water production by using various technologies.

Dehydration water if not handled properly poses a great environmental challenge in an oil producing country like Oman. In PDO much effort has gone into researching methods of getting rid of produced water. Initially basic research centered on understanding the physical and chemical properties of the dehydration water and potential impacts on the environment. Numerous options of getting rid of the water were explored including downhole separation, recycling or reusing the water and if all means were exhausted then disposal into deeper reservoirs from which the water originated became the only viable option. The largest governing factor in the choice of water disposal is the cost. PDO has relentlessly tried to use different technologies to overcome the challenge using five guiding principles:

1. Minimize the volumes of water produced during oil extraction.
2. Maximize the reuse of such waters.
3. Phase out the use of shallow disposal wells and prevent disposal into usable or exploitable aquifers.
4. Return the water to the producing reservoir.
5. Dispose surplus water to formations which have salinity >35,000 mg/l, in conjunction with case specific monitoring programmes.

It is acknowledged that there is no unique solution to the produced water problem; the optimal solution depends on actual field conditions like the volume of produced water, need for pressure maintenance, quality of the water stream and local opportunities for reuse. PDO has piloted a variety of technological opportunities for disposing produced water as an alternative to deep water disposal. An overview of some pilots is given and an emphasis is put on those that most effectively reduce the environmental impact of produced water discharge like reed-bed technology.

2. Aquifers of Oman

Water movement within the PDO concession has been well studied as a result of drilling numerous water supply and oil wells. It is well established that since Oligocene to Miocene times to the present day there have been two major recharge systems; one in the north (Oman Mountains) and one in the south (Dhofar Mountains) (**Figure 1**).

The Oilgocene/Miocene age is associated with the onset of mountain building both in the north and south of the country. Since that time it is envisaged that recharge into Tertiary and deeper aquifers have continued in the same manner until the present day. Water flow in the Oman hydrogeological basin is gravity induced and topographically driven. The flow of meteoric water into the formation is through highland recharge, and the discharge is at relatively low relief areas. Recharge takes place over the mountains into mainly Tertiary rocks initially and subsequently through seal breaching into deeper horizons (Figure 1).

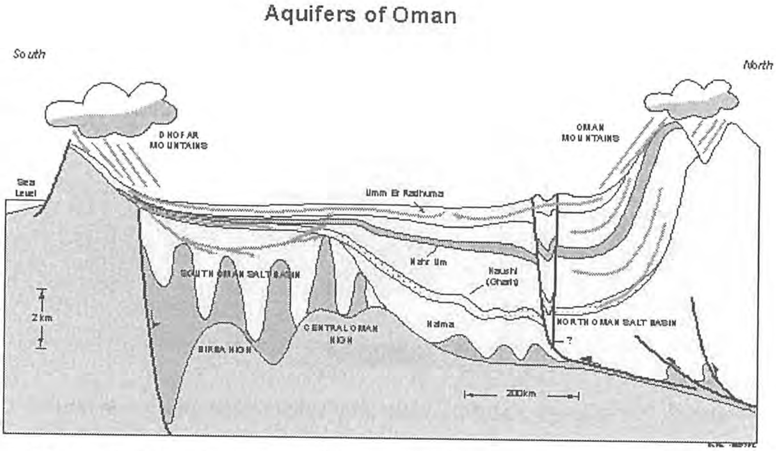

Figure 1: North -south hydrostratigraphic cross section across the Oman hydrogeological basin.

Recharge over the Dhofar Mountains is the most important, in that it is estimated that as much as 300 mm of misty precipitation takes place during the monsoon period (July-September). Precipitation over the Oman mountains to the north mainly occurs during winter months and is estimated to be 150 mm. Towards the end of the last pluvial period, 10,000 to 20,000 years ago, Oman was much wetter than it is today (Beydoun, 1980). Therefore more infiltration into the groundwater system was likely to have taken place. The bulk of the aquifer water volume is believed to have been recharged during this period. Two main groups of aquifers are recognized; the shallowest and currently most active aquifers are the Tertiary carbonates (approximately 500 m thick).The Tertiary interval comprises two aquifers: the Fars/Dammam and the Umm Er Radhuma, underlain by the Shammar shale layer. The thickness of the Shammar rarely exceeds 30 m and its continuity is questionable especially towards the southern Dhofar Mountains. The second main aquifer group comprises the fluvioglacial and essentially continental deposits belonging to the Haushi and Haima groups (Figure 2). It so happens that the Haushi and Haima sandstones are also significant hydrocarbon reservoirs.

The direction of flow is manifested by variations in total dissolved solids (TDS). Figure 3 & 4 confirm that the main recharge areas are the Dhofar Mountains to the south and the Oman Mountains to the north. The main discharge area is the inland

Umm as Samim sabkha. This is the area where water stagnates and groundwater under such conditions is known to exhibit high TDS.

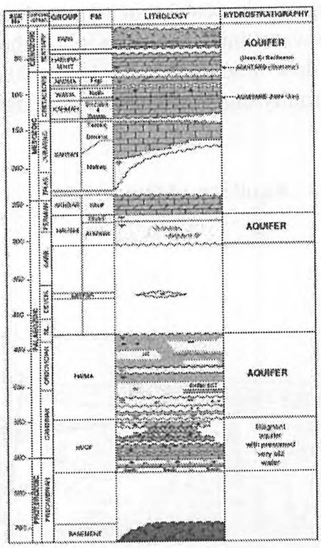

Figure 2: Stratigraphy and hydrostratigraphy in the PDO concession

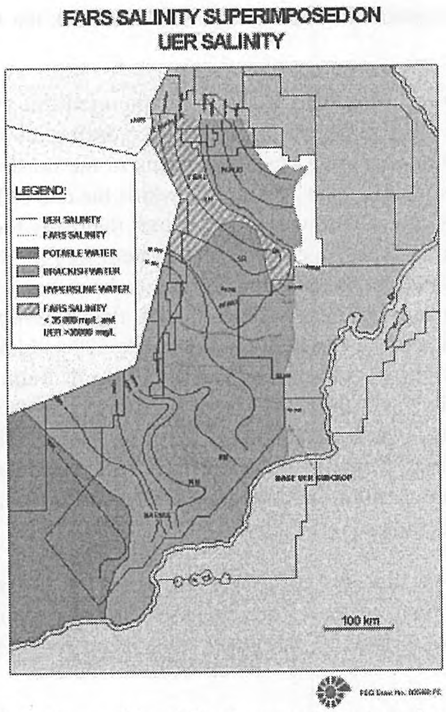

Figure 3: Salinity profile of Tertiary aquifers to a depth of 600m

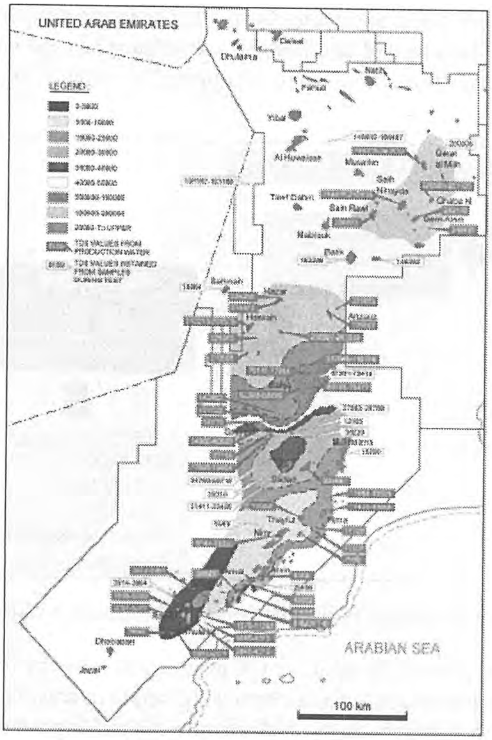

Figure 4: Oil producing reservoir salinity between 1000 and 2500m

3. Reed Beds

Deep water disposal is environmentally friendly but not directly beneficial to the community. Armed with water movement and quality understanding PDO embarked on projects that would be beneficial to the community e.g. reed bed cultivation and saline agriculture. In recent years PDO produced around 650,000 m^3/day of water and the trend is set to continue. Among the measures adopted to alleviate the problem was the cultivation of reed beds. The area chosen for the experiment was in one of South Oman oil fields with relatively low, 6000 mg/L, TDS saline production water (Figure 4).

Reed beds are artificially engineered wetlands used to clean water from pollutants. The reeds are specially adapted waterlogged plants with an extraordinary ability to use atmospheric oxygen in order to create both aerobic and anaerobic conditions in their roots. A variety of specialist bacteria and fungi flourish well in such conditions, their source of food is normally pollutants which include a wide range of organic compounds like hydrocarbons. The breaking down of such compounds normally results in harmless ones. Cultivation of reed beds has therefore been established as an important technique of ridding production waters of hydrocarbon.

The southern area was chosen because the field produces comparatively large amounts of water (200,000 m^3/day) and the water salinity although not potable was considered not high enough to destroy any chances of the reeds flourishing.

The source of reed beds water is directly from oil production dehydration tank facilities. The water is then pre-treated and de-oiled, but experience has shown that traces of oil in the water always remain sometimes up to 300 ppm. The water is then fed into the reeds directly (Figure 5).

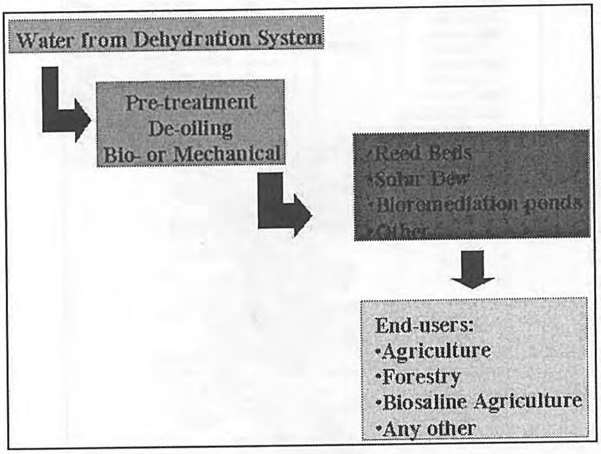

Figure 5: Greening the desert flowchart from the production station to the end-user

Since 1999, PDO has piloted the treatment of production water by reed beds. The current experimental site consists of some six hectares in the southern area. There are two treatment trains, each consisting of four beds; the beds in train-A are designed for testing throughput of water and efficiency for the removal of oil and heavy metals from produced water followed by bio-saline agriculture trials; train B is designed to maximize volume reduction (Figure 6). Effluent water from the reed beds is collected in two evaporation ponds and a spray-evaporation pond. These ponds feed plots for bio-saline agriculture. Although an established method for treating industrial wastewater, reed bed technology is a novel treatment process for produced water. Reed plants are halophytes that grow well in saline environments. Operations have demonstrated that plant growth may sustain even in desert environments and that natural processes in the reed bed degrade residual oil.

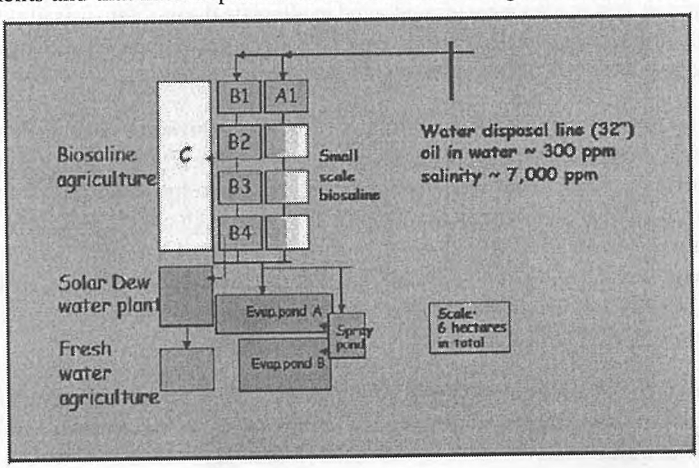

Figure 6: Overview of southern fields reed beds experimental plot

3.1 Reed bed performance

Through sampling and analysis, performance has been monitored and evaluated on the basis of key performance parameters, such as effluent quality, capacity to treat oil-contaminated water, integrity and logistics, crop health, as well as cost of ownership.

1. **Effluent quality**: If the effluent water is to be used for agriculture, reed beds should be designed to maximize oil reduction and minimize evaporation of water to maintain the water salinity as low as possible. The inlet oil-in-water concentration is about 200-300 ppm (water from Production Station), with a salinity of 6-8,000 ppm, and the oil concentration from the outlet in the range of 0-5 ppm and salinity is around 10,000 ppm.

2. **Oil treatment capacity**: Based on chemical analyses over a 6-month period in 2003, a capacity of oil treatment was calculated to be 17 ml oil/m^2/d.

4. Biosaline Agriculture & Forestry

Once the reed beds have removed the hydrocarbons, two limiting factors for crop selection are the salinity of the water and boron content. The salinity of the water after biological treatment could range between 6,000 and 11,000 ppm; boron would have a value between 4 -7 ppm, and the OIW content would be less than 5 ppm. Boron may be a limiting growth factor for certain agricultural options, although species that are salt tolerant can generally also tolerate high boron concentrations. Boron removal at the present time is not an economic viable option, although technically feasible.

A PDO imperative is that produced water should not enter the food chain, thus crops for human consumption, or fodder crops are excluded from the list of water use options. Instead the effluent of the reed beds may be used for forestry, e.g. to produce fibers as construction material.

A forestry system should be adjusted to local conditions, e.g. to soil depth. Due to a shallow depth of the soil profile, and the risk of salinization, a subsurface drainage system similar to that in the reed beds should be considered. A drainage system will remove accumulated salts from the soil to be collected in the evaporation ponds.

A fairly constant supply of water may be obtained from the reed bed as feed for a forestry system (Figure 7). The water supply should match the variable demand by trees, as a result of seasonal variations. Reservoirs may help to match the water supply and demand; however, salinity rises from evaporation would have an adverse effect on growth. The requirement for water storage is minimized noting that trees may grow with less water than under optimal watering conditions during the hotter months, whilst during cooler months; a larger area can be irrigated at the capacity of the tree uptake.

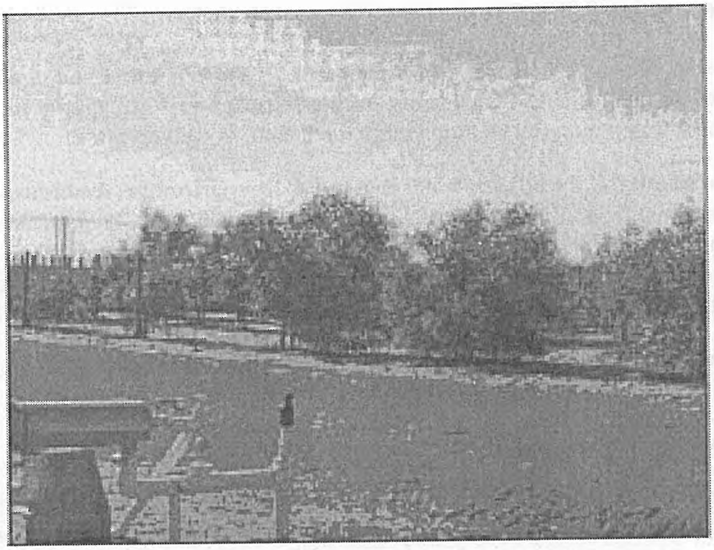

Figure 7: Desert forestry using saline irrigation

A large-scale reed bed system may be sized using the design data presented in this paper, i.e. remediation potential of 17.1 ml oil per day per square meter of reed bed allowing an outflow up to 5 ppm OIW. In order to treat 45,000 m^3/day by reed beds, with an average OIW content of 250 ppm in the feed, approximately 65 ha of reed beds would be required. The reed beds effluent would be used to irrigate a eucalyptus forest of about 1200 ha. It is noted that the proceeds from forestry to recover costs will come at the long term as it takes time for trees to grow (~ 12 years).

5. Water shut-off technology

In addition to reed bed technology, PDO has implemented water shut-off and profile control technologies aimed at reducing water production, and as a consequence reduce overall expenditure. An effective reduction in the volume of produced water reduces water treatment needs and the environmental impact considerably. Successful water shut-off can improve the sweep efficiency in natural water drive and water flooded reservoirs, thereby increasing oil production and ultimate recovery.

5.1 Chemical shut-off

To date, chemical profile control (CPC) technology to reduce unwanted water production has only been sporadically applied in PDO. Trials were carried out in various fields. Many of the trials were inconclusive or showed only limited reduction in produced water due to poor candidate selection, job preparation, execution, and follow-up. A review of the trial activities has led to a stronger focus and a more structured approach in PDO to fully tap the potential of this important technology. A promising CPC solution for reducing water production and its environmental impact is the newly developed gel-cement system for shallow shut-off of perforated intervals (Figure 8). The gel-cement system has been tried in the field and shows promising results under harsh downhole conditions (can withstand 3,000 psi differential pressure at 130°C BHT).

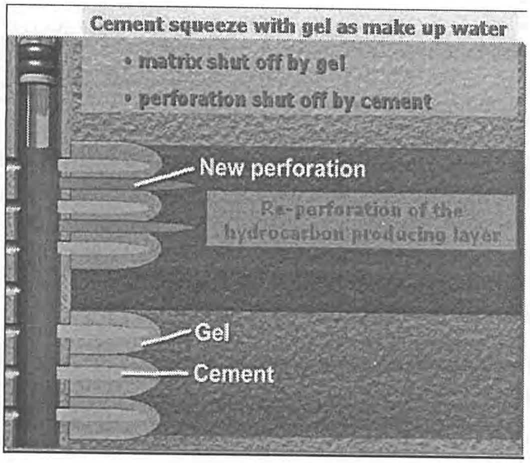

Figure 8: Principle of gel-cement shut-off system

5.2 Mechanical shut-off

Past cemented completions to isolate unwanted wellbore zones that bring in water into an oil producing wellbore have had limited success. In recent years expandable tubulars in conjunction with swelling elastomers have become practical tools in reservoirs that require zonal isolation, especially carbonate reservoirs. Expandable tubulars as the name suggests are capable of expanding once placed into a well in the subsurface to provide isolation wherever required. Together with swelling elastomers they form a dual sealing mechanism which provides an annular sealing of any desired length. This type of fracture shut-off has been tested in PDO's oil reservoirs and resulted in reducing water cut by 60%. PDO is continuing to pursue this technology in hope of reducing production water reaching the surface. For better well and reservoir management horizontal wellbores are divided into a distinct number of segments by swelling elastomer profiler tools. As soon as the water cut reaches an unacceptable level a well workover is carried out to identify the unwanted flow segments and shut them off.

6. Conclusions

Large amounts of production water have been and will continue to be a challenge for PDO's operations. All efforts are made to minimize the amount of production water reaching the surface using various shut-off technologies. However some percentage of water will inevitably be produced along with oil. Around desert oil fields opportunities arise to convert produced water to a usable resource through a combination of bio-treatment for clean-up and bio-saline agriculture. Research in the field has demonstrated that bio-treatment in reed beds is technically feasible as a pre-treatment of open field irrigation, thus enabling water reuse in agriculture or forestry. Studies have shown that environmental impacts, like soil salinization, can be managed.

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WATER, HEALTH & ENVIRONMENT

Assessment of Groundwater contamination by the 7th ring road landfill in Kuwait

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ASSESSMENT OF GROUNDWATER CONTAMINATION BY THE 7TH RING ROAD LANDFILL IN KUWAIT

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ABSTRACT

The scarcity of water resources in the State of Kuwait is evident. Such scarcity heightens the need for protecting the available water resources. A study was conducted to evaluate the impact of the 7th Ring Road landfill practices in Kuwait on the groundwater quality. The 7th Ring Road landfill is receiving liquid waste from different origins. For the purpose of characterization and monitoring, a total of 8 test holes and 5 monitoring wells with depths ranging between 25 and 30 m were completed. Samples were collected and analyzed from test holes, monitoring wells, existing wells in the landfill vicinity, and waste trucks in addition to other potential groundwater contaminating sources. Out of the 57 analyzed groundwater constituents, boron, nitrate, aluminum, lithium, fluoride, total petroleum hydrocarbons (TPH), total coliform bacteria (TCB), and TDS were found to be above the drinking water standards. Interpreting these results in the context of the prevailing conditions of lithology, upstream groundwater quality, sources and landfill operating conditions, only the elevated TPH and TCB in groundwater could be attributed to the landfill. The TCB plumes are extending well downstream of the landfills (i.e. more than three km), whereas the TPH, seems to be immobile. Reviewing the current regulations for land filling (EPA and municipality) and international guidelines, and building upon the information gained in this study, a set of guidelines were developed to ensure the safety of the groundwater quality.

KEYWORDS

Upstream; Groundwater quality; Plume; Total coliform bacteria; Total petroleum hydrocarbons.

INTRODUCTION

Kuwait is a small country with an area of 17,780 km^2; therefore, the climate of the country is considered to be the same in all parts including the studied landfill. The climate of Kuwait is hot in summers and mildly cold in winters. The mean annual precipitation is about 120 mm. Rainfall is strictly confined to the period from October to May. The average evaporation rate ranges between 3.0 and 16.0 mm/d (Al-Senafy, 2001).

In Kuwait, and in addition to the clear environmental need, the protection of groundwater takes a strategic angle as groundwater is the only natural water resource of the country. Despite the regulatory framework that considers proper land filling, hence protecting groundwater, none is provided as enforcement is absent. With waste generation in Kuwait vastly increasing in terms of both per capita and increasing population (Hamoda and Al-Yaqout, 2001), waste disposal is posing a serious threat to the environment that is likely to increase if the current practices continue. A limited number of studies were carried out on landfill practice in Kuwait. The aim of this study was to evaluate the impacts of the 7th Ring Road Landfill on groundwater quality.

METHODOLOGY

In order to achieve the objectives of this study, a typical site-specific monitoring approach was adapted. The following are the main benchmarks of this study.

Landfill Site Characterization

Starting with the available information the gaps were identified. These gaps were covered through field visits, interviews with municipality officials, topographic survey, exploratory drilling (test holes), exploratory sampling, and water level surveys. A conceptual model was built, describing the main lithological and hydrogeological features, as well as the significant land use of the landfill vicinity.

Location, Design and Operation

The 7th Ring Road Landfill started operation in 1986. It is located about 25 km at the south of Kuwait City. It is just upstream of another landfill, which is dedicated to solid waste only. The location was not designed for a landfill, but is an abandoned quarry. This implies the absence of any control and/or safety components, i.e., cap, leachate collection and drainage systems, and lining. It also implies the possibility of serious reduction of the unsaturated zone thickness. The area, as calculated from the topographical maps, for the 7th Ring Road Landfill is 4.8 km^2. The 7th Ring Road site is receiving large volumes of liquid waste, ranging from 600,000 to 750,000 gal/d. The origin of the wastewater is mainly municipal; however, industrial waste is also disposed of at the site. The so-called municipal waste contains liquid wastes from workshops and some factories. Additionally, wastewater from slaughterhouses is finding its way to the landfill. Solid waste dumped at the site, is composed mainly of typical domestic and construction wastes.

Topography and Lithology

The ground surface elevation around the landfill area is complex, varying from 48 to 53 m above mean sea level. The upper aquifer sequence in the 7th Ring Road Landfill is composed of the undifferentiated, fluviatile, clastic sequence of the Kuwait Group. The most dominant deposit in the sequence Is sand. These deposits were affected by the calcretization process. Calcretization was recognized in different ratios and was heterogeneous. Hard calcretic horizons were always encountered within the sequence of this landfill, generally at depths between 9 and 23 m below the ground surface. Clayey horizons were not encountered within the lithology of the 7th Ring Road Landfill.

Groundwater Flow

The depth to the water table from the ground surface varied between 17 and 20 m in the sites drilled at the 7th Ring Road Landfill. Flow direction was determined using piezometric groundwater levels in relation to mean sea level. The leveling survey results with the measured water depth at the test holes and monitoring wells were used to prepare piezometric level contour map (Fig. 1). The flow direction i.e., to the northeast, was similar to the general groundwater flow direction in Kuwait.

Design of a Monitoring System

Using the information gained from the characterization, basic criteria were set for allocating monitoring wells. This stage also included the design of the monitoring wells. Samples collected from test holes were analyzed for a wide range of parameters to determine the significant biological, organic and inorganic constituents of the waste and the groundwater. Combining the information gained from these analyses and the published literature from similar studies, a list was prepared to enable the identification of the landfill impacts on the groundv

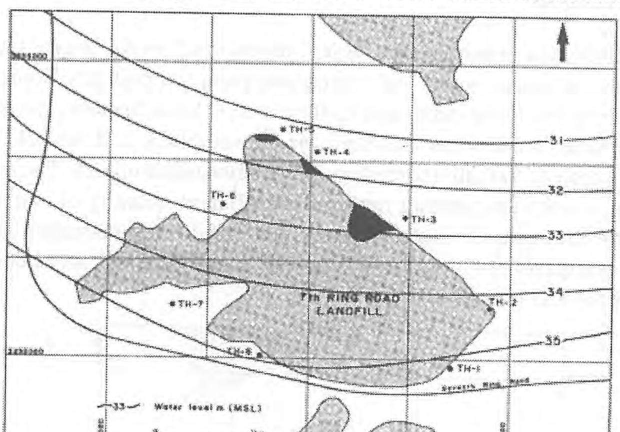

Figure 1: Groundwater levels AMSL at the 7th Ring Road Landfill.

Drilling and Well Construction

Drilling and well construction were accomplished in two phases. The first phase included the exploratory drilling, or test holes, while the second phase included the implementation of the monitoring wells. A total of 8 test holes and 5 monitoring wells were completed during the two phases in the 7th Ring Road Landfill.

Allocation of Test Holes

The criteria for the allocation were as follows:

- Enabled fair description of the lithological sequence at the landfill.
- Tapped the monitoring target zone, which in this case, was the uppermost aquifer.
- Enabled sampling of the groundwater upstream and downstream of the landfill.
- Enabled identification of the local flow direction in the vicinity of the landfill.
- Provided access to the groundwater downstream specific locations in the landfill where the pollution potential was considered highest.

Using these criteria, a total of 8 locations of 25-30 m depth were dedicated for the 7th Ring Road Landfill. These holes were used for conducting geophysical logging, and collecting drill cuttings and representative groundwater samples. Fig.1 shows the location of the drilled test holes.

Allocation of Monitoring Wells

Two upstream wells were allocated to ensure accurate representation of the incoming groundwater. At least one upstream monitoring well was placed at a distance from the landfill in an attempt to determine the extent of the plausible impacts. Fig. 2 shows the distribution of the monitoring wells around the 7th Ring Road Landfill. The final design of the installed monitoring wells is presented in Table 1.

Sampling and Analysis

Samples were collected from the test holes, monitoring wells, waste trucks, waste-infiltration ponds, reference wells, and extraneous potential pollution sources. Samples were analyzed according to the initial and final parameter list at the laboratories. A number, of available, portable instruments was used for the sampling, and the pH, temperature, EC, dissolved oxygen and alkalinity readings were determined on site. The purpose of the readings was to ensure that, purging removed a sufficient quantity of water, to provide valid on-site measurements of unstable parameters, such as temperature and dissolved oxygen; and for comparison with laboratory measurements to check for chemical changes due to holding time and transport.

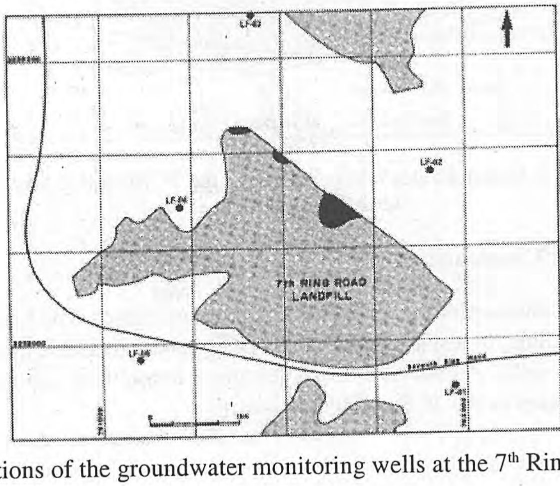

Figure 2: Locations of the groundwater monitoring wells at the 7th Ring Road Landfill.

Table 1: Design of the Installed Monitoring Wells

Well No.	Total Depth (m)	Screen Interval (m from ground surface)
LF-01	32.5	13.0 - 23.8
LF-02	32.5	18.4 - 29.2
LF-03	32.5	18.4 - 29.2
LF-04	32.5	18.4 - 29.2
LF-05	32.5	18.4 - 29.2

Identification of Parameters for Analysis

In this study, the identification of the baseline parameters was carried out in four main steps. First, internationally well recognized pre-prepared lists for baseline analysis for landfill monitoring were reviewed and selected. Second, the selected baseline list was modified according to the characteristics of the incoming waste types. Third, the list was modified according to the project scope and other limitations such as laboratory capabilities. Fourth, after analyzing the first round of sampling parameters with concentrations below the detection limits were discarded. Practically, all of the waste types (Liquid municipal, Solid municipal, Liquid industrial, Demolition and Slaughterhouse) are disposed in the studied landfill, however, at different volumes and proportions. The 7th Ring Road Landfill receives the largest volumes of waste in Kuwait, and currently, is the official location for liquid waste disposal. Combining the information obtained on the waste types, and hence, the plausible contaminant in the groundwater, with the US-EPA list, a shorter list was produced. The modified list contained only justifiable parameters. Parameters were considered justified if they fit within one of the categories of waste disposed of in the landfills and were likely to be produced in Kuwait, i.e., some of the parameters fit under the identified waste categories; however, they come from industries that do not exist in Kuwait.

Finally, after examining the results of the first sampling round, parameters with concentrations well below the drinking water standards were omitted from the list. This list was used for the rest of the analyses in the study. Table 2 presents the extracted list of parameters.

Table 2: Final Extracted List of Parameters

Description	Parameter (Unit)	Description	Parameter (Unit)
Major Constituents (Anions & Cations)	Potassium (mg/l)	Heavy Metals and Trace Elements	Aluminum (mg/l)
	Sodium (mg/l)		Barium (mg/l)
	Calcium (mg/l)		Cadmium (mg/l)
	Magnesium (mg/l)		Copper (mg/l)
	Strontium (mg/l)		Chromium (mg/l)
	Chloride (mg/l)		Lead (mg/l)
	Sulphate (mg/l)		Lithium (mg/l)
	Bicarbonate (mg/l)		Manganese (mg/l)
	Silicate (mg/l)		Nickel (mg/l)
	Total Hardness (mg/l)		Vanadium (mg/l)
	Total Alkalinity (mg/l)		Zinc (mg/l)
	Noncarbonated Hardness (mg/l)		Cyanide (mg/l)
	Total Dissolved Solids (TDS) (mg/l)		Fluoride (mg/l)
Minor Constituents	Total Iron (mg/l)	Micro-organism	
	Boron (mg/l)		
	Nitrate (mg/l)		Total Coliform Bacteria
	Nitrite (mg/l)		Faecal Coliform Bacteria
	H2S (mg/l)		Faecal Streptococci Bacteria
Organics	Total Organic Carbon (TOC) (mg/l)		Clostridium perfringens
	Total Petroleum Hydrocarbons (TPH) (mg/l)		Coliphage Indicator PFU/l
	Polyaromatic Hydrocarbons (PAH) (µg/l)		Biochemical Oxygen Demand (BOD)
Physical	EC		Chemical Oxygen Demand
	pH		

RESULTS AND DISCUSSION

Utilizing statistical analysis, trend analysis, and basic hydrogeological and geochemical principles, the laboratory results were interpreted to distinguish between the impacts of the landfills, the impacts of other pollution sources and the features of the indigenous groundwater quality.

Descriptive Statistics

The TDS concentration ranged between 6,140 and 10,290 (mg/l), with a mean value of 8,985 (mg/l) and a coefficient of variation of 17.6%. The major contributors to this TDS were as follows: sodium with concentrations ranging between 1,608 and 2,505 (mg/l); its mean value and coefficient of variation were 2090 (mg/l) and 16.7%, respectively, calcium ranged between 373 and 814 (mg/l) with a mean value and coefficient of variation of 134 (mg/l) and 22%, respectively, chloride ranged between 1,157 and 3,606 (mg/l) with a mean value and coefficient of variation of 3008 (mg/l) and 31%, respectively, sulfate concentrations ranged between 2,028 and 2,853 (mg/l) with a mean value of 2,484 (mg/l) and a coefficient of variation of 12.5%.

Other inorganic minor constituents were either below detection limits, or within the drinking water limits; except for boron, nitrate, aluminum, barium, lithium and fluoride, which exceeded the drinking water standards. Boron, a typical agricultural pollution indicator, ranged between 2.2 and 4.5 (mg/l), with a mean value of 3.3 (mg/l) and coefficient of variation of 30%. Nitrate, another agricultural as well as domestic pollution indicator, was measured as Nitrogen, and ranged between 0.56 and 39 (mg/l) with a mean concentration of 29.9 (mg/l) and a coefficient of variation of 54%. On the other hand, the trace elements aluminum, barium, lithium and fluoride had concentrations ranging from 1.4 to 3.2, 0.07 to 0.16, 0.15 to 0.24, and 1.06 to 2.7 (mg/l) with mean concentrations of 2.29, 0.12, 0.19 and 2.13 (mg/l), respectively. The corresponding coefficients of variation for the trace elements were 24, 33, 16 and 26%.

Out of the wide range of organic parameters analyzed, only the total petroleum hydrocarbons (TPHs) were above the drinking water limits. In this regard, the only standards obtained for TPH were the Tennessee Cleanup Standards (Davis et al., 1995). The standardizing organizations reviewed, i.e., the WHO (1988), US-EPA (1991), and El-Kuwait El-Youm, (2001) have issued no standards for TPH. The TPH values ranged from below the detection limit to 0.3 (mg/l) with a mean value of 0.21 (mg/l) and a coefficient of variation of 76%.

Microorganism analysis indicated the presence of total and fecal coliform bacteria as well as fecal Streptococci bacteria in the analyzed samples. The total coliform bacteria count ranged between 1,300 and 78,000, with a mean of 20,775 and a significantly high variation coefficient of 148%. Table 3 presents a statistical analysis of the results for the significant parameters.

Interpretation Context

The identification of the upstream and downstream conditions for the facility is often the basis for deeming a groundwater characteristic as an impact or original quality. The flow direction for the studied landfill followed the same regional trend of northeasterly flow, as was shown in Fig. 1. The 7th Ring Road Landfill is currently receiving considerable volumes of liquid waste on a daily basis. The liquid waste is mainly dumped in depressions forming ponds of collective types of liquid waste. These ponds, which represent perfect point sources, are exposed to a variety of chemical, biological and weathering conditions that undoubtedly affect the chemical composition of the seeping liquid. In addition, the liquid waste trucks frequently spray their loads on the ground while driving around in the landfill, creating a possibility of non-point sources covering practically, the whole landfill. The 7th Ring Road Landfill is located just downstream of another landfill located south of the road. However, that other landfill is dedicated to solid waste only, which suggests that little impacts are to be expected from that landfill. Nonetheless, background wells were allocated at the downstream end of the more southern solid waste landfill.

Table 3: Descriptive Statistical Analysis of the Results for Selected Parameters

Parameter (Unit)	Mean	SD	VC	Min	Max	* Standards
Potassium (mg/l)	25.80	9.10	35.2	14.60	43.00	10
Sodium (mg/l)	2090.00	350.00	16.7	1608.00	2505.00	200
Calcium (mg/l)	620.00	134.00	21.6	373.00	814.00	200
Magnesium (mg/l)	242.00	72.00	29.5	137.00	291.00	150
Strontium (mg/l)	9.44	2.40	25.4	6.20	11.70	
Chloride (mg/l)	3008.00	919.00	30.5	1157.00	3606.00	250
Sulphate (mg/l)	2484.00	310.00	12.5	2028.00	2853.00	250)
Bicarbonate (mg/l)	167.00	187.00	112.2	74.00	613.00	
Silicates (mg/l)	9.94	2.30	23.2	7.60	14.70	
Tot. Hardness (mg/l)	2546.00	601.00	23.6	1495.00	3173.00	500
Total Alkalinity (mg/l)	167.00	187.00	112.2	73.00	613.00	100
TDS (mg/l)	8985.00	1586.00	17.6	6140.00	10290.00	1000
Boron (mg/l)	3.30	0.98	29.6	2.20	4.50	0.3
Nitrate (as Nitrogen - mg/l)	29.91	16.08	53.8	0.56	39.00	10
Aluminum (mg/l)	2.29	0.55	24.1	1.40	3.20	0.2
Barium (mg/l)	0.12	0.04	32.9	0.07	0.16	0.7
Lithium (mg/l)	0.19	0.03	15.9	0.15	0.24	
Fluoride (mg/l)	2.13	0.55	25.7	1.06	2.70	1.5
TOC (μg/l)	3.66	3.66	99.9	1.75	12.40	
TPH (μg/l)	0.21	0.16	75.7	0.07	0.50	
Total Coliform Bacteria (CFU/100 ml)	20775.00	30711.00	147.8	1300.00	78000.00	
BOD (mg/l)	4.50	4.74	105.4	0	13.00	
pH	7.43	0.35	4.7	6.86	7.92	

SD = Standard deviation, VC = Coefficient of variation

*Kuwait (EPA) Standards (El-Kuwait El-Youm, 2001), United States Environmental Protection Agency Standards (US-EPA, 1991) and World Health Organization (W.H.O.,1988)

Trends and Implications

The data obtained from analyzing the collected samples were studied carefully in an attempt to characterize the groundwater quality at the landfill site. The evolution trends were analyzed using graphs of concentrations versus distance in the flow direction. In the 7th Ring Road Landfill, all of the analyzed samples exceeded the WHO standards for drinking water with respect to TDS. The TDS concentrations ranged between 6,140 and 10,290 (mg/l). By averaging the TDS concentrations at four intervals, a linear decreasing trend with a high correlation coefficient of about 0.8 in the concentrations of TDS over the flow direction as the groundwater passes beneath the 7th Ring Road Landfill is evident.

The results of analyzing the liquid of the landfill's dumping pond indicated a TDS concentration of 3,000 (mg/l), while the waste trucks' samples contained TDS concentrations of 1,330, 780 and 1,740 (mg/l) for the untreated wastewater, industrial waste and slaughterhouse waste trucks, respectively. The discrepancy between the concentrations in the waste pond and the truck samples may be attributed to the evaporation process among other factors. In all cases, the liquid waste contained TDS concentrations significantly lower than the concentrations in the groundwater in the landfill area.

These results suggest that the decreasing TDS at the 7th Ring Road Landfill may be attributed to the infiltration of the wastewater into the groundwater causing slight dilution of the groundwater's salinity.

Given the low boron concentrations in the entire probable waste, the elevated boron in the groundwater must be attributed to the original characteristics of the incoming groundwater. Both lithium and fluoride showed trends similar to the TDS. Thus, the interpretation suggested for the TDS results is also applicable for those two trace elements. Nitrate showed a pattern similar to boron. Accordingly, the interpretation given for boron is also suggested for the nitrate.

The total coliform bacteria (TCB) at the 7th Ring Road Landfill were generally elevated well above the WHO limits, with an average of $17x10^3$. The incoming groundwater, as sampled from two background monitoring wells LF-01 and LF-05 (Fig. 2), had an average TCB of 220. With a TCB count in the incoming groundwater that was two to three orders of magnitude less than what was detected at the landfill, the elevated TCB was confidently attributed to anthropogenic sources. No accumulation of impact (i.e., increase in the TCB count) was observed along the flow path beneath the landfills, but rather, patches of high and low counts without a clear relation to the flow direction were found. This is common with microorganisms in general, which, unlike dissolved parameters, have a complex relation between spatial distribution in groundwater and flow direction due to their tendency to attach themselves to the soil surface and their rate of growth, which is totally independent of the flow direction. Nonetheless, signs of the flow's influence on the TCB spatial distribution was evident in TCB detection at the furthermost monitoring well (LF-03) about 2 km downstream of the landfill, in contrast to almost zero TCB at closer distances upstream of the landfill. This evident TCB transport with the groundwater flow indicates a potential for a widespread impact if no action is taken.

The TPH concentrations for the samples collected in the vicinity of the 7th Ring Road Landfill were mostly below detection limits, except for four samples (TH-3 to TH-6). As the groundwater samples collected from the background wells (LF-01 and LF-05) showed no TPH, the TPH in the groundwater must be attributed to the landfill activities.

Out of the four samples with detectable TPH, two were above the drinking water limits (TH-3 and TH-4). The spatial spreading of the TPH seems to be very limited - confined to those four locations - without any trends relating to the flow of the groundwater. All four samples with detectable TPH were immediately downstream of the dumping ponds, indicating an obvious correlation. Apparently, the dumping ponds are contaminating the groundwater with TPH, whereas the spreading of the raw waste is not. Two processes may be responsible for this. First, and from a hydraulic point of view, the spreading of the liquid waste over the landfill is not creating enough hydraulic head to drive the large molecules of the TPH down to the groundwater table. The ponds, however, which are essentially depressions, have considerable hydraulic head and minor or probably no unsaturated zone between their bottom and the groundwater table. The other process is the degradation of the TPH by the high bacterial content in the waste under the oxidizing conditions created by spreading, while anaerobic conditions prevail in the ponds. The last process is supported by the negative correlation between the bacterial count (represented by the TCB) and the TPH concentrations, in which the average TCB count in the four samples with detectable TPH was $2x10^3$, while elsewhere the average TCB count was $5x10^4$ (i.e., 250 times greater).

CONCLUSION

In summary, out of the 57 analyzed groundwater constituents, boron, nitrate, aluminum, lithium, fluoride, TPH, TCB and TDS were found to be above the drinking water limits. Analyzing the same 57 parameters in the types of waste disposed of at the landfill, boron, aluminum, iron, TPH, TCB and TDS were above the drinking water limits. Interpreting these results in the context of the prevailing conditions of lithology, upstream sources and operating conditions, only the elevated TPH and TCB in the groundwater could be attributed to the landfill. More specifically, at the 7th Ring Road Landfill, the elevated TPH and TCB can be attributed to the landfill with a considerable level of confidence. The TCB plumes extend well downstream of the landfills (i.e., more than 3 km). The TPH at the 7th Ring Road Landfill seems to be immobile and degradable by bacterial activities.

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ARSENIC POLLUTION IN GROUNDWATER OF BANGLADESH

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ABSTRACT

The scientists in Rajshahi University together with the Research Group for Applied Geology (RGAG) and the Asian Arsenic Network (AAN) of Japan suspect that groundwater of about 60 districts out of 64 are seriously contaminated with arsenic. There are 11 million tube wells in Bangladesh out of which about 5 million are highly contaminated with arsenic. About 75 million people of the affected districts are at risk and the total number of patients suffering from Arsenicosis is about 7000 out of which about 200 persons have already died. To provide safe water further investigations in the whole country is essential. The source of arsenic in the groundwater of Bangladesh is as yet unknown. But it is now widely believed that the high arsenic levels in the groundwater in Bangladesh have a natural geological source which may be due to water abstraction from Quaternary confined and semi-confined alluvial or deltaic aquifers. Groundwater in Bangladesh from sandy alluvial deposits is considered to be arsenic free. It is essential to consider the groundwater occurrences, its distribution and geological and hydro-geological settings for the mitigation of the arsenic problem. In order to understanding the source and mobility of arsenic it is essential to investigate the sampling depth and aquifer provenance. Present study will give some clue about the future action plan for the mitigation of the arsenic problem in Bangladesh. The results of investigation have been discussed. To save the huge population in the area all sorts of international help are essential. If precautionary measures against arsenic contamination are not taken immediately consequences such as the death of many people will be inevitable and massive. Awareness raising about the issue among people should be the first step for precaution.

KEYWORDS: Aquifer Provenance, Arsenic, Contamination, Groundwater, Hydro geological Settings, Geo-chemical occurrences, Bangladesh.

INTRODUCTION

The continental fluvial sediments have known to be common in the present upper continental crust (UC) because sediments have large provenance and are well mixed during the transportation of the particles (Taylor and McLennan, 1985, McLennan et al, 1993. Condie, 1993). The Ganges River is one of the biggest rivers in the Indian sub-continent with its source in the Indian shield and the Himalayan Mountains. Thus fluvial sediments of the Ganges can be regarded as a representative of the Asian continental crust.

Besides the homogeneous compositions of these sediments, they are rich in organic matters derived from plants vegetated under humid and warm climate. This is due to favorable aquifer supply by the Ganges. Tube wells have been utilized for irrigation and drinking water since such systems were established (Yokota et al, 1997). However, arsenic contamination of groundwater is now confirmed in the wide regions of Bangladesh and eastern India, and became one of the serious impacts on health conditions (Rahman, 1997, Yokota et al, 1997). Under these circumstances, core-boring was operated to obtain sediment samples at Samta village in southwestern Bangladesh to examine arsenic concentration of the sediments (RGAG 1997, 1999). Geochemical composition of the core samples were studied by examining the source materials of the Ganges fluvial system. Possible linkage of arsenic to other elements in the sediments have been described. Core samples mainly consist of alternating beds of fine and very fine sands, and mud occurs at a depth of 3-4.9m (upper muddy layer). Sands are composed of well sorted quartz grains accompanying significant mica flakes. Tourmaline and zircon are abundant heavy minerals. The color of the mud is dark gray to black, rich in organic matters. Both sands and mud include carbonate materials which were tested by HCl acid treatment in the field.

History of Arsenic Contamination in the Groundwater of Bangladesh

Field experience and available data shows that both soil and groundwater of a vast area in Bangladesh has been threatened with arsenic contamination affecting the health of millions of people. In Bangladesh high arsenic concentrations are suspected to exist so far in 60 districts out of total 64. About 75 million people of the affected districts are at risk and the total number of patients suffering from Arsenicosis is about 7000.

The physiographic regions (Fig-1) vulnerable to arsenic contamination are the (a) Ganges flood plain (b) the Atrai flood plain (c) the Tidal region (d) the Coastal plain and (e) the Meghna flood plains. The high probability zone, moderate probability zone and low probability zone of arsenic contaminated area together cover almost 60% of the total country area. Arsenic pollution in the groundwater of Bangladesh is possibly the largest mass poisoning case in the world today. The large scale unplanned withdrawal of groundwater may be theoretically the main cause of arsenic contamination in Bangladesh. There are several speculations about the potential source of arsenic. Now it is established that the real source of arsenic in groundwater is the geological formation of the region.

The arsenic affected areas of west Bengal i.e. on a sediment of younger deltaic deposition which extends eastward towards Bangladesh covering most of the 60 districts which lie mainly in the Gangetic, Atrai, Meghna flood plains and the tidal regions of the country (Fig-1). The data and records of geological investigations of the areas adjoining to the western border in West Bengal shows that there are three types of aquifers presently in use for groundwater occurrences and distribution: (i) the shallow aquifer (about 30 m) (ii) the intermediate aquifer (40-80 m) with an aquitard (or an impervious layer) of about 10 to 15 meters and (iii) the deep aquifer (below 100m) under the second aquitard (Fig. 2) (Dave 1996). Arsenic was observed to have been deposited in the first aquitard as an absorbed primary metal on sand grains of biotite and quartz with a few scattered grains of arsenopyrite. Arsenic (leached out of the first aquitard) appears to be confined to the intermediate aquifer (30-45 m depth). Four million tube wells in Bangladesh supply drinking water among which 1.12 million tube wells are contaminated with arsenic.

METHODOLOGIES

Sample Preparation

Core boring has been done in different locations in Samta village under the Sharsa Police station of the Jessore district, Bangladesh for the collection of soil samples for analysis in the laboratory. About 50 g of each sample was dried at 110 °C, and was powdered to a particle size (<63μm) using an automatic agate pestle and mortar for 30 minutes. About 5 g of each sample was ignited at 1000°C for 1 hour to examine the loss on ignition (LOI). Twenty samples were tested in this manner.

Analytical Procedures

Major elements (SiO_2, TiO_2, AI_2O_3, Fe_2O_3, MnO, MgO, CaO, Na_2O, K_2O, and P_2O_5) and trace elements (Th, Sc, Pb, Cu, Zn, Zr and Sr) were analyzed using the RIX-2000 XRF system (Rigaku Denki Co. Ltd.) at Shimane University. Analyses of these elements were made on glass beads prepared with a flux (mixture of lithium tetraborate and lithium metaborate in ratios of 4:1) to a sample ratio of 2: 1 followed by Kimura and Yamada (1996). Other trace elements (As, Ni, V and Cr) were analyzed by the power press method

(Ogasawara, 1987). Average errors for traces comprise less than 10% and the results are acceptable compared with other values.

RESULTS and DISCUSSION

No	Feet	SiO_2	TiO_2	Al_2O_3	Fe_2O_3	MnO	MgO	CaO	Na_2O	K_2O	P_2O_5	Total	LOI
0	10.5	57.67	0.94	19.07	7.79	0.12	3.47	5.14	0.83	3.81	0.14	98.98	8.78
1	12	61.57	0.83	16.49	6.54	0.10	3.05	6.33	0.99	3.32	0.13	99.35	8.31
3	13.8	59.28	0.89	17.63	8.05	0.12	3.08	5.90	0.87	3.44	0.14	99.40	8.72
5	15	59.49	0.96	20.99	8.48	0.05	2.79	1.39	0.57	3.68	0.12	98.52	8.13
7	17	58.55	0.85	18.39	7.66	0.16	3.15	6.50	0.76	3.44	0.15	99.60	9.13
9	19	61.15	0.83	16.68	8.64	0.15	2.95	3.64	1.01	3.44	0.27	98.76	7.16
11	21	60.15	0.91	20.02	7.92	0.9	3.12	2.40	0.83	3.68	0.15	99.26	8.89
20	31	76.58	0.51	10.09	3.31	0.05	1.87	3.76	1.36	2.25	0.12	99.90	3.99
23	35	75.14	0.53	10.85	4.15	0.06	2.00	3.20	1.32	2.59	0.10	99.94	3.62
28	45	79.35	0.39	9.37	2.70	0.05	1.35	3.13	1.39	2.19	0.10	100.00	2.93
37	63	78.97	0.39	9.39	2.79	0.04	1.49	3.20	1.35	20.20	0.09	99.91	3.18
45	79	77.88	0.42	9.93	3.43	0.05	1.72	2.90	1.29	2.28	0.09	99.98	0.55
51	91	87.40	0.14	6.82	0.83	0.01	0.59	1.30	1.22	1.56	0.04	99.93	2.05
53	95	84.46	0.21	7.87	1.52	0.02	0.93	1.83	1.22	1.88	0.05	100.00	2.56
55	99	78.42	0.42	9.53	3.14	0.06	1.55	3.27	1.35	2.13	0.10	99.98	3.22
64	117	81.87	0.32	7.70	1.89	0.04	1.28	3.95	1.26	1.56	0.12	99.98	3.28
70	129	81.13	0.30	9.06	2.15	0.04	1.06	2.40	1.45	2.24	0.07	99.90	2.35
76	141	81.78	0.29	8.70	2.18	0.03	1.20	2.23	1.35	2.14	0.06	99.96	2.99
81	151	82.07	0.32	8.22	2.23	0.05	1.10	2.27	1.29	1.74	0.10	99.40	2.78
86	161	84.77	0.21	7.77	1.41	0.03	0.78	1.30	1.27	1.70	0.06	99.30	2.43
94	177	82.64	0.26	7.30	1.69	0.04	1.19	3.86	1.23	1.49	0.09	99.80	3.79
100	187	81.17	0.46	8.59	2.45	0.05	1.44	4.10	1.33	1.84	0.13	101.55	1.95
105	197	72.23	0.56	12.44	4.93	0.05	2.38	2.86	1.28	3.17	0.06	99.96	3.54

Oxides (Wt. %).

Table 2: Trace Elements of Core Sample Sediments In Samta.

As	Cr	Cu	Ni	Pb	Sc	Sr	Th	V	Zn	Zr	TOC	CC	TN
14	88	52	45	30	18	124	22	138	52	182			
8	76	24	38	20	17	137	20	126	44	255			
7	86	24	42	22	18	130	19	142	46	222			
12	113	30	60	34	17	91	21	201	65	182			
16	88	24	45	23	19	101	20	144	47	197			
9	80	24	36	21	16	104	22	148	42	244			
16	96	27	52	42	16	96	22	178	55	199	1.35	0.29	0.09
5	58	19	20	18	10	116	18	59	26	361	0.18	0.47	0.03
4	67	19	26	20	9	110	14	81	33	262	0.12	0.38	0.04
2	59	19	20	20	10	109	13	49	20	260	0.10	0.26	0.05
4	67	9	18	17	8	111	12	46	22	226	0.09	0.36	0.04
4	67	8	25	19	10	103	9	90	29	150	0.16	0.30	0.04
3	54	5	10	21	6	102	6	21	9	147	0.08	0.29	0.04
4	63	6	16	19	6	98	8	53	17	134	0.08	0.29	0.04
3	63	7	16	20	9	110	15	61	24	262	0.14	0.31	0.04
2	57	6	7	17	9	109	13	31	13	214	0.08	0.48	0.04
3	56	4	14	19	7	105	9	47	19	176	0.10	0.17	0.04
3	60	6	10	18	7	107	9	52	22	180	0.12	0.42	0.04
3	64	7	19	20	9	114	18	67	16	254	0.09	0.17	0.04
3	58	6	8	19	7	106	10	48	13	201	0.10	0.22	0.04
3	53	4	7	17	8	119	14	29	13	232	0.08	0.65	0.04
4	65	7	12	17	8	117	20	42	18	436			
4	68	11	24	19	10	105	8	103	41	151			

Trace elements (ppm), Wt. %.

Results are given in Tables 1 and 2.

Table 1: Major Elements of Core Sample Sediments in Samta.

Analysis of Organic Carbon, Nitrogen and Sulfur

Contents of total organic C (TOC), total N (TN) and total S(TS) were measured after 1M-HCI treatment of 15 mg samples by combustion and gas chromatography using a Fisons (Carlo Erba) EA 1108 CHNS Elements Analyzer at Shimane University (Sampie et al, 1997). The errors inherent to this analysis are within 3% for TOC and TN and 4% for TS. The results are given in Table 2. Total sulfur was less than 0.01 wt % in all examined samples

Broad variation with stratigraphy

Sands show high concentration of SiO_2 (mostly over 80 wt %) which is due to enrichment of quartz grains in sand samples. Among other major elements, TiO_2, Fe_2O_3, MgO, K_2O and P_2O_5 show gradual increase in the upsection probably related to the sorting effect (Taylor and McLennan, 1985; Nesbitt et al., 1997) or absorption on clays. Mud contains rich organic matter revealed by higher LOI over 8 wt %. It has TOC (total organic carbon) values over 1.0 wt% (Table 2). Sands and mud have carbonate carbon (CC) values of about 0.3 wt%. The total sulfur concentration, however, has not been detected for these samples. The core sediments generally homogeneous compositions in terms of geochemical indices representing source rocks, SIO_2/Al_2O_3, Al_2O_3/TiO_2, and Th/Sc ratios are examined with stratigraphy .These ratios

INAA									
No	0	3	7	20	37	53	70	86	105
ppm									
La	42.7	41.0.	40.4	39.5	26.1	16.9	19.8	23.0	17.9
Ce	85.8	81.2	83.0	81.2	55.5	35.9	44.2	48.7	37.6
Sm	7.7	7.3	6.9	6.9	4.4	3.0	3.5	3.9	3.3
Eu	1.2	1.2	1.3	1.0	1.7	0.6	0.6	0.6	0.6
Gd	10.8	10.0	9.3	10.1	7.5	6.8	7.4	6.4	3.6
Tb	9.0	1.0	1.1	9.0	5.0	4.0	5.0	5.0	6.0
Yb	3.0	2.3	2.9	2.6	1.8	1.3	1.7	2.0	1.4
Lu	0.53	0.48	0.54	0.47	0.3	0.2	0.3	0.3	0.2
Hf	4.2	4.9	4.9	9.1	5.8	3.1	4.2	4.4	3.4
Ta	1.6	1.4	1.3	1.1	0.7	0.6	0.7	0.7	1.3
Cs	14.7	11.7	11.5	6.2	6.5	5.5	5.9	4.3	10.8
Ba	55.8	44.2	49.4	29.2	21.6	28.3	28.4	22.3	41.6
Sb	1.3	1.4	1.3	0.7	0.6	0.3	0.5	0.4	0.8

show small variation in the column excluding the middle portion of the horizon 27.7 m and 28.9 m shown in SiO_2/Al_2O_3, Al_2O_3/TiO_2 ratios, which may be due to influx of coarse sands.

Table 3: Rare Earth Elements and Other Trace Elements Found in Core Samples in Samta.

As, Pb, Zn and Sb

Arsenic occurs in mud samples of the upsection (4.6 to 6.4m, over 20 mg/l excluding one sample) and As/Al_2O_3 and As/Pb show significant projection in this horizon. In the natural system, As behaves similar to Pb thus As/ Pb ratios may indicate As enrichment related to other reactions. As in concentration, however, it is still lower than that of average soil (As = 30 mg/l) thus higher concentrations in groundwater should be considered as an active solution of As from the sediments. Zn shows no significant concentration in the section. Sb concentrations range from 0.4 to 1.4 mg/l (Table 3) showing lower concentration compared with average soil (1.0 mg/l).

Cr, Ni, V and Cu

In general, concentrations of these compatible elements are affected by diagenesis especially related to sulfur reducing reaction (Calvert and Pedersen, 1993, Jones and Manning, 1994. The results show no anomalous enrichment in samples for Ni and Cr compared to Al_2O_3 (Table 1) Vanadium, however shows enrichment relative to Al_2O_3 in the samples ,V was probably effectively fixed in association with rich organic compounds (e.g. porphyrin compounds), during progressive burial (Arthur and Sageman, 1994, Calvert and Pedersen, 1993). Sediments show positive correlation between As and V (correlation coefficient, R^2=0.75), but Cr, Ni and Cu do not show correlation with As. This geochemical feature suggests that As is not related to sulfide minerals but to organic compounds, because As may substitute for Fe.

A Possible Linkage of Concentration to Other Elements

Mud with rich organic matters generally concentrate metal elements and are utilized as mineral resources (e.g. Arthur and Sageman, 1994; Calvert and Pedersen, 1993). The reaction of such elements is complex and proceeds through several stages. The present limited data could not reveal reaction of As concentration in the sediment samples, nor As contamination of groundwater. The present study, however, demonstrates the relationship of As concentration with other possible guide elements having similar behavior. Arsenic generally behaves very much like divalent Fe and they correlate well in concentration. The samples show positive correlation between Fe_2O_3 and As (R^2=0.71) suggesting that Fe may be a guide element for As concentration in the groundwater of Bangladesh.

CONCLUSION AND RECOMMENDATIONS

On the basis of the analytical results obtained and considering the magnitude of the arsenic poisoning problem; priority should be given to the following points for the mitigation of the arsenic problem in Bangladesh and give people access to arsenic free water:

a) Arsenic affected areas on the Ganges Delta are so vast with a huge population. To save this huge population all sorts of help from experts, national and international organizations and NGO's all over the world are expected to come forward to carry out field investigations and to take proper measures for the mitigation of arsenic calamity in Bangladesh.

b) If precautionary measures against arsenic contamination are not taken immediately, consequences like death of many people will be inevitable and massive. Awareness raising about the issue among the people should be the first step for precaution.

c) Without quality assured data correct steps can not be taken for the mitigation of the arsenic problem of Bangladesh. Establishment of quality assurance in the analysis of soil and water samples related to the arsenic problem is very important.

d) It is essential to find out the exact and possible sources of arsenic in the groundwater in the arsenic affected areas of the whole country.

e) It is also very important to know the mechanism of arsenic contamination in groundwater.

f) Not to jump from one local explanation to a nation wide or basin wise explanation because there is no reason to think that these answers will be applicable in all cases.

g) To identify arsenic patients and initiate their treatment. Skin testing is the most reliable to detect chronic arsenic poisoning.

h) Surface water such as ponds, lakes and rivers can be used as a source of drinking water after boiling it. Rain water can be another safe source.

i) Immediately identify the high risk zone of arsenic. The people of the area should stop drinking tube well water.

j) Sophisticated laboratory facilities should be developed to detect arsenic concentration in tube well water as well as that in the human body.

k) Efficient watershed management is necessary.

l) Arrange to supply arsenic free drinking water because safe water is the best medicine for the people of the arsenic poisoning areas.

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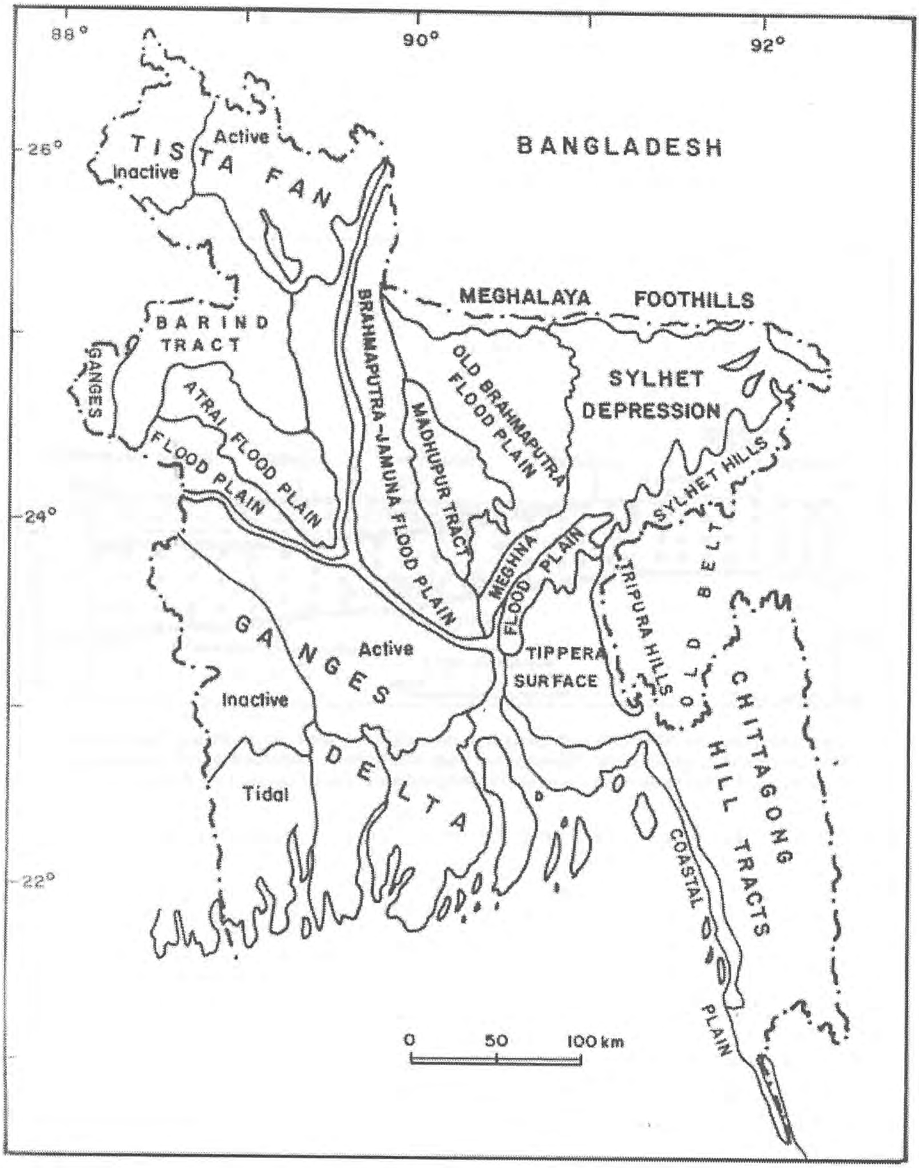

Fig-1. Generalized Physiographic Map of Bangladesh.

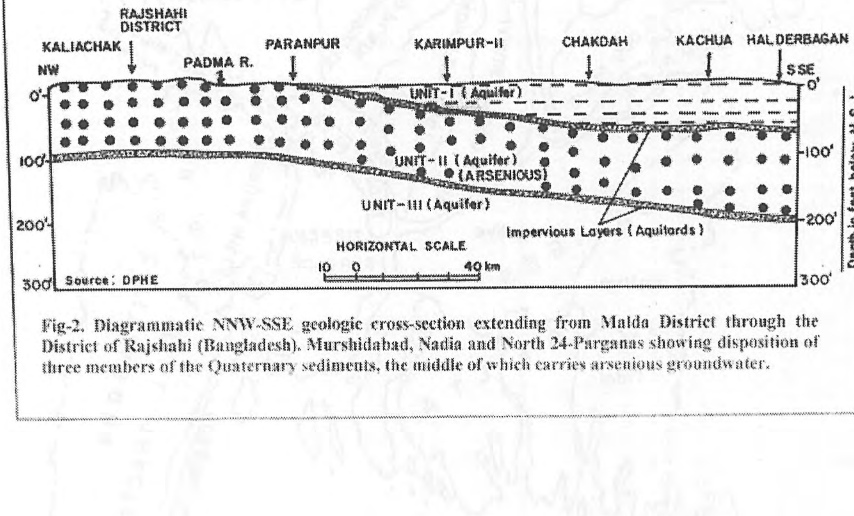

Fig-2. Diagrammatic NNW-SSE geologic cross-section extending from Malda District through the District of Rajshahi (Bangladesh). Murshidabad, Nadia and North 24-Parganas showing disposition of three members of the Quaternary sediments, the middle of which carries arsenious groundwater.

Environment impact of desalination plants on the environment

Khalid A. Mohamed

ENVIRONMENTAL IMPACT OF DESALINATION PLANTS ON THE ENVIRONMENT

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ABSTRACT

Desalination of seawater is one of the main alternatives for the substitution of water shortage in the Arabian Gulf countries and other countries in the Middle East like Egypt. Although desalinating seawater is costly, it is still an important option for compensating for the water shortage. Most of Gulf countries built power and desalination plants for water and power production. We should be aware of the fact that the effluent discharges from the plant back into the sea may have a negative impact on environment. They adversely affect the marine life and the ecology in the plant vicinity. The paper will present the negative impact of desalination plants and how it can be minimized to keep the marine life and the ecological environment in a good condition.

Keywords: desalination, ecology, water quality, numerical models

1. Introduction

Desalination plants are being widely used in the Gulf countries as a main source of providing fresh water to overcome the water shortage. Some other Middle East countries have already started building desalination plants, such as Egypt which built a large plant on the Mediterranean coast. Most of the desalination plants are combined with power plants for power production. There are many desalination plants in the Gulf region and there are plans to build more. Abu Dhabi Emirate, as an example, has 5 plants producing 550 MIGD water and 7,164 MW power. There is a plan to extend the capacity of the existing plants in parallel with building new ones to satisfy the rapid urban and industrial developments in the Emirate. Figure 1 shows a general layout of the desalination plants in the Abu Dhabi Emirate. The desalination plants in the Gulf region are built either on the coast of the Arabian Gulf or in the lagoons. They abstract the seawater through their intakes and discharge the effluents back to the sea through the outfalls. The effluent discharges have a high concentration of seawater, temperature, and salinity and other substances which may adversely affect the water quality in the plant vicinity and species living in the area. The change in the water quality will impact the biota and the possible effects of the plant on water quality will be presented in Section 2. Section 3 will give a general overview on the rich ecological environment in the Arabian Gulf waters which may be affected by the desalination plants. The techniques used to minimize the effect of the plants on the environment will be discussed in Section 4. A case study on one of the desalination plants in the Abu Dhabi Emirate will be presented in Section 5 and the Conclusions will be in Section 6.

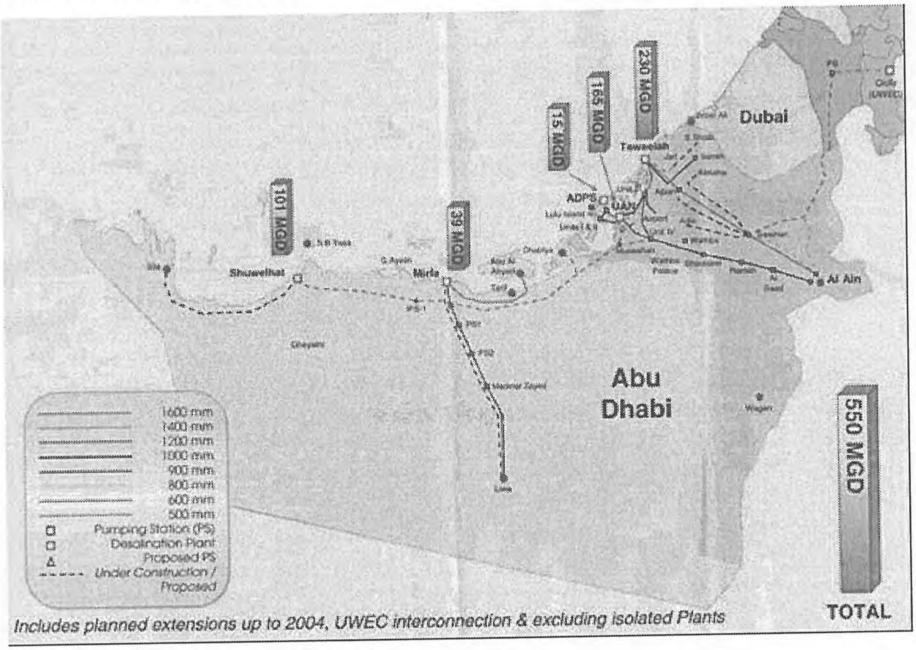

Figure 1: General layout of the desalination plants in Abu Dhabi

2. Possible Effects of the Effluents of Power and Desalination Plants

There are effects on the marine environment arising from the operation of power and desalination plants from the routine discharge of effluents because these water effluents typically cause a localized increase in sea water temperatures, which can directly affect the organisms in the discharge area. Increased temperature can affect water quality processes and result in lower dissolved oxygen concentrations. Furthermore, chlorination of the cooling water can introduce toxic substances into the water. Additionally, desalination plants can increase the salinity in the receiving water. The substances of focus for water quality standards and of concern for the ecological assessment can be summarized as follows:

- Salinity: High concentration of salt is discharged to the sea through the outfall of desalination plants, which leads to the increased level of salinity of the ambient seawater. Generally, the ambient seawater salinity in the Gulf is about 45 ppt and the desalination plant increases this level in its vicinity by about 4 to 5 ppt on average above the ambient condition.

- Temperature: If the desalination plant is combined with a power plant, as is the case in most plants in the Gulf area, the water temperature of the effluents of the power plants will be high and will increase the seawater temperature of the ambient water in the plant vicinity. In summer, the ambient seawater temperature is about 35 °C on average and the power and desalination plants cause an increase in the temperature level in its vicinity by about 7 to 8 °C above the ambient condition.

- Oxygen: Dissolved oxygen in water in the plant vicinity is affected by the effluent discharges from the plant. The concentration and saturation of oxygen will decrease due to the higher temperature and salinity of the effluents. The concentration of dissolved oxygen depends on the seawater temperature in the plant vicinity, concentration of oxygen in the discharge and the mixing of the discharge with the ambient water.

- Chlorine Concentration: chlorine concentration in the effluents of the plants depends on the dosing rates used in chlorination of the seawater. Increasing the concentration of residual chlorine may affect the water quality of the ambient water and, hence, the ecological system. The concentration of Chlorine in the discharge depends on the number of dosings per day and the concentration of the Chlorine used in each dosing.

- Un-ionized ammonia: Ammonia is one of the substances of concern as un-ionized ammonia (NH_3) is very toxic to aquatic species. In the environment, both ionized and un-ionized species occur. The ratio of the two species is a function of the pH. If pH is high then the concentration of the un-ionized ammonia is high and may affect the marine life.

The concentrations and levels of these substances in the plant vicinities depend on the size of the plant and the ambient seawater conditions. Generally the concentrations and levels of these substances should be within the water quality standards to avoid the negative impact on the environment.

3. Description of Main Ecosystem in the Arabian Gulf

The main hydrodynamic force in the Arabian Gulf is the tide. Large areas of tidal flats are in the Arabian Gulf. These areas are flooded during high tide and dried during low tide. Tidal flats, subtidal areas and mudflats are a good environment for many habitat and species. The following ecotopes are the main ecosystem in the Arabian Gulf region:

Mangrove swamps: They grow extensively in the tidal flats. The combination of the mangrove swamp and the large neighboring mudflat is an important ecosystem for many birds.

Seagrass meadows: Dense and spare seagrass were observed in large areas in the Arabian Gulf waters. They differ in types and density from one location to another. Seagrass plays an important role in the Gulf marine environment. About 9% of the Gulf's faunal texa are endemic to seagrass meadows (48 out of 530 recorded species). Of these, about half are molluses. Seagrass also play a major role as a sole food for endangered species such as dugong and the main food source for all marine turtle species, but in particular the green turtle. Among the commercial species, the pearl oyster often settles in or near seagrass beds and of course there are many important fisheries species, such as shrimps. Seagrass helps in stabilization of mobile sands and therefore shorelines.

Corals: Coral areas in the Arabian Gulf are primarily controlled by the availability of suitable substratum. They are extensively found in the Gulf region with marvelous colors. Coral reefs are the most diverse environment of the marine realm. They are not only important biodiversity batteries, but also important for fisheries. The mortality of a part of the coral reef system may have somewhat decreased the number of fish.

4. Techniques to Minimize the Negative Impact of Desalination Plants on the Environment

The role of the water and power research centers in the Arab countries; especially in the Gulf area is very important in minimizing the impact of the desalination plant on the environment. It is a big challenge to utilize of the desalination techniques with a minimum adverse impact on water quality and environment. A comprehensive environmental impact assessment study should be carried out before building a new desalination plant or extending the capacity of an existing one to limit the negative impact of the plant.

The Water and Power Research Center of the Abu Dhabi Water and Electricity Authority has set up study procedures to be followed on the study of the environmental feasibility of building or extending the capacity of the desalination plants. These procedures are as follows:

a) Baseline data collection

Field measurements should consist of hydrodynamic, water quality and biological measurements as follows:

Hydrodynamic field measurements: Measurements should be carried out in the plant vicinity and should include water levels, current flow velocities and directions, and flow discharges.

The hydrodynamic measurements will be used in understanding the flow pattern in the plant vicinity and in the calibration of the hydrodynamic model of the area.

Water quality measurements: water quality measurements should be carried out to evaluate the concentrations of the substances of importance to the water quality and aquatic species. The substances include residual chlorine, dissolved oxygen, ambient seawater temperature and salinity, pH and ammonia. The measurements will be used to evaluate the water quality with regard to the water quality standards and used for the water quality model calibration.

Biological survey: A biological survey should be carried out in the plant vicinity to evaluate the ecosystem in the area. A detailed sampling grid should be constructed in the plant vicinity and surveyed by the ecologist. Data should provide a detailed description of local habitats and species. Photos on the ecosystem should be taken on the grid by divers with underwater cameras. The value of the ecosystem in the study area can be evaluated by the ecologist based on the findings of the survey.

b) Develop the numerical hydrodynamic flow model

The flow velocities and flow pattern is the main transport and dispersion mechanism of the effluents from the outfall. A numerical flow model simulates the flow pattern in the plant vicinity and the configuration of the intake and outfall of the plant should be developed and calibrated with the field measurements. In the Arabian Gulf, the main driving hydrodynamic force is the tide. The model will reproduce the flow pattern which will be used as an input for the water quality model. The flow pattern from the hydrodynamic model can be used to predict the morphological changes of the shoreline due to the construction of the intakes and outfall of the plant.

c) Develop the numerical water quality model

A numerical water quality model should be developed. The goal of water quality modeling is to simulate the water quality of the waters around the plant, as influenced by the discharges from the power and desalination plant. The diffusion and dispersion of the modeled substances discharged from the outfall to the seawater will be simulated. The flow pattern from the hydrodynamic model will be used as an input for the water quality model as it is the main transport mechanism of the substances. The model calibration will be carried out by with the water quality measurements.

d) Evaluation of the water quality results

Water quality model results should be evaluated against the water quality standards. If the modeled substances violates the water quality standards and may affect the marine life, the configuration of the intake and outfall of the plant should be modified (i.e. using a pipeline instead of an open channel) in the hydrodynamic model and repeat the water quality model computations until the modeled substances meet the requirements.

e) Habitat evaluation procedures

The effect of the water quality change due to the outfall discharge should be evaluated against the nature of the habitat in the plant vicinity. The output concentrations of the modeled substances obtained from the water quality modeling should be compared with the species thresholds. If the study shows that the plant discharge will adversely affect the environment, measures should be taken to minimize this effect like changing the proposed configurations of the intake and outfall structures to redistribute the substances in the effluent discharge in a way to reduce their concentrations. Hydraulic structures can be designed and used to guide the flow pattern and flow velocity to control the diffusion and dispersion of effluents. This can be a solution to minimize the negative impact of the effluents on the environment.

5. Case Study

a) Plant description

The Taweelah Power and Desalination Plant is one of the main plants in the Abu Dhabi Emirate. The plant is located on the coast of the Arabian Gulf, as it can be seen in Fig. 2. The plant produces 1000 MW and 100 MIGD of power and water, respectively. It is proposed to extend the plant capacity by 66.5 MIGD.

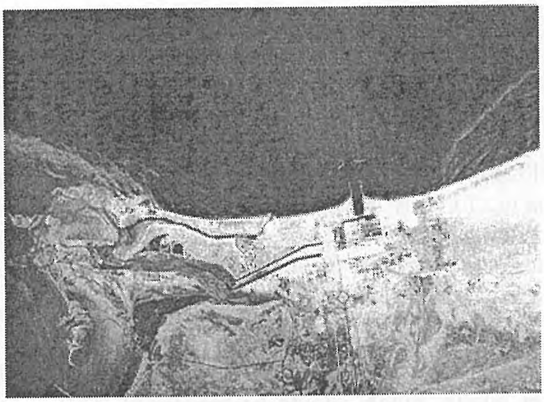

Figure 2: General layout of Taweelah plant

Water quality modeling and ecological study was carried out to evaluate the impact of the proposed capacity extension on the water quality and the marine life in the plant vicinity. An ecological reach and unique sea area with coral reefs, dense seagrass, mangroves and aquatic life is located at Ras Ghanada, which is about 1 km east of the plant intake. A biotopes survey was carried out in the vicinity of the plant to collect information on the species and habitat living in this area. Figure 3 shows the biotope map of the habitat in the vicinity of the plant.

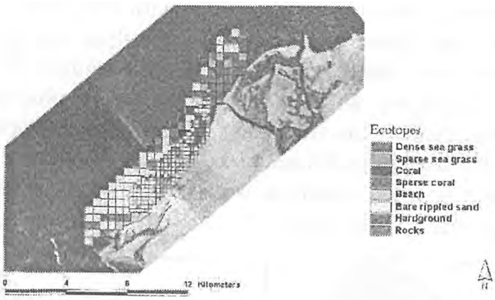

Figure 3: Ecotopes at the marine environment near Taweelah Plant

Data was digitized and mapped into a GIS. Ecological information was gathered on their specific sensitivities and threshold values for abiotic parameters, such as temperature, salinity and oxygen. A water quality model was developed describing the transport, diffusion and dispersion of a number of typical pollutants associated with the power and desalination plant. The results from the water quality model serve as an input for the study to assess the impact on the environment and quality of local habitats

b)Set up of the water quality model

The water quality model was set up after the feasibility of the hydraulic extension was confirmed. The water quality calculations use the same model grid as used in the hydrodynamic model. The basis for the water quality model is the hydrodynamic model. The water quality simulations use the computed water levels and velocities from the hydrodynamic computations as an input. A number of the important substances for the marine life were modeled. These substances are the fraction of water from the discharge, age of water from the outlet, dissolved oxygen in water and chlorine concentration. Modeling considers the effects of the loads of the plant discharges as given in Table 1.

Table 1: Concentration of substances in the outfall discharge

	Discharge conc.	**Peak conc.**	**Remarks**
Oxygen	2.68 (mg/l)	-	67% saturation
Chlorine low decay	0.15 (mg/l)	-	
Chlorine high decay	0.15 (mg/l)	0.6 (mg/l)	Peak 1 hour at High tide
Acid wash	-	5.7×10^{-4} (m^3/m^3)	10 minutes at high tide
Fraction water	1.0 (m^3/m^3)		

Three situations are modeled as follows:

1 - The present situation (reference);

2 - T03: proposed extension is a hybrid of RO + MSF plant. The intake of the RO is an offshore pipeline at about 3.8 km offshore and it uses the existing outfall. The intake of the MSF is onshore at about 1200m to north-east of the existing intake and the outfall of the plant is about 2 km from its intake.

3 - T32: same as T03 but the intake of the RO is at 2000 m offshore and the outfall of the MSF is moved 2000 m further north east.

Figures 4 to 7 show the spreading and the distribution of the main substances influencing the eco-system. In the figures, the water quality results are presented as contour plots comparing the extension scenario with the present design capacity for the two wind conditions. These substances are dissolved oxygen, residual chlorine, seawater temperature, and salinity, respectively. The figures show that the residual chlorine and seawater salinity will not be affected by the proposed extension in the ecologically rich sea area of Ras Ghanada. The seawater temperature and dissolved oxygen in the sea area nearby Ras Ghanada will be affected by the proposed extension. The figures show that no wind condition is worse than a daily wind cycle because of the longer residence times of the water.

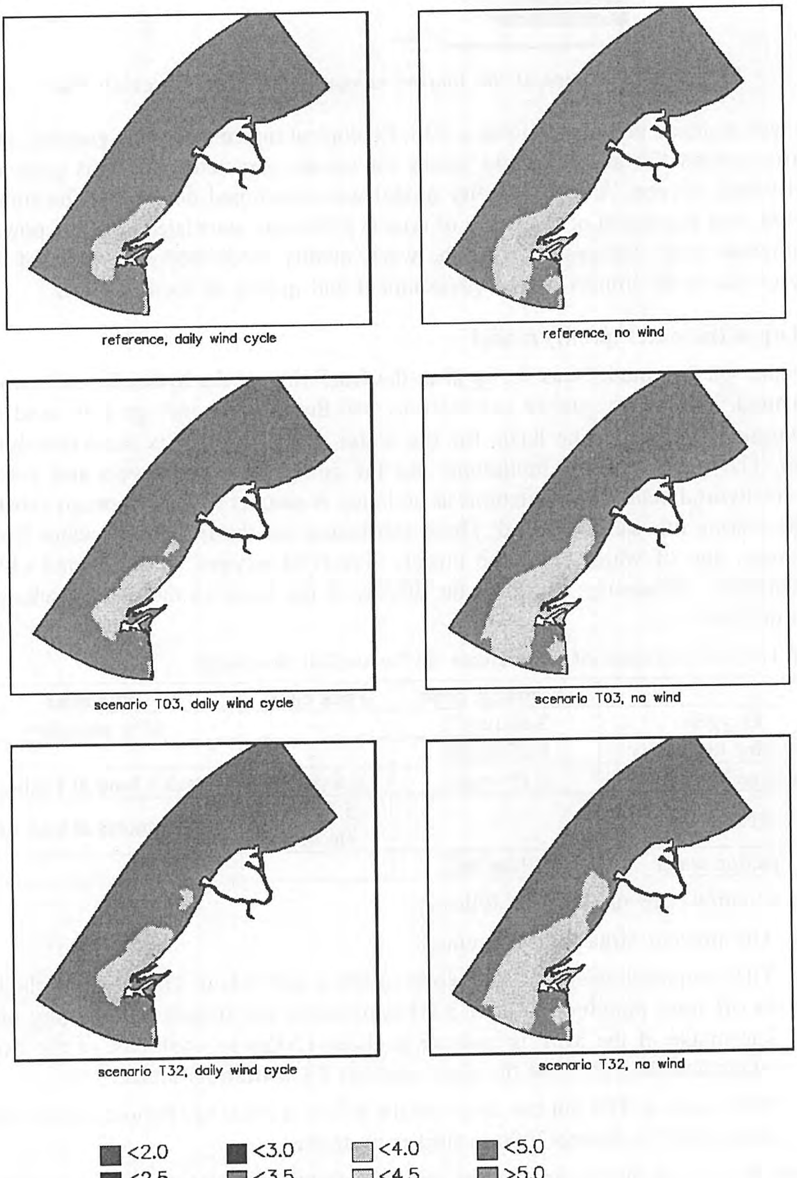

Figure 4: Dissolved oxygen distribution (mg/l)

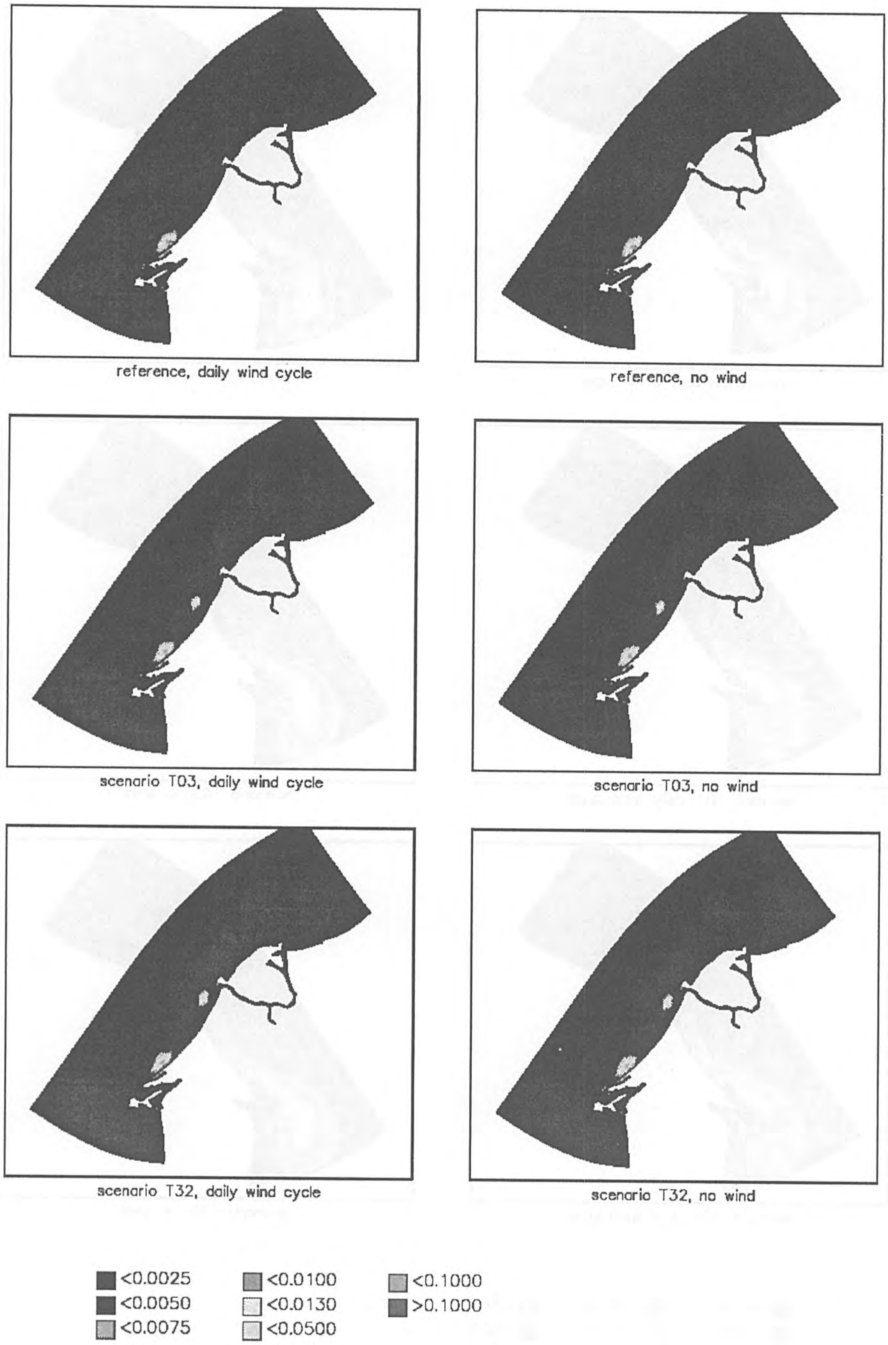

Figure: 5: Residual chlorine distribution (ug/l)

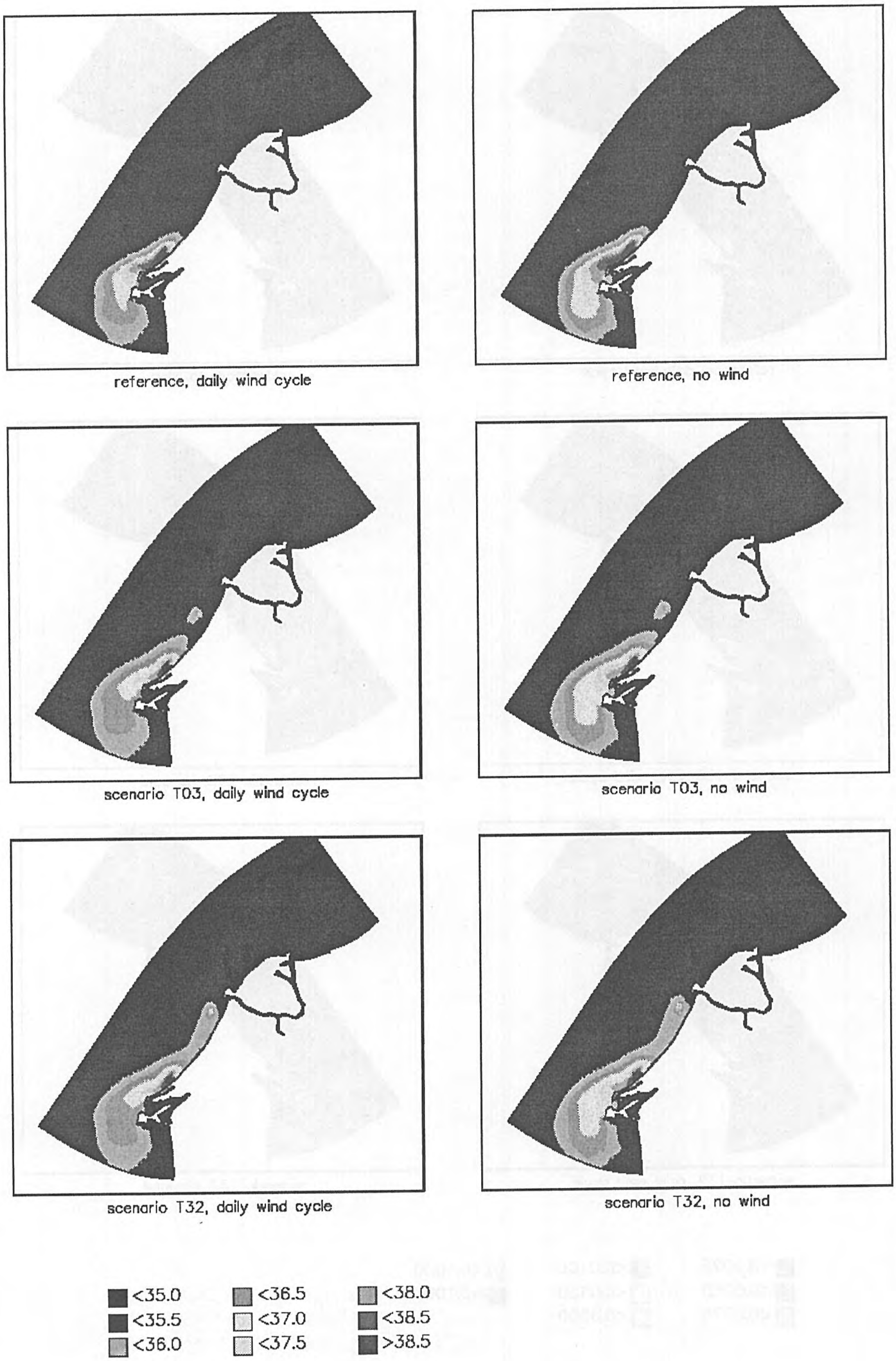

Figure 6: Temperature distribution (c)

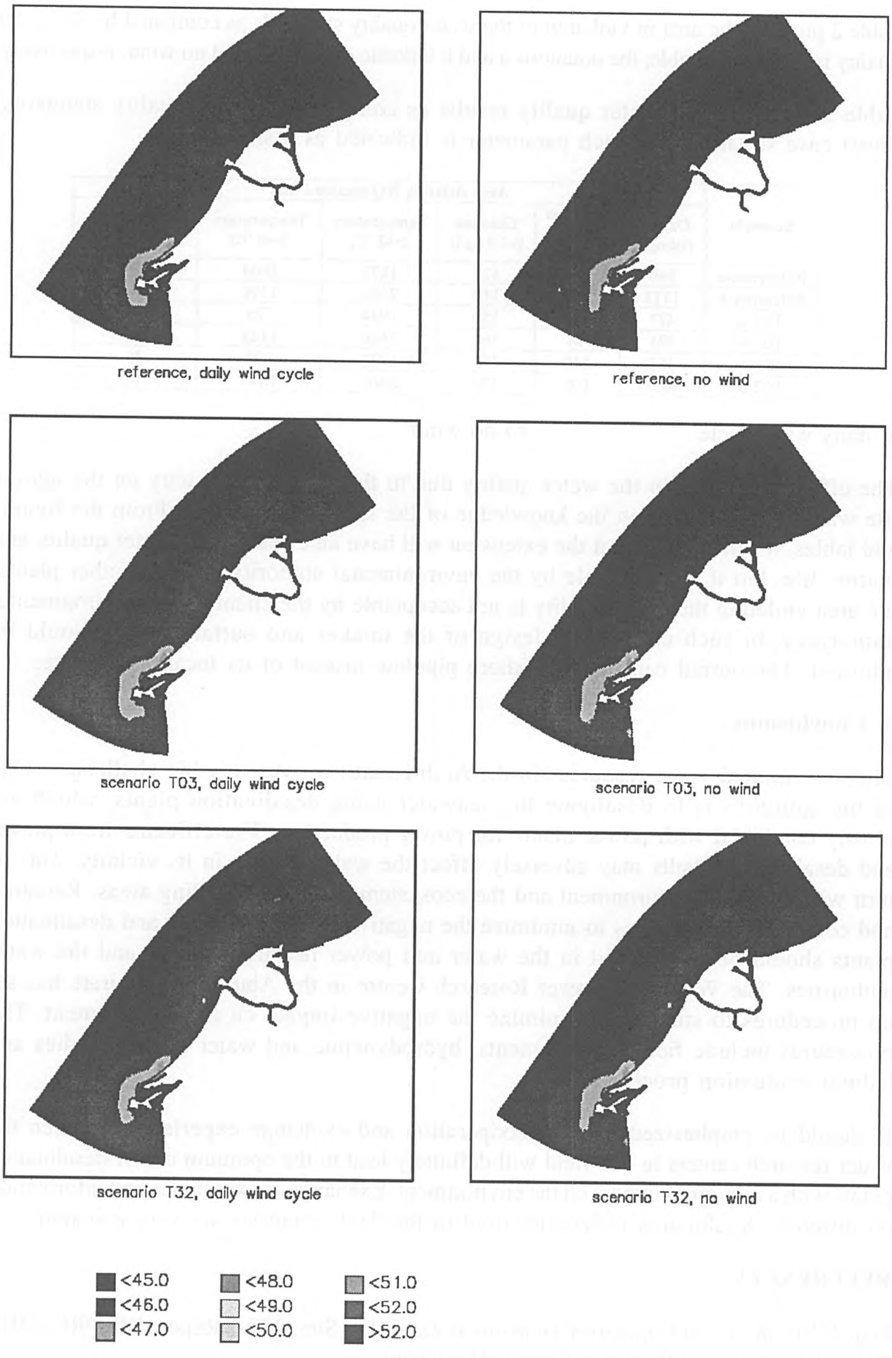

Figure 7: Salinity distribution (ppt)

Table 2 presents the area in violation of the water quality standards as computed by the water quality model. In the table, the notations a and b indicate daily wind and no wind, respectively.

Table 2: Summary of water quality results as compared to water quality standards. Worst case situations for each parameter is indicated as a shaded area

Scenario	Area violating WQ standard (ha)					
	Oxygen (<4mg/l)	Chlorine (>13 ug/l)	Chlorine (>7.5 ug/l)	Temperature (>+2 °C)	Temperature (>+3 °C)	Temperature (>+5 °C)
Reference-a	840	72	114	1871	1061	199
Reference-b	1313	74	119	2305	1396	280
T03-a	471	98	156	1944	726	90
T03-b	991	96	166	2456	1123	113
T32-a	452	110	169	1932	692	76
T32-b	1018	108	172	2560	1045	109

a) daily wind cycle b) no wind

The effect of change in the water quality due to the proposed capacity on the marine life was obtained based on the knowledge of the species in the area. From the figures and tables, it can be seen that the extension will have an effect on the water quality and marine life, but it is acceptable by the environmental authority. In some other plants, the area violating the water quality is not acceptable by the client or the environmental authorities. In such cases, a re-design of the intakes and outfall layout should be adjusted. The outfall can be an offshore pipeline instead of its location onshore.

6. Conclusions

The very limited water resources in the Arab countries pose very big challenges. One of the solutions is to desalinate the seawater using desalination plants, which are mostly combined with power plants for power production. The effluents from power and desalination plants may adversely affect the water quality in its vicinity. This in turn will affect the environment and the ecosystem in the surrounding areas. Research and comprehensive studies to minimize the negative impact of power and desalination plants should be carried out in the water and power research centers and the water authorities. The Water and Power Research Centre in the Abu Dhabi Emirate has set up procedures to study and minimize the negative impact on the environment. The procedures include field measurements, hydrodynamic and water quality studies and habitat evaluation procedures.

It should be emphasized that the cooperation and exchange experience between the water research centers in this field will definitely lead to the optimum use of desalination plants with a minimum impact on the environment. Exchange of experience and information on different desalination techniques used in the Arab countries are very essential.

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Water – related Ecological hazards and their mitigation in Qatar

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WATER—RELATED ECOLOGICAL HAZARDS AND THEIR MITIGATION IN QATAR

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ABSTRACT

The State of Qatar is a small peninsula in the Arabian Gulf covering an area of 11493 km^2. The population was 743, 000 in 2004, living mostly in the Greater Doha. The climate is a typical arid desert type characterized by scanty rainfall with an annual mean of 84 mm. Agricultural expansion; urbanization, industrialization and socio-economic development are putting tremendous pressures on Qatar's limited water resources. The concentration of the population in Doha has resulted in increasing activities accompanied by the increase of water consumption and solidwaste generation. The objective of this paper is to investigate the ecological hazards caused by agricultural activities and the disposal of solid and liquid wastes in addition to proposing mitigation measures. Based on the available data and results of previous studies used in the assessment, the primary ecological hazards identified were groundwater pollution and desertification. These hazards were caused by successive years of drought, uncontrolled excessive groundwater extraction, heavy application of chemical fertilizers, and improper disposal of solid and liquid wastes, increasing water salinity, moving sand dunes, soil degradation and abandoned farms. Both constructional and non-constructional measures could be adopted to reduce the ecological risks. Constructional measures include rehabilitation of farms such as construction of protective embankments and intercept drains, construction of recharge wells, implementation of an artificial recharge pilot project, lining of landfills and the establishment of an agricultural project in the southwestern part of Qatar. Non-constructional measures include the assessment of groundwater resources, integrated water management, groundwater protection, combating desertification, enforcing groundwater laws, promoting public awareness and developing education and training programs.

Keywords: groundwater pollution, desertification, constructional measures, non-constructional measures

1. INTRODUCTION

Qatar is a small peninsula in the Arabian Gulf covering an area of 11493 km^2 including a number of small offshore islands. The population was 743, 000 in 2004, living mostly in Greater Doha, the capital of the country. The climate is a typical arid desert type characterized by scanty rainfall with an annual mean of 84 mm, high temperature (> 40oC) during the summer, high evaporation with an annual average of 2200 mm, very strong winds and high relative humidity. The arable soil area is estimated to be 33, 677 ha, but only 6,330 ha are cultivated, mainly with green forages and date palms. There is no surface water and the main renewable water resource available for agriculture is groundwater. The total water consumption in 2002 was 428.605 million cubic meters (MCM) of which 276.930 MCM was used for crop production (242.633 MCM groundwater and 34.297 MCM treated sewage effluent). The desalinated seawater used for domestic and industrial purposes was 151.340 MCM, in addition to 0.335 MCM of relatively good quality groundwater used for drinking in rural regions (DAWR/MMAA, 2003).

Increasing population, socio-economic development, high standards of living, urbanization, industrialization, agricultural expansion are putting tremendous pressures on Qatar's limited water resources. Groundwater pollution and creeping of desertification are increasing continuously in Qatar. These are manifested in the depletion of the groundwater reservoir, deterioration of water quality, scanty of natural vegetation and abandonment of farms. The concentration of the population in the Greater Doha has resulted in increasing activities accompanied by the increase of water consumption and solidwaste generation. Improper disposal of solidwaste in unlined municipal landfills lead to groundwater pollution and serious health hazards. The occasional high intensity rainfall events attack unprotected farms located in depressions and cause damage to buildings, soils and crops. The present situation seems to be heading towards several ecological hazards. This paper is sought to identify these hazards and propose risk reduction measures for groundwater pollution and desertification.

2. IMPACT OF AGRICULTURE ON THE ECOLOGY

There are several practical problems associated with using saline water in the Qatari farms. The most serious of these problems are groundwater pollution, degradation of soils and consequent abandonment of farms.

2.1 Groundwater Pollution

Groundwater pollution in Qatar is caused by several factors. The most important factor is over pumping from wells. Other factors include seawater intrusion, irrigation returns, heavy application of chemicals, high evaporation rates, and liquid effluents from septic tanks, defective wells construction and failure to seal abandoned wells.

2.1.1 Excessive abstraction

The main reason for groundwater pollution is the uncontrolled excessive pumping from wells. The present extraction is estimated to be about four times the average recharge from rainfall. The extraction of groundwater increased from 51.2 MCM in 1975/76 to 218.4 MCM in 2003/04, equivalent to a 327% increase (Fig. 1). The number of wells in use for irrigation increased from 660 to 2981 during the same period. The over extraction from wells in the same farm or in neighboring farms leads to lowering the water table and consequent up flow of brackish water from the underlying aquifer and thus, increases the water salinity. The increase in total salinity with time is clearly shown by the diminishing areas with good water quality as clear in the 2003 isosalinity map compared to that of 1971 (Fig. 2).

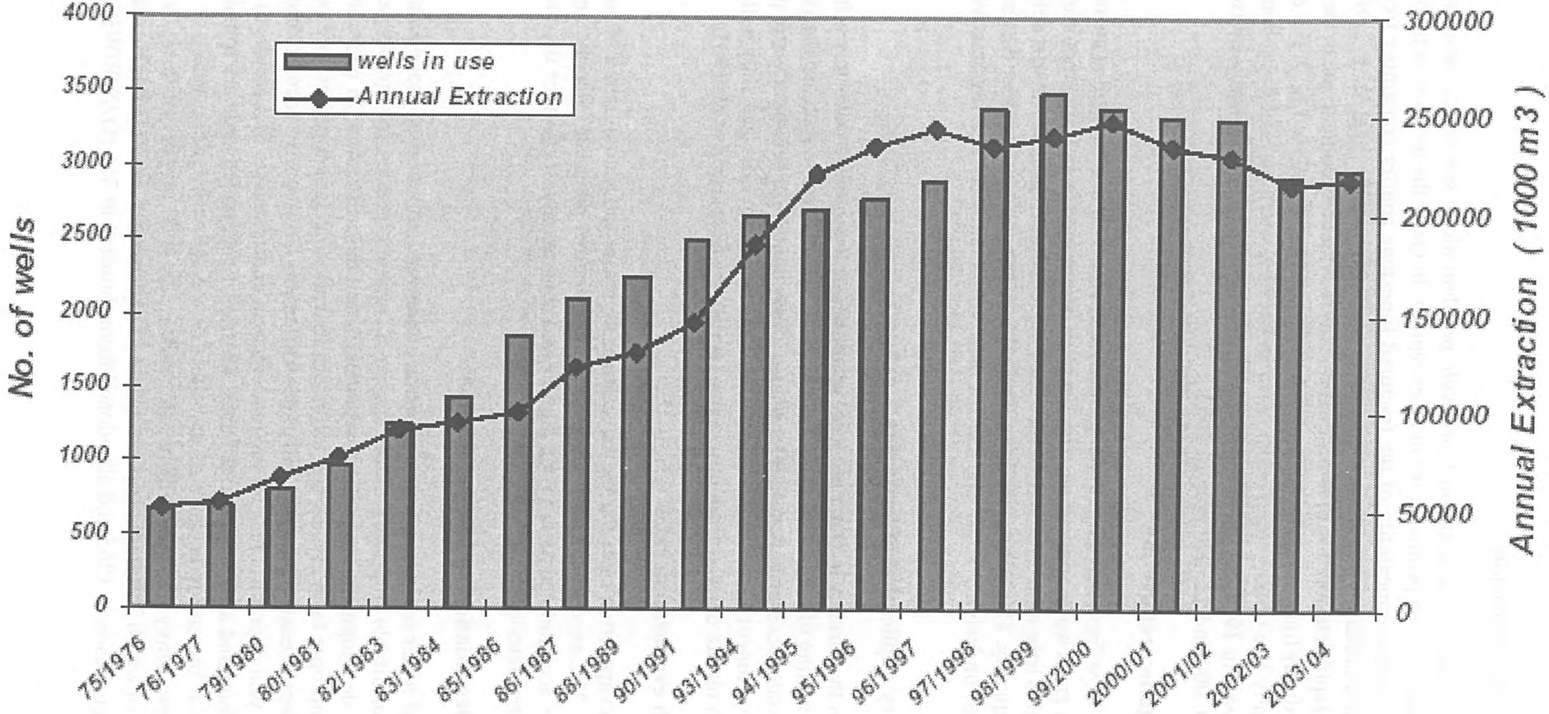

Fig. 1: Annual Groundwater Extraction and Wells in Use

2.1.2 Seawater intrusion

Seawater intrusion is a common worldwide problem along sea coasts, peninsulas and islands. Qatar being a peninsula is not an exception. In Qatar the problem is more severe, because the high permeability of the fractured limestone aquifer containing fresh water permits rapid intrusion of seawater. Over-extraction from wells situated along the coasts caused the rapid deterioration of water quality. This is evident by comparison to the location of the isosalinity lines along the coasts for the years 1971 and 2003 in Fig. 2. For example, the isosalinity line of 6000 ppm in the north and northwestern parts of Qatar was aligned near the coast in 1971 but shifted inland and was replaced approximately by isosalinity lines of 10, 000 and 12, 000 ppm in 2003.

2.1.3 Irrigation return flow

The return flow from irrigation to the groundwater reservoir is estimated at an average of 25% of the gross water application. This has been determined from lysimeter observations. Although this irrigation returns flow increases the recharge to groundwater, it deteriorates the water quality. This is because the percolating poor quality water dissolves salts from the soil and underlying strata and carries them to the aquifers bearing relatively fresh water.

2.1.4 Heavy application of fertilizers

Sometimes the farmers in Qatar use large quantities of low quality water to wash the salts and avoid wilting of plants and apply heavy chemical fertilizers to increase the yield. This practice is not necessarily beneficial, because it may contribute to groundwater pollution. In the Government Experimental Farm, drainage water analysis has shown significant increase in nitrate derived from nitrogenous fertilizers (Eccleston et al. 1981).

2.1.5 High evaporation rates

The high evaporation rates during the summer increases the accumulation of salts in the root zone. The excess irrigation water (irrigation return) percolates deeply, and carries with it the accumulated salts to the aquifer and thus aggravates further the problem of groundwater deterioration.

2.2 Soil Degradation and Desertification

Scarcity of water resources, severe climatic conditions, pollution of groundwater, water logging, unsuitable cropping patterns, incorrect cultural practices, overgrazing and socio-economic development lead to soil degradation and desertification. In addition to these factors, improper farm layouts and erroneous irrigation designs together with poor water management intensified the problem of desertification. Consecutive accumulation of salts year after year degrades the soils and renders them unproductive. This results in stoppage of agriculture and abandonment of farms. Most of the degraded soils are found in farms located near the coasts because of the effect of the saline irrigation water or in inland farms where heavy textured soils become saline. The total number of farms was 434 in 1975/76 , of which 259 were in operation and 175 abandoned. In 2003/04 the total number of farms increased to 1265 and abandoned farms boosted to 356 (DAWR/WRS 2003).

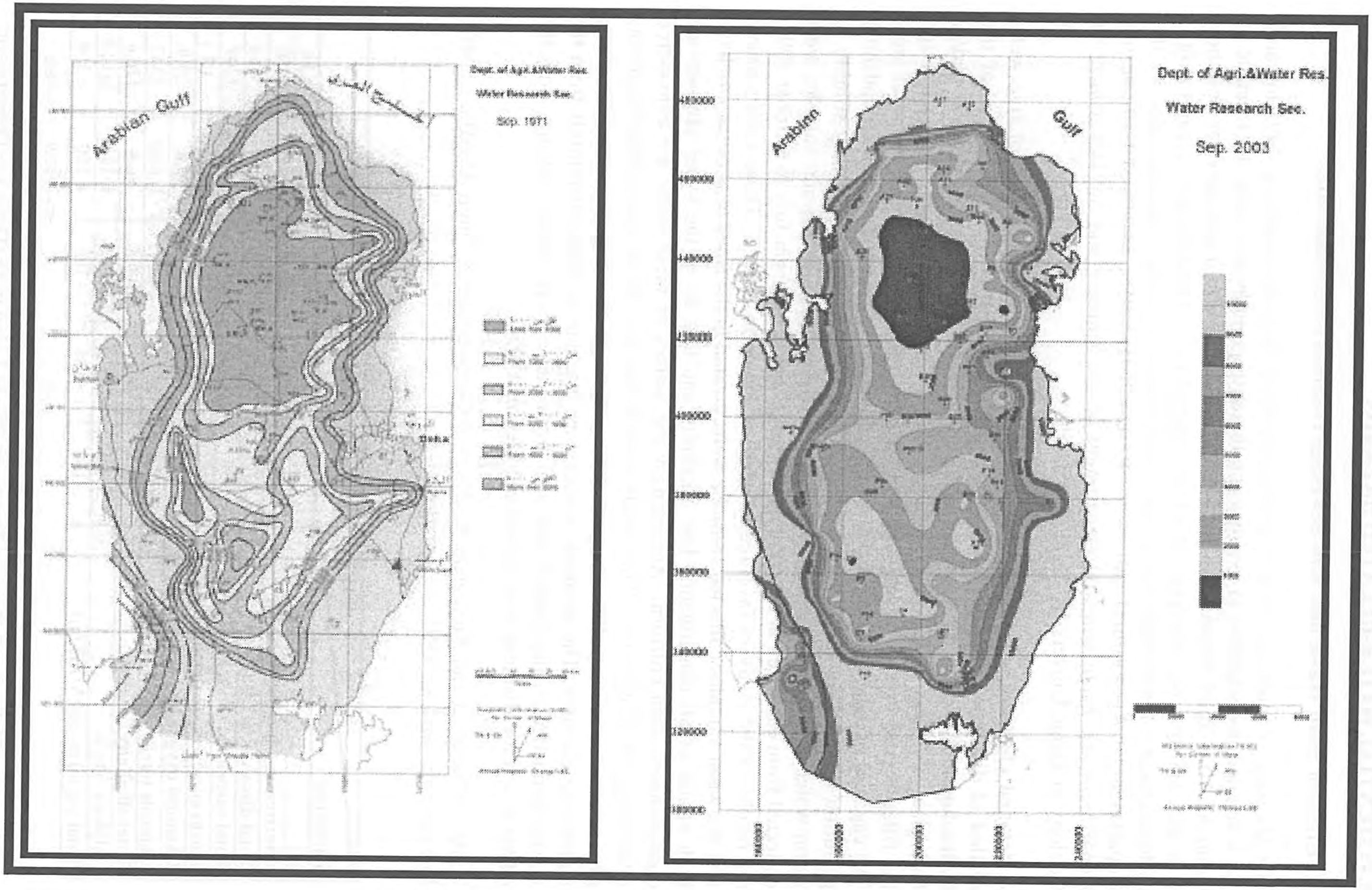

Fig.2: Isosalinty map for Qatar (1971) and (2003) in ppm

3. POTENTIAL WATER-RELATED HAZARDS

3.1 Effect of Solid Waste and Raw Sewerage Disposal on Groundwater

The Ministry of Municipal Affairs and Agriculture (MMAA) decided in 1995 to carry out a detailed investigation to evaluate the threat to groundwater from the leachate at Umm Al Affai Solid Waste Landfill. The extent and nature of groundwater pollution was determined by studying geology, geophysics, hydrology, climatology, hydrochemistry and topography of the landfill site. The study was executed by the Department of Agricultural and Water Research (DAWR), the Civil Engineering Department, the Environmental Department of the Ministry and the Department of Geology at Qatar University.

A detailed chemical analysis study was undertaken for water samples of five wells drilled at different locations at Umm Al Affai Landfill (Fig. 3). The aim of the hydrochemical section of the study is to investigate the extent of chemical and/or biological pollution of the groundwater resulting from leachate percolating through the filled wastes containing garbage and also from the nearby sewage disposal area. The capacity of solidwaste contribution to leachate generation and thus to groundwater pollution depends on the amount of the wastes components are readily soluble in storm water or that which are weakly bound in the wastes. Three water samples were collected from each well at different dates during the period from July to October 1995, the results of the analyses (average values) are shown in Table 1. These results reveal that the water samples collected from the wells located in the filled area of the site have high values of all constituents, low alkalinity and that all of the water samples are contaminated with coliform. Accordingly the waters of these wells are considered unhealthy for human use. This study concluded that there was pollution of the groundwater due to pollutants percolating through the highly permeable strata underlying the area. The probable sources are leachate in times of rainfall from the wastes deposited in the landfill and also from the seepage of raw sewage from the adjacent sewage disposal area (Abdulmalik et al. 1995).

Table 1: Chemical and Biological Analyses of Groundwater at Umm Al-Affai Landfill Site (average values), 1995

Parameter / Well No.	I	II	III	IV	V
pH	7.22	7.06	7.22	7.25	6.86
Electrical conductivity (µS/cm)	3253	3403	6537	10077	4950
Total dissolved solids (mg/l)	2277	3246	4576	7054	3464
Total suspended solids (mg/l)	355	1239	2319	1431	1249
Biochemical oxygen demand (mg/l)	1.4	1.7	4.2	1.7	1.7
Chemical oxygen demand (mg/l)	25.2	26.3	39.3	59.8	20.0
Chloride (mg/l)	914.0	955.7	1483.2	2669.0	1347
Total alkalinity (mg/l)	146.0	158.8	109.6	86.8	176.6
Sulphate (mg/l)	387.5	2742.5	3682.0	4510.5	2612.0
Total coliform (MPN/100 ml)	> 100	> 100	> 100	> 100	> 100

Analyses by: Doha South Sewage Treatment Works Laboratory (Drainage Department)

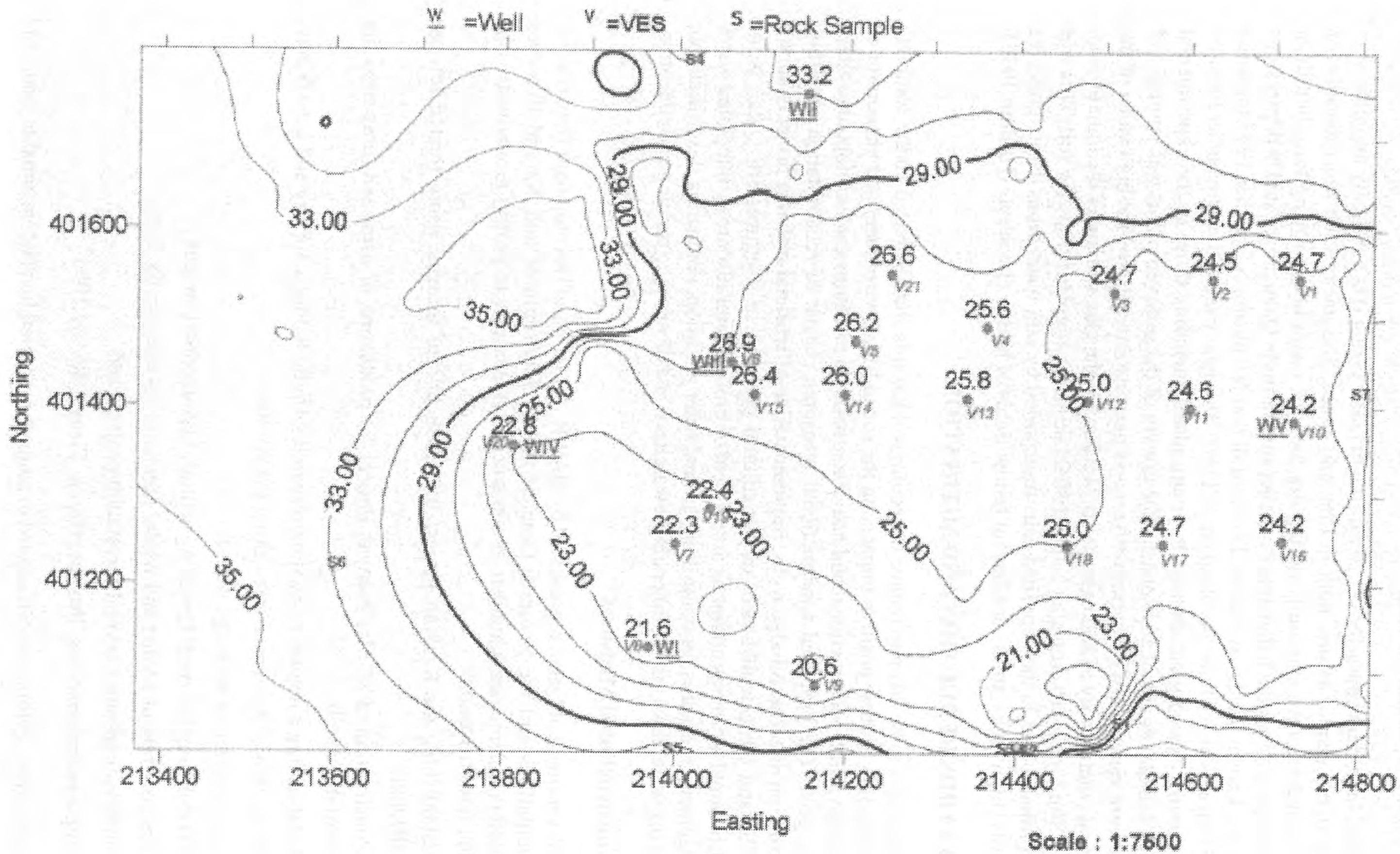

Fig.3: Location and Elevation (masl) of Wells at Umm Al Affai Landfill

3.2 Hazards of Thunderstorms

The most important characteristic of rainfall in Qatar is that it is extremely unpredictable and of highly erratic nature, both in time and space. The unexpected short duration of heavy thunderstorms, especially during dry months, result in destroying farm buildings and damaging crops. Such damages occur because farms in Qatar are located in depressions which are known locally as rodent. The rainfall that fell during November 1997 mounted to 155 mm of which 61 mm occurred on 11/11/97. This heavy rainfall in November (usually a dry month) caused great damage to young plants including those of the experimental crops of tomato, eggplant and squash. The growth of the survived plants was stunted and yields were very poor. This harmful effect was demonstrated also by the high value of the electrical conductivity of soil extract: 16.34 dS/m for the clay soil under the tomato crop, 13.91 dS/m for squash crop clay soil and 10.24 dS/m for eggplant crop clay soil. This was most probably due to the deposited salts transported by rain water from the neighboring lithosols (high rocky areas) or adjacent fallow fields of higher elevations (Hashim 1999).

4. HAZARDS REDUCTION AND MITIGATION

The measures to be taken to reduce the ecological hazards shall be based on the complete understanding of their positive impacts on the social life, environment and economics of the country. Both constructional and non-constructional measures can be adopted to attain such a goal. The proposed constructional measures consist of rehabilitation of farms, construction of extra recharge wells, implementation of artificial recharge projects, lining landfills and the establishment of agricultural projects, in southwestern Qatar. Non-constructional measures comprise assessment of groundwater resources, integrated water management, measures for controlling groundwater pollution, combating desertification, enforcing water laws, promoting public awareness, and developing training and education.

4.1 Constructional Measures

The Government of Qatar, represented by DAWR, Agricultural Development Department, Municipalities and the Supreme Council for the Environment and Natural Reserves (SCENR) implemented a number of constructional measures to protect groundwater and combat desertification:

- Establishing the Rakiyah Project for production of forages, using treated sewage effluent;
- Construction of Al Mas'habiyah Project for production of dates and forages on the sand dune soils;
- Establishing a natural vegetation research farm including a nursery and gene bank;
- Plantation of mangrove trees along the coasts;
- Construction of recharge wells;
- Establishing the Arab-Qatari Agricultural Production Company;
- Construction of cooled and winter green houses (Area = 68.2 ha);
- Implementation of several forestation projects and
- Implementation of the Project "Green, Clean Qatar by 2006".

However, more efforts are still needed to attain the required level of sustainable water and agricultural development.

4.1.1 Rehabilitation of farms

Rehabilitation of farms requires modifying the layouts, modernizing irrigation systems and designing new cropping patterns. Hashim and Abdulmalik (2001) proposed typical layouts of farms including circular mains, windbreaks, an irrigation piping network and drains. This layout proposes the construction of an intercept drain with a single protective embankment along the lowest part of the farm perimeter. The design depicted in Fig. 4 protects the farm from the destructive effects of heavy thunderstorms and from salinization by salts carried by rain water from near-by lithosols. The design also makes provision for drainage of excess rain water which would otherwise remain on fields of heavy texture soils, causing water logging and destroying young plants. The drainage water can be directed to relatively low depressions and recharged into the groundwater aquifer through specially designed recharge wells.

4.1.2 Construction of recharge wells

Drilling of wells (with special design that include perforated casing and graded gravels) in depressions to depths reaching the water bearing formation will accelerate natural recharge of floodwater. The project started in 1986 and the number of wells drilled up to date is 341. Continuation of this project to cover the whole 850 depressions could probably increase the natural recharge of the northern groundwater province by 17% (Harhash and Al Mahmoud 1991).

4.1.3 Implementation of a pilot artificial recharge project

The objective of the artificial recharge study is to determine the feasibility of a large-scale artificial recharge project to augment the depleting northern groundwater aquifer and improve the water quality. This study was implemented by Entec Hydrotechnica with cooperation of the DAWR and the United Nations Development Programme (UNDP) during the period 1992 to 1994. The study indicated that the artificial recharge freshwater (TDS 1000 mg/l) recovery efficiency or what is called "the user specific recovery efficiency" (water quality range 1000–3600 mg/l) could reach 100% in the Rus and transition Rus/UER zone. This study recommended the implementation of a pilot artificial recharge project in one of the promising regions by injection of the surplus desalinated water during the winter. The pilot project which will subsequently become an operational site can be used to investigate the impact of recharge on quantity and quality of groundwater and assess its economic feasibility (Entec Hydrotechnica 1993).

4.1.4 Saline water and sand dunes development in southwestern Qatar: a project for food and feed production

Based on local and worldwide research findings, Hashim (2003) proposed the establishment of a pilot project for the development of the coastal waste lands in southwestern Qatar. The objective of this project is to divert the coarse dune sand into irrigated farming, using the saline groundwater for production of salt-tolerant crops and seawater for production of Salicornia crop. The results show that implementation of this project could increase production of forage crops by 16.3% and raise the degree of self-sufficiency in potato from 0.1 to 34.8%. This project could, hopefully, lead to sustainable agriculture and contribute to combating desertification and preserving the environment.

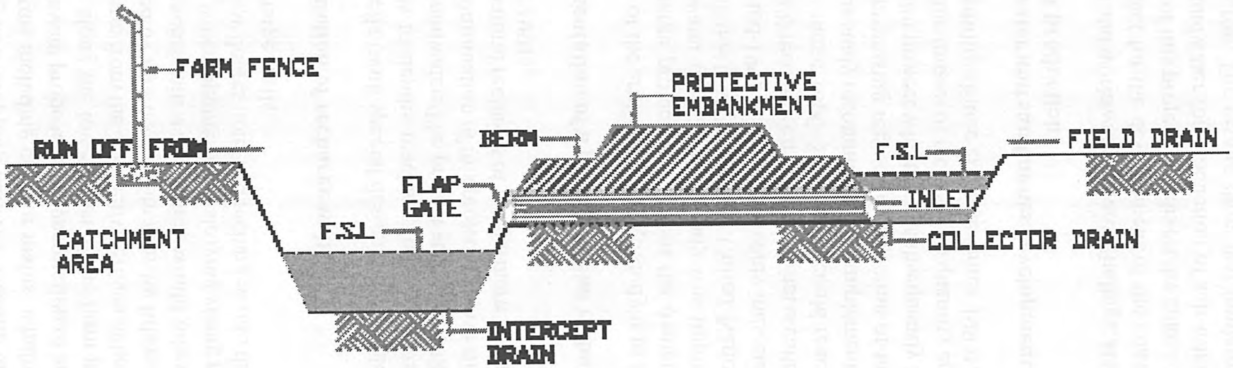

Fig.(4) Proposed Drain and Protective Embankment

4.1.5 Lining of Umm Al Affai Landfill

To prevent further pollution of the groundwater by leachate from the existing and future wastes deposited in Umm Al Affai Landfill, the following measures have to be implemented. Confine at the present time dumping of the solid wastes on the part of the site which has already been used for deposition and continue filling that part to its maximum settling level and then implement partial closure with a final cover. While doing so, measures shall be taken to design and implement a suitable lining of the unused part of the landfill. Phases of project implementation shall be arranged in such a way that the lining of the unused part of the site shall be completed before the partial closure of the filled part so that filling in the unused lined part will start before the partial closure of the site (Abdulmalik et al. 1995).

4.2 Non-constructional Measures

DAWR implemented several projects that provide reliable data for formulation of plans to combat drought and desertification. These include the topographical survey for the farms and arable lands in 1981-86 and Agricultural Census in 2000/01, automatic hydrogeological and meteorological monitoring network in 2001-04 and a recent soil survey, classification and land use specification. Other non-constructional measures which have to be carried out are explained briefly hereinafter.

4.2.1 Assessment of groundwater resources

The DAWR is planning to carry out a technical study to investigate the current status of the groundwater resources. The evaluation of the actual status of groundwater quantity and quality is an essential requirement for proper plan, design, construction, operation and maintenance of irrigation projects. Therefore state-of-the-art techniques and methods in hydrogeology and data management shall be combined to establish a comprehensive groundwater information system as a powerful planning tool for a sustainable water use.

4.2.2 Integrated water resources management

According to the Dublin Statement (1992), there emerged four main principles that need to be applied in taking actions to achieve integrated water resources development and management. The first principle is a holistic approach, the second is a participatory approach, the third is a pivotal role of women and the fourth principle is that water should be considered an economic good. This requires changing the organizational structure of the existing institutional setup and formation of the central water body for establishing and implementing a sustainable national water policy. It is worth mentioning that the Permanent Committee for Water Resources has been established by the Ministers cabinet decree No. 7 for the year 2004.

4.2.3 Groundwater Protection

In addition to the measures taken by DAWR to increase irrigation efficiency and decrease groundwater depletion (See section 4.2.5), the following water pollution prevention control measures are proposed:

- Preventing the sewerage tankers from emptying their loads on depressions. plans shall be made for all the sewerage tankers loads to be discharged in the sewage treatment plants;

- Declaring groundwater protection areas in the most vulnerable areas in north and central Qatar;
- Issuing regulation of siting, construction, operation and closure of landfills;
- Setting out quality criteria for treating wastewater from industries before draining into wells;
- Fixing maximum permissible application rates of manures, fertilizers and pesticides to minimize the effect of nitrates on health;
- Supervising proper storing, handling and disposal of hazardous chemicals;
- Mixing of fertilizers and pesticides to be carried out away from wells;
- Supervising adequate sealing of abandoned wells by using pentonite products;
- Checking maintenance of petrol pipes and tanks and sewage piping systems;
- Establishing laws and regulations with regard to wastewater treatment and recycling and enforcing the current groundwater laws (See section 4.2.5);
- Heightening public awareness and increasing training and educational activities (See section 4.2.6).

4.2.4 Measures to combat desertification

In addition to the non-constructional measures mentioned previously, which contribute also to combat desertification, Al-Yousif (1977) proposed the following:

- Taking precautionary measures for those lands which have not yet deteriorated or those that have only slightly deteriorated;
- Formulating plans for drought emergency at the local, national, regional and sub-regional levels;
- Compiling a desertification map of Qatar;
- Monitoring desertification due to desert creep and/or salinity from the decline of water quantity and quality;
- Surveillance of human behavior in the vulnerable environment;
- Making laws that restrict overgrazing and prevent desertification of natural pastures.

4.2.5 Enforcement of groundwater laws:

Based on the recommendations of DAWR, an Ameri Decree No. 1 (1988) was issued for governing drilling of wells and use of groundwater. The MMAA formed "The Permanent Committee for Farms, Wells and Organizing Farmers' Affairs" to be responsible, in addition to other duties, for implementing groundwater laws. Unfortunately, the only articles which have been implemented are those connected with granting permits for drilling, alteration and modification of wells. What is required now is to put into action the articles concerning water use and specifically the following (Hashim 1995):

- Installation of water meters in all wells in the State;
- After completion of water meters installation, it should be closely observed that the water allocated for each farm shall not be exceeded;
- The farm owner shall not irrigate more than the specified area and shall not install any water conveyance and irrigation systems in contravention of the instructions issued by DAWR;

- The owner of the farm shall be required to take all necessary steps for the protection and maintenance of wells, pumps, conveyance and distribution pipelines, irrigation systems and all control devices.

Notwithstanding the implementation of groundwater laws, DAWR has taken several measures to increase the irrigation efficiency and decrease groundwater depletion:

- A ban on the drilling of new boreholes in the mostly affected areas where there is excessive abstraction or where the water salinity of wells exceeds 12 000 μmhos/cm;
- Stop awarding permits for establishing new farms or extending existing farms until the aquifer will be returned to its equilibrium state;
- Encourage the shift to protected agriculture;
- Expropriate all unproductive farms and some of the farms located in the groundwater good quality zone and compensate their owners;
- Making full use of the non-conventional water resources in the irrigation of crops. These include the use of treated sewage effluent and the possible use of the gas-to-liquid industrial water;
- Study the possibility of introducing pricing systems for water consumption with penalties for extravagant water use and incentives for water saving;
- Provide interest-free loans to farmers to promote modernizing irrigation systems with a repayable period of several years.

4.2.6 Public awareness and education

It is anticipated that public awareness could be one of the most effective measures for mitigating water-related hazards and combating desertification. Proper education and training programmes could result in considerable water saving and consequently may lead to canceling some of the expensive water enhancement projects or at least postpone their implementation. The State of Qatar has launched several public awareness and education programmes on conserving water resources and combating desertification.

The programmes have been executed by DAWR, SCENR and the Qatar General Electricity & Water Corporation. The Environmental-Friends Center (NGO) has also participated in public awareness especially among students and youth. The salient features of these programmes include the following:

- Organizing field days and exhibitions;
- Conducting specialized lectures, seminars, conferences, symposiums and workshops;
- Issuing technical bulletins, folders and posters;
- Displaying films, presenting TV programmes, broadcasting programmes and publishing articles in newspapers;
- Running campaigns;
- Arranging competitions among school children;
- Celebrating the World Water Day (22nd March), the GCC Water Week (22-28 March), the Arab Environmental Day (14th October), the Qatari Environmental Day (26th February, and the Gulf Environmental Day (24th April).

Achievement of greater cooperation and involvement of the public need the implementation of more future plans which can be summarized as follows:

- Issuing: "Water and Land Conservation Awareness Guidebook". The guidelines should make clear to the public, at all levels, the present situation of groundwater depletion and low-quality, the high cost of seawater desalination and wastewater treatment and conveyance, causes of water-related ecological hazards and their remedies and the need to conserve water and land resources for the future generations;
- Establishing permanent water and agricultural museums;
- Increasing the number of seedlings of wild trees and shrubs being distributed to the public and support their plantation;
- Establishing an emergency preparedness team to fight the water-related ecological hazards. This needs coordination between the relevant departments and organizations and
- Allocating a special budget for the execution of the above mentioned programmes.

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Microbial Risk Assessment Concept in Developing Wastewater Reuse Guidelines in the Gulf Countries

Tahir Husain and Ekram Y. Danish

MICROBIAL RISK ASSESSMENT CONCEPT IN DEVELOPING WASTEWATER REUSE GUIDELINES IN THE GULF COUNTRIES

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ABSTRACT

The Gulf countries are facing acute water shortage and as a result, a significant amount of non-renewable deep groundwater resources has been exploited in the last 20 years. To meet the increasing demand of water, treated wastewater effluent for landscaping, irrigation, and aquifer recharge are quite popular. Reuse of treated wastewater, although a quite viable alternative for the future, must be evaluated for environmental and health impact of the crops, on farm workers and exposed population from enteric viral and bacterial infections. This paper presents a step-by-step approach on quantifying risk due to the presence of microbes in the wastewater for agricultural uses. The proposed methodology will help in developing cost-effective and environmentally safe wastewater reuse guidelines for the region by incorporating exposure patterns, climatic conditions, and socio-cultural environmental factors in the region. The concept is applied on the use of cucumbers grown using raw and treated wastewater and it can be extended to develop guidelines for other uses of wastewater effluent in the region. The preliminary investigation presented in this paper shows that the risk-based methodology for the site-specific situation will help in developing sustainable and cost-effective wastewater treatment methods without causing any detrimental effects on human health. The current reuse guidelines in the Gulf countries usually requires tertiary treatment with limited reuse practice and specify very strict effluent quality standards The stringent regulations and advanced treatment technology (tertiary treatment with disinfection) may not be needed for some specific uses of wastewater. In many cases primary and secondary treatment would suffice environmental and health requirements. This will save significant amounts of resources. However the effectiveness and reliability of decisions made using a risk-based approach will rely on accurate and systematic data collection, on waste characterization and laboratory and field investigations on the inactivation of microorganisms in the climatic conditions prevailing in the Gulf region.

Keywords: dose-response, exposure assessment, hazard identification, human health, microbial risk assessment, wastewater reuse

1. Wastewater Reuse in the Gulf Countries

More than 80% of total water demand in the Middle East region is for agriculture use. To meet such increasing demand, treated wastewater reuse is considered as an important viable long-term alternative which not only preserves high quality, expensive fresh water for drinking but it is also much cheaper than developing new supplies in the region. Other than this, treated wastewater effluent is rich in nutrients needed for plant growth. Kuwait, Saudi Arabia, Oman, the United Arab Emirates, and Qatar have introduced wastewater treatment and reuse policies in their developmental plan.

The Ministry of Agriculture and Water in the Kingdom of Saudi Arabia has set specific standards for the use of wastewater in agriculture. These specific standards are listed in Table 1. To meet these standards, tertiary treatment of the wastewater including disinfection is required and the use of the treated water is restricted to agricultural activities (e.g., irrigation of green fields, gardens or crops not eaten fresh), limited industrial uses, and aquifer recharge.

Table 1: Specifications on treated wastewater for irrigation in Saudi Arabia (Al-Jal'oud, 1997)

Element	Maximum content permissible limit (ppm)	Element	Maximum content permissible limit (ppm)
pH	6-8.4	Lead	0.1
BOD	10	Lithium	(0.07)
Suspended solids	10	Manganese	0.2
Aluminum	5	Mercury	0.001
Arsenic	0.1	Molybdenum	0.01
Beryllium	0.1	Nickel	0.02
Boron	0.5	Nitrate	10
Cadmium	0.01	Selenium	0.02
Free chlorine	0.5	Vanadium	0.1
Cobalt	0.5	Zinc	4.0
Copper	0.4	Phenol	0.001
Sulfur	0.05	Coliforms	2.2 units/100 mL
Fluoride	2.0	Turbidity	1.0 unit
Iron	5.0	Intestinal Nematodes	1 live egg/L

The Gulf countries have recognized the importance of the reuse of treated wastewater effluents. Several advanced wastewater treatment plants have been constructed as listed in Table 2. However, despite initiatives taken by these countries in developing treatment facilities, reuse is still less than 30% of the treated wastewater. Oman produces 23,000 m^3/day of treated wastewater which is fully utilized for landscaping. In Bahrain, 154,000 m^3/day of water is treated but only 30,000 m^3/day is used for irrigation and landscaping which accounts only for 20%. The remaining 80% is discharged into the sea. Kuwait has four large tertiary treatment plants with 208,000m^3/d treated waste. 62% of this water is used for landscaping, irrigating tress planted on roadsides and developing green belts in the coastal areas. In Qatar, 75,000 m^3/day water is treated and almost all of it is used for fodder crops, tress and landscaping. In the United Arab Emirates, 300,000 m^3/day of water is treated and used to irrigate green areas. Saudi Arabia has a capacity to treat more than 1.2×10^6 m^3/day of wastewater with tertiary treatment. Only 22% of the treated wastewater is utilized for fodder crops, landscaping, aquifer recharge and developing greenbelts in coastal areas and highways, the remaining 78% is discharged into the sea.

Table 2: Treated and reused water, treatment facilities and water utilization in the GCC countries (Al-Zubari, 1997)

Country	Treated waste-water (m^3/d)	Reused water			Treatment facilities		Type of water utilization
		Volume (m^3/d)	Percent	Number of plants	Total capacity (m^3/d)	Level of treatment	
Saudi Arabia	1,230,000	275,000	22	30	>1,230,000	Tertiary & secondary	Crop and highway irrigation, landscaping and artificial recharge
Bahrain	154,000	25,000-30,000	16-20	1 (large)	158,000	Tertiary	Irrigating fodder crops and gardens, and highway landscaping
Kuwait	208,000	129,400	62	4 (large)	208,000 (354,000)	Tertiary	Irrigating crops, highways, coastal zones and the Kuwait zoo
Oman	23,000	10,850-17,350	54-86	2 (large) 53 (small)	24,000 50-5,000	Tertiary	Irrigating landscape areas and parks, recreational activities, and fountains
Qatar	75,000-80,000	69,000	92-86	2 (large) 9 (small)	80,000 120-3,000	Tertiary & secondary	Fodder crops, gardens, and landscaping
United Arab Emirates	482,000	170,000	61	4 (large)	295,000	Tertiary	Irrigating parks, golf courses, highways and urban ornamentals

There are several benefits from utilizing wastewater effluent. It provides additional water sources for agriculture and hence helps in increasing supply of conventional water resources which is very critical in the Gulf countries. Furthermore, it provides natural fertilizers with increased productivity. Compared to other alternatives of water resources, wastewater is relatively cheap. The cost of the treated wastewater varies from \$0.11 to \$0.43 per cubic meter depending upon the degree of treatment, plant capacity, and other infrastructures required to water conveyance and distribution. The cost of desalinated water in the region varies from \$1.70 to \$2.45 per cubic meter (Ukayli and Husain, 1988 and Husain and Ahmed, 1997). Another economic incentive for treated wastewater effluent reuse is the use of nutrients present in the treated water (nitrogen, phosphorus). Study shows that about \$200 worth of phosphorus and potassium per hectare is saved by irrigating land using treated wastewater instead of regular water. Increase in the crop productivity by almost 30% has also been observed if treated wastewater is utilized.

National wastewater regulations drafted in 1984 in Saudi Arabia require secondary or tertiary treatment, depending on the planned reuse practice, and specify very strict effluent quality standards. It is likely that all sewage treatment plants in the country will be upgraded gradually to include tertiary treatment, but there is a growing concern about both the need for such strict standards and the ability to meet strict effluent standards at all times. The reuse of treated wastewater, which is part of the Kingdom's declared water resources strategy, would be difficult to put into practice because the stringent effluent standards might not always be met unless even more sophisticated treatment technologies are used. Also depending upon its uses in site-specific situations, such strict regulations may not be needed if the environmental human health risks to the exposed population are within the acceptable range.

The principal infectious agents, which are usually present in the wastewater, are grouped into (a) bacteria, (b) parasite (protozoa and helminthes) and (c) viruses. The probability of infection and illness depends on factors such as the numbers of microorganisms getting into the host known as dose, the minimum numbers of microorganisms causing infection, known as "infective dose" and the microorganism's ability to cause disease (pathogenicity). Since infective dose is defined as the dosed numbers of microorganisms starting immunological response by a host, the actual numbers of microorganisms showing signs of a disease could be much higher than the infective dose. Also, susceptibility is highly dependent on the host immunity and health conditions of the exposed individual. Infants, elderly persons, malnourished persons and persons with illness are more susceptible to illness than a healthy person. If treated wastewater is not properly used, it can pose threat to human health and ecosystems.

2. Quantitative Microbial Risk Assessment

Microbiological risk assessment is a tool, which is used to predict the consequences of potential or actual exposure to infectious microorganisms (Haas et al, 1999). It was first developed to estimate risk due to presence of Giardia and viruses in drinking water supply (Regli *et al.* 1991) and later for estimating risk of wastewater effluents for irrigating crops and discharge to recreational impoundments (Ashbolt *et al.* 1997; Shuval *et al.* 1997; Tanaka *et al.* 1998). Asano et al (1992) estimated risk of infection from poliovirus 1 and 3, and echovirus 12 using chlorinated tertiary effluents to irrigate market-garden produce, landscaping of golf courses and groundwater recharge. QMRA is similar to the concept on chemical risk assessment as proposed by the National Academy of Sciences (NAS, 1983) and it can be quantitative or qualitative depending upon data and methods used. It has four basic elements - hazard identification, dose-response assessment, and exposure assessment and risk characterization. The interrelationships of these elements are shown in Figure 1.

Hazard Identifications

In a microbial risk assessment, hazard identification involves identifying pathogenic organisms that can be transmitted by wastewater reuse. The list of potential waterborne pathogens contains bacteria, viruses and protozoa, and helminthes. These organisms can be harmful causing infection and releasing toxins. There are a large number of potential pathogens and it is almost impossible to include all these pathogens for risk assessment due to lack of data on their occurrence and quantitative data to assess risk under exposures and pathways of transmission. From a technological viewpoint, it is best to select organisms with high persistence and with the highest resistance to destruction or inactivation.

Exposure Assessment

Exposure assessment in QMRA is a very critical component and requires input from hazard characterization, which in this case is the characterization of pathogens. It involves the determination of the pathogen characteristics with its ability to transmit to hosts (i.e., human beings) and cause diseases. The ability of pathogens to cause disease depends upon factors such as virulence and pathogenicity of microorganisms, microbial growth, survival and decay rate and their resistance to control or treatment methods, route of transmission and mechanism and route of infection, and potential for secondary transmission. The above characteristics of pathogens are evaluated under different temperature and nutrients availability. The occurrence of pathogens in a medium including their spatial and temporal variations in the form of counts under different climatic conditions is also important in exposure analysis. The ages structure of the population, pathways, pattern, frequency and duration of exposure, are also considered in the exposure assessment.

Dose response Assessment

The dose response assessment is the quantitative relationship between dose (the amount of pathogens that enters or interacts with host) and a response (such as infection or illness). It is used to characterize the severity and magnitude of infection or illness due to the exposure to a particular pathogen. Development of dose response relationships is quite complex and requires information from animal studies, human clinical studies, and outbreaks. There are uncertainties involved in developing such relationship due to interspecies and interspecies variations, clinical dose, duration of exposure and latency period variations. Animal studies in the laboratories may be useful in determining dose-response curves but due to variations in body weight, age, latency period and duration of exposure, dose-response curves developed should be interpreted with caution. Dose response assessment based on epidemiological or clinical data would be a good representative for such studies but may require adjustment. There is a limited amount of information available in the literature on dose-response models.

Risk Characterization

Risk characterization takes input from exposure assessment and dose response relationships to estimate and characterize risk in terms of likelihood, type and magnitude of the infection and/or illness to the host (i.e., human beings) as a result of exposure to a single or group of pathogens. The likelihood or probability of adverse human health effects occurring as a result of a defined exposure scenario to a microbial contaminant or medium is the estimation of risk, an individual risk estimate or population risk estimate. Risk can also be characterized dynamically to consider the individual within a community rather than an isolated individual. Time-dependent elements such as secondary transmission, host immunity, and animal reservoir are included in dynamic models. Severity or outcome of an infection depends on several factors such as age, immune status, genetic background, medical treatment, age and nutritional status of the subject.

In case of microbial risk assessment, there are two sequential steps causing infection to a subject. In step 1, the subject which is usually a human host must be exposed to the organisms at a dose which can cause disease. The second sequential step is that the organisms in the subject body will go through a process of decay, excretion and multiplications. The antimicrobial defense mechanisms in the human body will resist to disease. However, once the level of microorganisms in the human body are sufficient enough to cause infection, the exposed person will get infected and fall sick. These two sequential processes are mathematically expressed as follows:

In the first sequential process, assume that a person is exposed (mainly through ingestion) 'j', organisms with corresponding mean dose 'd'. The dose is usually the product of the volume of water (say liter) in one exposure and density of microbes in the water (say counts/liter). The probability of ingesting j organisms given that the dose is d is $P_1(j|d)$. This probability takes into account individual to individual variation in actual number of organisms ingested.

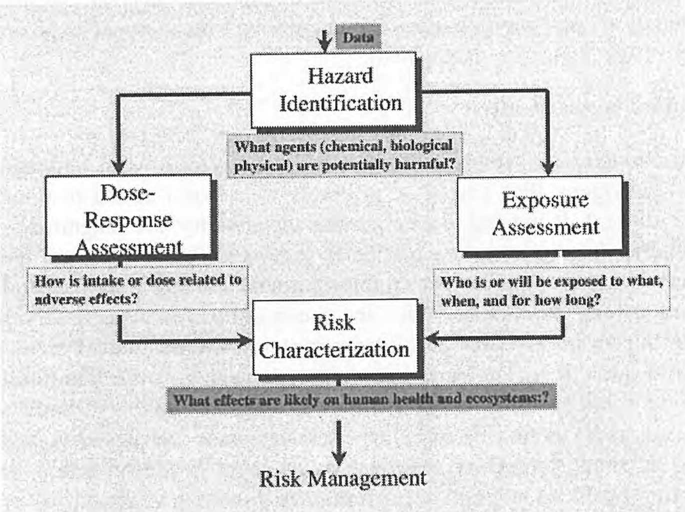

Figure 1: Interrelationship of various elements in risk characterization and risk management

Also assume that once j organisms are ingested, there are k organisms (k£j) which survive after undergoing decay and excretion in the body of exposed person and its probability is $P_2(k|j)$.This probability takes organism-to-host interaction, defense mechanism, immunity and health condition of exposed person. If the above two processes are independent, the overall probability of k organisms surviving to cause infection is written as follows:

$$P(k)=\sum_{j=1}^{\infty}P_1(j|d)\,P_2(k|j)$$

(1)

The exposed person develops infection if at least a critical number of organisms survive to initiate infection. Assume that k_{min} is such a number. The probability of infection to the fraction of subjects exposed to an average dose 'd', denoted by $P_I(d)$ can be written as:

$$P_I(d) = \sum_{k=k_{min}}^{\infty} \sum_{j=k}^{\infty} P_1(j|d)\, P_2(k|j) \tag{2}$$

There are several types of dose response models available in the literature but exponential and Beta-Poisson dose response models are the most commonly used in microbial risk assessment studies. In case of exponential dose response models, the distribution of organism between two doses is assume random and follows the Poison distribution. In this case, it is also assumed that that each organism has an independent and identical survival probability, 'r' and 'k_{min}' is assumed one. From these assumptions, $P_1(j|d)$ can be written as:

$$P_1(j|d) = \frac{d^j}{j!} e^{-d} \tag{3}$$

The second sequential probability component which relates organism-host interaction and deals with the survivability of organisms in the host body, follows binomial distribution and is mathematically represented as:

$$P_2(k|j) = \frac{j!}{k!(j-k)!}(1-r)^{j-k} r^k \tag{4}$$

The fraction of the subjects who are exposed to an average dose and get infected is denoted by $P_I(d)$ in Equation 2. By substituting the values of P1(j|d) from Eq. (3) and P2 (k|j) from Equation 4 in Eq. (2) yields

$$P_I(d) = \sum_{k=k_{k\,min}}^{\infty} \sum_{j=k}^{\infty} \left[\frac{d^j}{j!} e^{-d}\right]\left[\frac{j!}{k!(j-k)!}(1-r)^{j-k} r^k\right] \tag{5}$$

By rearranging and simplifying, we get,

$$P_1(d) = 1 - \sum_{k=0}^{k_{min}-1} \left[\frac{(dr)^k}{k!} e^{-dr}\right]$$

If $K_{min} = 1$, then:

$$P_1(d) = 1 - \sum_{k=0}^{k_{min}-1} \left[\frac{(dr)^k}{k!} e^{-dr}\right] \tag{6}$$

The above equation is known as exponential dose-response model. It is a simple and commonly used model for microbial risk characterization. The only drawback of this model is that it assumes a constant pathogen-host survival probability 'r'. In the real situation, due to diversity in host response and pathogen diversity, the r should follow certain probability distribution. Armitage and Spicer (1956) defined 'r' as beta distribution as follows:

$$P_I(d) = \frac{\Gamma(\alpha+\beta)}{\Gamma(\alpha)} \sum_{j=1}^{\infty} \left[\frac{\Gamma(\alpha+j)}{\Gamma(\alpha+\beta+j)} \frac{(-1)^{j-1}(d)^j}{j!} \right] \tag{7}$$

Due to computation limitations, the above model had limited use and at a later stage, Mickey (23,24) simplified the above model as follows:

$$P_I(d) = 1 - \left(1 + \frac{d}{\beta}\right)^{-\alpha} \tag{8}$$

Where b can be approximated in term of median infectious dose N50 and a as follows:

$$\beta = \frac{N_{50}}{2^{1/\alpha} - 1} \tag{9}$$

By substituting the value of b in Eq. (9), the relationship of $\mathbf{P}_I$(d) in terms of N50, dose d in the wastewater and a can be rewritten as follows:

$$P_I(d) = 1 - \left(1 + \frac{d(2^{1/\alpha} - 1)}{N_{50}}\right)^{-\alpha} \tag{10}$$

Due to the limited dose-response data on pathogens, it is difficult to derive model parameters very precisely. However, because of the importance of microbial risk assessment and pathogen outbreaks in the past, emphasis is made to develop dose response models. Haas et al. (1999) compiled such information which is listed as follows:

The above dose response relationships were developed based on extensive research and data obtained through experimental works on volunteers, outbreaks of specific diseases in a community, animal tests and epidemiological studies derived by various research groups. Such studies were initiated as early as the 1960s and still being validated and tested on other pathogens in the water. The values of the coefficients derived as listed in the above Table are based on best fit with uncertainties. Any conclusions drawn using this coefficient in risk characterization will help in establishing guidelines for treatment required to reduce pathogens in the water and are useful as a screening tool.

Other factors to be considered in the risk characterization are median infectious dose (N_{50}) - a dose at which 50% subjects will respond to the specific diseases. Although infection is considered as a symptom of illness but in order to be sick, the infection should be severe enough to produce symptoms in the exposed person in the form of diarrhea, vomiting or fever. Due to complexities involved in isolating pathogens, lack of analytical methods and diagnosis of specific diseases, it is difficult to collect precise infectious dose data. Also due to the degree of immunity, genetic history, health and nutrition levels, infectious dose can vary. Due to inadequate sanitation and poor quality water supplies, people from poor countries are frequently exposed to unhealthy living conditions but usually they enjoy higher immunity to microorganisms. On the other hands, due to unhygienic living conditions, lack of medical facilities, and malnutrition, these people are frequently susceptible to infection. High range in N_{50} on specific organisms indicates uncertainty in the median dose and requires further investigation. Also, the significance of low concentrations of microorganisms in mixtures has not been studied well and requires further research. More focus has been on individual microorganisms. Using dose-response models, extrapolating a low dose can be made but it makes risk assessment simplified and additive effects may not be a true representation of overall risk.

Another important factor in the microbial risk assessment is precise data on the survivability of microorganism under different environmental conditions. The survivability depends on a number of factors such as humidity, content of organic matters present in the medium, temperature, sunlight and pH. In the dry environment, microorganism will usually decay faster than in the humid environment. In the cold climatic conditions, microorganisms can survive for a longer period. Some bacteria can survive longer in alkaline soils than in acidic soil and sunlight works as a disinfectant to kill microorganisms. At this stage, there is very little data available except that some ranges are established on the survival of different waterborne pathogens in different media.

3. Analysis

In order to apply microbial concepts in wastewater reuse, a simple case study on the application of raw, primary treated, secondary treated, and tertiary treated water is presented in this paper. The first step in applying the concept is the selection of microorganisms which depend upon several factors. Low infective dose and frequent occurrence of epidemics and outbreaks, their high persistence in the environment and resistance to treatment, availability of data on their concentration in raw and treated water, dose-response relationships are some of the factors which will help in the selection of pathogens for the risk assessment study. It has been observed that in most of the wastewater treatment plants in developing countries, there is very limited data being collected on the count of microorganisms in both raw and treated water. Some of the basic pathogens which require to be studied are enteric viruses, *Salmonella typhi, Giardia, Shigella, and Giardia.* Revised WHO guidelines have listed data that refer to the content of different microorganisms in wastewater worldwide (WHO, 2003). The data is found very highly variable with significant differences between the level of pathogens in developed and developing countries. For this study we have used typical pathogen data as reported by Feachem et al. (1980), Engineering Science, Inc., 1987, and U.S EPA 1992). The microorganisms considered for the study are: Salmoneaa, Shigella, Fecal coliform and Giardia lamblia cysts. Using the levels of microorganisms listed in the Table 3, the probability of infection to individual based on single hit and annual mortality rate were calculated for a scenario where the raw and treated water is used to grow cucumber and cucumbers are eaten 3 days after irrigation and after washing. The following assumptions were made in the analysis:

- Average consumption of cucumber = 100 g per person per day
- Number of days cucumber used per year = 60 days per year
- The amount of raw water absorbed by the cucumber =0.36 ml per 100 g cucumber (Shuval et al., 1997)
- Inactivation and decay of pathogens in 2-days of irrigation can be up to <5log (Armon et al., 1994) but to be on the conservative side we assume 90% decay in 3 days.
- Removal of organisms by washing the cucumber before eating is assumed as 95% (Shuval et al., 1997)
- Intake of cucumber in Saudi Arabia is assumed as 100g/per person per intake for only 60 days a year since it is grown only in the winter season.
- Morbidity rate is highly variable (between 1% to 50%) and is dependent on dose, age group, and type of outbreaks. To be on the conservative side, it is assumed that 25% of the infected population will get sick although for small doses it is a relatively high number.

In order to translate probability of sickness to annual mortality rate, morality ratio data as compiled by Gerba et al. (1988, 1996) were used. NAS (1996) has compiled data on the characteristics of wastewater in terms of the typical number of microorganisms for raw sewage, primary treatment, secondary treatment, and tertiary treatment. The typical numbers of microorganisms per 100 ml of water are listed in Table 3. In hot and arid climatic conditions, the survival of these microorganisms will be much less than listed in the Table 3. However, assuming the above levels as the typical number, a microbial risk assessment study was conducted for *Salmonell, Salmonella typhosa, Shigella, Giardia,* and *Fecal coliforms.* Dose response models have been developed for these pathogens. For *Giardia*, the best fit model was exponential with r as 1/50.23 = 0.0199 (Rose et al. 1991). For other selected pathogens, the best fitted distribution model was Beta Poisson model with parameters as follows:

- Salmonella : N_{50} = 23,600, a = 0.3126 (Hoenick et al., 1966)
- Salmonellatyphosa : N_{50} = 3.60×10^6, a = 0.1086 (Hoenick et al., 1966)
- Shigella N_{50} = 1.120, a = 0.0.210 (Hoenick et al., 1966)
- Fecal coliforms N_{50}= 8.6×10^7 , a = 0.1778 (Medema et al. (1996))

Table 3: Typical number of microorganisms found at various stages of wastewater[1] (NAS, 1996)

Organisms	Number per 100 ml of effluents			
	Raw Sewage	Primary Treatment	Secondary Treatment[a]	Tertiary Treatemnt
Fecal coliforms MPN[b]	10^9	10^7	10^6	<2
Salmonella MPN	8,000	800	8	<2
Shigella MPN	1,000	100	1	<2
Enteric viruses, PFU[c]	50,000	15,000	1,500	0.002
Helminth ova	800	80	0.08	<0.08
Giardia lamblia cycsts	10,000	5,000	2,500	3

[1] Date sources: (Engineering Science, Inc., 1987, U.S. EPA 1992; and Feachem et al. 1980).
a Includes coagulation, sedimentation, filtration and disinfection
b MPN = Most Probable Number
c PFU = Plaque-forming units

The detailed analysis is presented in Table 4. The highlighted numbers in the Table show high risk (e.g., Shigella and Giardia). As shown in the Table, primary treatment meets health requirements for salmonella pathogens present in the primary treatment unit. The accepted annual level of risk for getting infected is 10^{-4} (U.S. EPA/U.S. AID, 1992) while the level of risk for both Salmonella and salmonella typhosa is less than 10^{-4} in primary treatment units. Even in raw wastewater, the risk $9.3x10^{-4}$ to $1.5x10^{-4}$ with 90% inactivation of microorganisms. In case of Giardia and Shigella, high risk has been estimated which can be reduced significantly if conventional filtration used is in the primary and secondary treatment methods. The filtration units are found very effective in reducing bacteria and protozoa up to $3\log_{10}$ reduction (Health Canada, 2003). Another way to reduce risk is to increase the duration between harvest and the last irrigation. It has been reported in the literature that if duration between harvest and last irrigation is increased beyond a week, more than 99% reduction in microorganisms can be achieved which will reduce risk from Salmonella to an acceptable level using raw wastewater effluent. A rapid die away or removal of indicators as well as pathogens by $<5\log_{10}$ has been reported within two days in the field conditions (Sadovski et al., 1978; Armon et al., 1994). The analysis presented in this paper does not have any data on the inactivation of microorganisms in hot and sunny conditions. It is advisable that inactivation rate under Saudi conditions should be monitored in the field conditions with time when waste is stored in the lagoons and while being treated through different detention tanks and filtration units.

Table 4: Risk of infection, disease and mortality from eating 100 g cucumbers daily for 50 days grown in raw and treated wastewater

Micro-organisms	Waste-water Type	Counts per 100 ml	Decay and Inactivation Efficiency		Dose		Dose Response Model			Single hit Infection Risk, $P_I(d)$	Annual Risk to Individual		Annual Mortality Risk to Individual
			3-day decay	Was hing	N or d Per 100ml	N or d ingested	Expo-nential	Beta-Poisson					
							r	N_{50}	α		Infected	Sickness	
Salmonella[a]	Raw	8000	90%	95%	40	0.14	--	23600	0.3126	$1.56x10^{-5}$	**$9.36x10^{-4}$**	$2.34x10^{-4}$	**$2.34x10^{-7}$**
	Primary	800	90%	95%	4	0.014	--	23600	0.3126	$1.56x10^{-6}$	$9.36x10^{-5}$	$2.34x10^{-5}$	$2.34x10^{-8}$
	Secondary	8	90%	95%	0.04	0.00014	--	23600	0.3126	$1.56x10^{-8}$	$9.36x10^{-7}$	$2.34x10^{-7}$	$2.34x10^{-10}$
	Tertiary	0.002	90%	95%	10^{-4}	$3.6x10^{-8}$	--	23600	0.3126	$3.9x10^{-13}$	$2.34x10^{-10}$	$5.85x10^{-11}$	$5.85x10^{-14}$
Salmonella typhosa[a]	Raw	8000	90%	95%	40	0.14	--	$3.6x10^{6}$	0.1086	$2.56x10^{-6}$	**$1.54x10^{-4}$**	$3.85x10^{-5}$	$3.85x10^{-8}$
	Primary	800	90%	95%	4	0.014	--	$3.6x10^{6}$	0.1086	$2.56x10^{-7}$	$1.54x10^{-5}$	$3.85x10^{-6}$	$3.85x10^{-9}$
	Secondary	8	90%	95%	0.04	0.00014	--	$3.6x10^{6}$	0.1086	$2.56x10^{-9}$	$1.54x10^{-7}$	$0.39x10^{-8}$	$0.39x10^{-11}$
	Tertiary	0.002	90%	95%	10^{-4}	$3.6x10^{-8}$	--	$3.6x10^{6}$	0.1086	$6.4x10^{-13}$	$3.9x10^{-11}$	$9.6x10^{-12}$	$9.6x10^{-14}$
Shigella[b]	Raw	1000	90%	95%	5	0.018	--	1.12	0.21	0.07098	**1**	0.25	**$0.5x10^{-3}$**
	Primary	100	90%	95%	0.5	0.0018	--	1.12	0.21	$8.6x10^{-3}$	**0.52**	0.13	**$0.26x10^{-3}$**
	Secondary	1	90%	95%	0.005	$1.8x10^{-5}$	--	1.12	0.21	$8.8x10^{-5}$	**0.0052**	0.0013	**$0.26x10^{-5}$**
	Tertiary	0.5	90%	95%	0.0025	$9x10^{-6}$	--	1.12	0.21	$4.41x10^{-5}$	**0.0026**	0.00066	**$1.32x10^{-6}$**
Giardia[b]	Raw	10000	90%	95%	50	0.18	0.01991	--	--	0.00358	**0.2146**	0.05365	**$1.14x10^{-4}$**
	Primary	5000	90%	95%	25	0.09	0.01991	--	--	0.00179	**0.107**	0.0268	**$5.37x10^{-5}$**
	Secondary	2500	90%	95%	12.5	0.045	0.01991	--	--	0.0009	**0.0537**	0.0134	**$2.69x10^{-5}$**
	Tertiary	3	90%	95%	0.015	$5.4x10^{-5}$	0.01991	--	--	$1x10^{-6}$	$6.5x10^{-5}$	$1.6x10^{-5}$	$3.2x10^{-8}$
Fecal coliforms[a] MPN	Raw	10^{9}	90%	95%	$5x10^{6}$	18000	--	$8.6x10^{7}$	0.1778	0.0018	**0.108**	0.027	**$2.7x10^{-5}$**
	Primary	10^{7}	90%	95%	50000	180	--	$8.6x10^{7}$	0.1778	$1.8x10^{-5}$	**$1.08x10^{-3}$**	$2.7 x10^{-4}$	**$2.7x10^{-7}$**
	Secondary	10^{6}	90%	95%	5000	18	--	$8.6x10^{7}$	0.1778	$1.8x10^{-6}$	**$1.08x10^{-4}$**	$2.7x10^{-5}$	$2.7x10^{-8}$
	Tertiary	2	90%	95%	0.015	$5.4x10^{-5}$	--	$8.6x10^{7}$	0.1778	$3.6x10^{-12}$	$2.16x10^{-10}$	$5.4x10^{-11}$	$5.4x10^{-14}$

[a] mortality ratio = 0.1%; [b] mortality ratio = 0.2%

4. Conclusions and Recommendations

1. Middle Eastern countries are facing acute water shortage. To meet the increasing demand for the basic water needs, long-term sustainable water resources alternatives need to be explored. Among various options, reuse of treated wastewater for irrigation is considered to be cost-effective strategic alternatives.

2. There is a serious concern on the use of treated wastewater effluent in the Gulf Countries due to presence of pathogenic microorganisms. To address this issue, more in-depth investigation is required by collecting data on microbiological characteristics for different degree of treatment.

3. The preliminary investigation presented in this paper shows that risk-based methodology for site-specific situations will help in developing a long-term cost-effective wastewater treatment method without causing any detrimental effects on human health. National wastewater regulations drafted in Saudi Arabia require tertiary treatment with limited reuse practice and specify very strict effluent quality standards. The stringent regulations and advanced treatment technology (tertiary treatment with disinfection) may not be needed for some specific uses of wastewater. In many cases primary and secondary treatment would suffice environmental and health requirements. This will save significant amount of resources. However the effectiveness and reliability of decisions made using a risk-based approach will rely on accurate and systematic data collection on waste characterization and laboratory and field investigations on the inactivation of microorganisms in the climatic conditions prevailing in the Gulf region.

In order to assess the environmental and human health impact due to wastewater reuse projects in the region, it is recommended that a systematic step-by-step approach be followed to conduct environmental impact and health assessment study. This will help in cost-effective strategies on the reuse of wastewater effluent.

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DESALINATION & TREATMENT TECHNOLOGIES

Power and Water Integration Innovation in Hybrid and Nanofiltration

Leon Awerouch

POWER AND WATER INTEGRATION INNOVATION IN HYBRID AND NANOFILTRATION

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INTRODUCTION

The hybrid desalting concept is the combination of two or more processes in order to provide better solutions, a lower cost product than either alone can provide. In desalination, there are distillation and membrane processes. Under hybrid conditions, they can be combined to produce a more economic process. Thus, two or three elements that are integrated to make hybrid desalination are:

- distillation- MSF, MED, MED-TVC, VC
- membrane desalination- RO, NF
- power. Steam Power Plants, Combined Cycle Power Plants

Large dual - purpose power desalination plants are built to reduce the cost of production of electricity and water. Over 30,000 MW of power is combined with desalination plants in the largest use of cogeneration concepts.

In many countries, particularly in the Middle East peak power demand occurs in summer and than drops dramatically to 30-40%, in contrast the demand for desalinated water is almost constant around the year. Therefore, the design of future plants requires careful consideration of power (MW) to water (MIGD) ratio PWR.

The examination is made of hybrid system both simple and integrated approach in order to take full advantage of both thermal and electrical energy.

Simple hybrid

In the simple hybrid MSF/RO desalination power process, a seawater RO plant is combined with either a new or existing dual-purpose MSF/power plant to offer some advantages. Several plants currently installed are using some of these advantages. Examples are in Jubail and Madina-Yanbu II in Saudi Arabia and Fujairah in UAE.

Integrated hybrid

The fully integrated MSF/RO desalination power process, which is particularly suitable for new sea-water desalting complexes, takes additional advantage of integration features.

Power/water hybrid

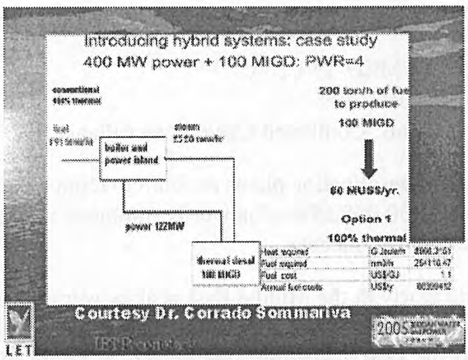
Integration of the power and water cycle is to obtain the optimum cost for both water and power. Important parameters in the design of these systems include:

- seasonal electric and water demand;
- power-to-water ratio.
- Minimize fuel consumption and increase in the power plant efficiency.
- Minimize carbon dioxide environmental impact, including potential consideration of CO2 tax credit.

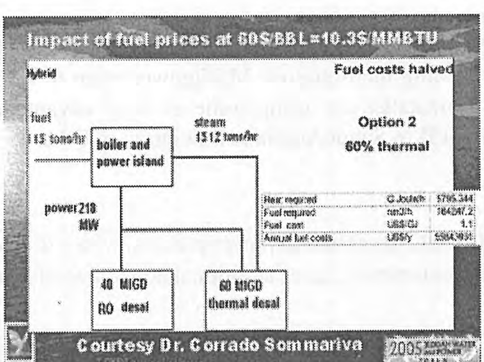
Some of the earlier analysis in the references showed when seasonal and daily variations occur; electrically driven technology can provide an excellent choice for hybridization with more conventional dual-purpose plants. The hybrid approach could achieve the lowest cost of total investment, flexibility in production and the lowest cost of power and water production.

The Energy Conservation using Hybrid

In view of dramatic rise in fuel prices in excess of US$ 60/barrel which is equivalent to 10. 3$/ MMBTU hybrid offers significant saving in fuel cost in comparison with only distillation option. This well demonstrated by simple presentation provided by Dr. Corrado Sommariva in his course on Thermal Desalination Processes and Economics.



In this case for 100 MIGD MSF desalination and 400 MW of power we will the annual fuel cost will exceed 86 million US $ based on fuel cost of only 1.1 US$/GJ. By comparison a Hybrid 100 MIGD desalination plant based on 60 % thermal and 40 % RO will reduced fuel consumption to only 55 million US$ per year.



This annual fuel cost difference of over 30 million per year is based on 1.1 \$/GJ, considering the impact of today's fuel price of 10.\$/GJ the annual cost differential will exceed 300 million dollars and will pay back for the total Capex in less then 3 years. Of course in base case we produce more power and to some extend this compensates the additional cost, but this assumes that we need the power.

Water can be stored while electricity storage is not practical. In this case excess electricity can be diverted to water production incorporating electrical driven Sea Water Reverse Osmosis (SWRO) and/or Vapor Compression and combined with low pressure steam driven technology of MSF or MED, making its advantages to design an integrated Hybrid Plants. One method of making use of idle power capacity is the use of electrically driven RO or VCD plants in combination with Desalination Aquifer Storage Recovery (DASR) both for averaging the desalination capacity, for strategic fresh ground water storage or improving quality of the basin.

The increase in the unit size of MSF, MED, VC and RO will lead to reduction of capital costs, but combined with unique application of hybrid ideas will offer reduction in water cost.

Effective integration of membrane/thermal desalination and power technology can reduce the cost of desalination and electrical power production (hybrid desalination). In the early 1980s MSF was the most prevalent seawater desalination process and seawater reverse osmosis was in its early stages of development. Because of RO's development status, two-pass RO is often required in MENA applications. Early suggestions for hybrid desalination were based upon elimination of the requirement for a second pass to the RO process so that the higher-salinity RO product could be combined with the better quality product from an MSF plant. This is the simplest application of hybrid desalination. Since then, other concepts have been proposed for hybrid desalination. Today although RO can produce product TDS in one pass, blending allows reducing the requirements for second and third partial pass to solve critical Boron issue.

The dual purpose power-desalination plants make use of thermal energy extracted or exhausted from power plants in form of low pressure steam to provide heat input to thermal desalination plants for multistage flash (MSF) or multi-effect (MED) distillation processes. The electrical energy can be also effectively used in the electrically driven desalination processes like Reverse Osmosis (RO) and Vapor Compression distillation (VCD) processes.

There are unique conditions in the Gulf where peak demand for electricity rises significantly during summer mainly because of the use of air-conditioning, and than drops dramatically to 30-40% of summer capacity. This creates situation that over 50% of power generation is idled. In contrast the demand for desalinated water is almost constant. Water can be stored while electricity storage is not practical. In this case excess electricity can be diverted to water production incorporating electrical driven technology of sea water Reverse Osmosis (RO) and/or Vapor Compression Distillation (VCD) and combined with low pressure steam driven technology of MSF or MED, making its advantages to design an Integrated hybrid plants.

HYBRID THE NEW ALTERNATIVE

The idea of combining electrical power, multistage flash (MSF) distillation and seawater reverse osmosis (RO) has been reported in a number of publications. Initial publications were in the early 1980s. The Hybrid Desalting Systems idea of combining power, multistage flash (MSF) distillation plant and membrane seawater reverse osmosis (RO) plant was previously reported to offer significant advantages [1-7,10,11]

In the simple hybrid MSF/RO desalination power process, a seawater RO plant is combined with either a new or existing dual purpose MSF/power plant with the following advantages:

- A common, considerably smaller seawater intake can be used.
- Product waters from the RO and MSF plants are blended to obtain suitable product water quality.
- Product waters from the RO and MSF plants are blended to allow higher temperature of distillate.
- A single stage RO process can be used.
- Blending distillation with membrane products to reduce strict requirements on Boron removal by RO.
- The RO membrane life can be extended. Excess power production from the desalting complex can be reduced significantly, or power to water ratio can be significantly reduced.

The fully integrated MSF/RO desalination power process which is particularly suitable for new seawater desalting complexes, takes additional advantage of integration features, such as:

- The feedwater temperature to the RO plant is optimized and controlled by using cooling water from the heat-reject section of the MSF/MED or power plant condenser.
- The low-pressure steam from the MSF/MED plant is used to de-aerate or use de-aerated brine as a feedwater to the RO plant to minimize corrosion and reduce residual chlorine.
- An integrated seawater pretreatment and post-treatment is used for the product water from both plants.
- The brine discharged-reject from the RO plant is combined with the brine recycle in the MSF or is used as a feed to MED.
- The hybridization of nanofiltration as softening membrane for feed of distillation plants MSF and MED could lead to significant improvement in productivity of desalination plants.
- Blending distillation with membrane products to reduce strict requirements on Boron removal by RO.

The "classic scheme"

This is the most common and straightforward hybrid plant scheme. It has been adopted in Jeddah to blend higher TDS permeate with distillate from existing MSF plants, and is described in detail by Awerbuch et al. [10] and by many other papers.

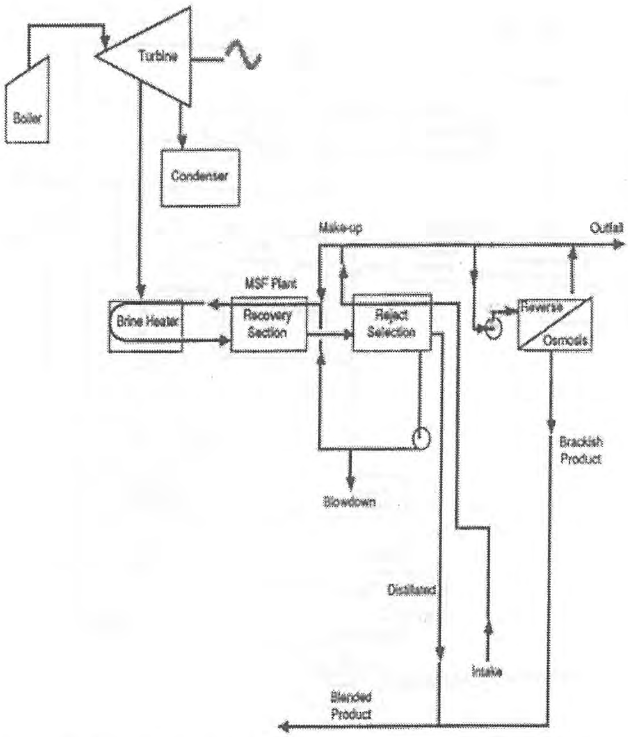


Figure 14. The "classic" hybrid SWRO/MSF desalination plant scheme, as proposed by L. Awerbuch *et al.* [10]

In general in this scheme part of the MSF plant's heated coolant reject is de-aerated, using low-pressure steam from the MSF plant (to reduce corrosion and residual chlorine), and used as the feed to the SWRO plant.

The higher temperature of the feed improves membrane performance (flux, at constant pressure, increases by 1.5–3% for each degree C). This is particularly important during the winter, when seawater temperatures can drop to as low as 15°C. The MSF plant's distillate, at less than 20 ppm TDS, is blended with the SWRO plant's product, making it possible to meet potable water standards for maximum TDS and chloride concentrations with higher SWRO plant product salinity. This, in turn, means that the SWRO plants can be operated at higher conversion ratios, thereby reducing consumption of energy and chemicals and extending membrane lifetimes.

The "classic scheme" variant

In one variant of the "classic scheme", the SWRO plant's reject brine becomes the feed to the MSF plant, utilizing its high pressure, with a special turbocharger, to boost the MSF plant's recirculation pump. The conversion ratio of the SWRO plant is then limited by the maximum brine recirculation concentration possible. With a once-through MSF plant this limitation is avoided.

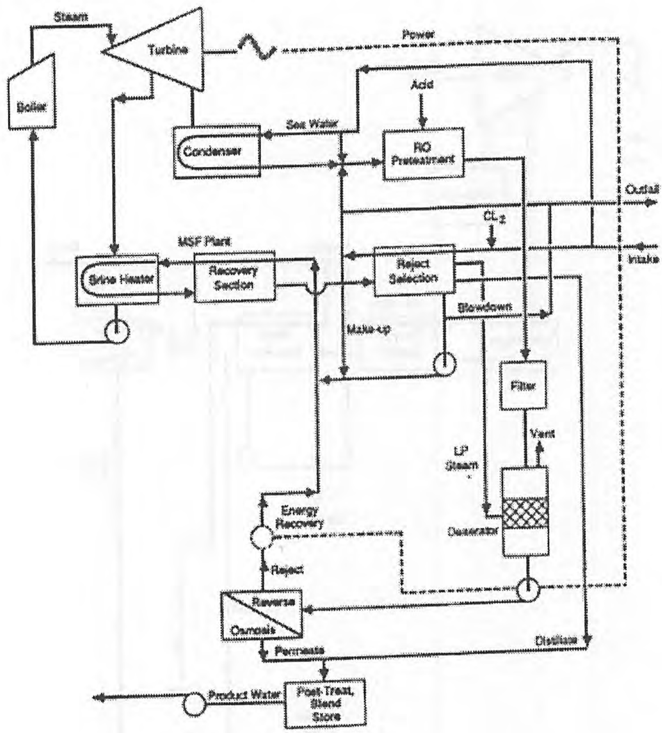


Figure 15. The "classic" hybrid SWRO/MSF desalination plant scheme variant, as proposed by L. Awerbuch *et al.* [10]

Current industry and academia R&D related to improving hybrid systems

The R&D activities pursued today that are most relevant to cogeneration and/or hybrid systems are those relating to the creation of a wider range of nanofiltration and SWRO membranes and the pilot-plant testing and prototype plant designing of low-cost high-GOR high-temperature MED plants. As Awerbuch [11] suggested, an optimal hybrid system would benefit from SWRO membranes with higher fluxes and lower rejections than currently being offered commercially. The minimal accepted membrane rejection will be that which will give a permeate with a salinity sufficient to provide, after dilution with the MSF plant's distillate, a combined product salinity of 500 ppm TDS. Some membrane manufacturers have been investigating the potential performance and markets for such high-flux SWRO membranes.

Examples of Existing Hybrids

Jeddah Hybrid The results of conceptual and design work [1, 3] led to construction of the simple hybrid project at Jeddah 1, phase I and II plants. The Jeddah 1RO plant is 30 Mgd (113600 m^3/day) combining Phase I which has been operated since 1989 and Phase II has been operated since March 1994. The plant is owned by Saline Water Conversion Corporation (SWCC), design by Bechtel, constructed by Mitsubishi Heavy Industries, Ltd., under the supervision of SWCC technical committee. Al-Badawi et al. [9] reports the operation and analysis of the plant which utilized Toyobo Hollosep double element type hollow fiber RO modules. The Jeddah complex in addition to 30 Mgd RO permeate, produces 80 Mgd distillate from Jeddah II, III and IV and 924 MW. Jeddah I RO plant adopted successfully an Intermittent Chlorine Injection method (ICI) in order to prevent membrane degradation by oxidation reaction and biofouling.

Yanbu –Medina Hybrid The drive to minimize power to water ratio lead to construction of Madina and Yanbu Phase II. Nada et al. [12] describes the design features of the largest SWRO plant in the Kingdom 33.8 Mgd in Madina and Yanbu. The plant is able to produce power 164 MW and 76 Mgd desalinated water. Two 82 MW back pressure steam turbine (BTG) provides steam to four 9.5 mgd MSF distillation units and the electricity to fifteen RO units of 2.25 mgd. Although the plant was not design as an integrated Hybrid it provided very good example of significant reduction of the power to water ratio (PWR).

Fujairah Hybrid.

This seawater desalination and power plant is the biggest hybrid configuration of thermal processes and reverse osmosis to be implemented up to now.

The very good paper presented by Ludwig, [2] describes in detail the design considerations for this hybrid plant.

For this plant a design decision was made to separate intake for the RO plant, through which the specific chlorination requirements for SWRO can be maintained. It was chosen over the use of a common seawater extraction system. Feeding of preheated cooling water from the MSF reject section to the RO plant was also rejected because, here too, only water that had been chlorinated continuously, and in part shock dosed, was available.

In my opinion this decisions are controversial and in the future more considerations could be given to take clear advantage of common intake and feed temperature control. A study of shock chlorination on top of residual chlorine or de-areation/dechlorination of RO feed could allow the benefits of hybrid integration.

The RO plant is designed as a two-pass system, specifically to obtain the low chloride and TDS contents of the drinking water required for reasons of corrosion suppression.

The blended product from MSF and SWRO is treated in a joint potabilization facility, supplied by CO_2 from the MSF vent gases. To cover one of the MSF units being taken out of service or for enhanced hardening of the water, the CO_2 demand can be met by an additional CO_2 generation plant.

The Fujairah plant due to hybridization generates only 500 MW net electricity for export to the grid, and 662 MW gross for water production capacity amounts to 100 MIGD. Otherwise similar MSF only plant in Shuiwaihat required 1500 MW for the same 100 MIGD capacity. The Fujairah desalination plant is split into 62.5 MIGD from the thermal part and 37.5 MIGD from the membrane process.

The power plant is configured as a combined cycle with supplementary firing. It comprises four gas turbines each rated 109 MW (oil- or gas-fired) and four heat recovery steam generators each of 380 t/h at steam parameters of 68 bars/ 537°C that supply the two steam turbine generators. The expanded steam from the turbines serves as process steam for the MSF units.

The seawater desalination processes are designed for seawater TDS of 40,000 ppm and a seawater temperature design range of 22 to 35°C. Specified for the drinking water product is a maximum TDS of 200 mg/l and its chloride content should not exceed ~ 85 mg/l.

The thermal part of the facility comprises five MSF units each of 12.5 MIGD capacity, with a performance ratio of 8 and a top brine temperature (TBT) range of 107 to 109°C.

The RO plant has a net permeate rating of 170.475 m³/d, corresponding to 37.5 MIGD The first pass is design for a recovery rate of 43% and consists of 18 trains, with 17 normally being in operation and one on standby. The second pass of eight trains has a capacity of 74% of the maximum total output of the SWRO, and is designed for a recovery rate of 90%. The blended product from thermal desalination and SWRO is treated in a common potabilization plant. The process employs dosing with lime water and injection of carbon dioxide. The latest description of the Fujairah Hybrid is contained in paper presented by Doosan [46] at IDA Congress in Singapore

Quantifying the benefits of the preferred hybrid SWRO/thermal plant scheme in cogeneration stations without mismatch problems

Savings due to reduced seawater requirements

The use of distillation plant coolant reject as feed to a SWRO plant within selected hybrid plant scheme reduces both seawater supply and brine and coolant rejection requirements vis-à-vis non-hybrid, separate and independent ("stand-alone") thermal and SWRO plants.

The cost savings are derived from four sources:

- reduced investment in the seawater intake and supply system;
- reduced investment in the brine and cooling water discharge system;
- reduced seawater pretreatment costs;
- reduced seawater-pumping energy.

Using 0.6 exponent for scale-ups and scale-downs Hoffman have computed the savings in intake and outfall system investments and annual operating costs (including pumping and treatment, i.e. chlorination) for a 150,000 m3/day hybrid plant, for the full range of SWRO to thermal plant outputs ratios. Figure 18 present these figures.

Unit costs (in US ¢/m3) relate to total intake and discharge costs per 1 m3 of seawater supply, and are based on a 20-year depreciation period, 7% interest, 8,000 h/year utilization, seawater pumping power at 0.05 kWh/m3 min to 0.15 kWh/m3 max, coolant and brine discharge by gravity (i.e. no pumping energy), an OPEC country electricity cost of US ¢3.0/KWh and costs for parts and chemicals (chlorination, etc.) of US ¢0.4–0.7/m3.

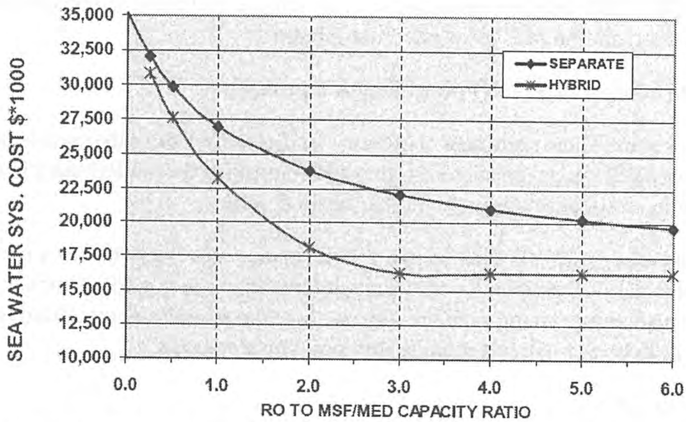


Figure 18. Investments in sea water supply and brine discharge systems for hybrid and non-hybrid 150,000 m^3/day plants

RO Membrane Life

For all membranes, production goes down with time while product salinity and chloride concentration goes up. The drop in production with time can be compensated by installing extra membrane rack space and installing additional membranes as required. The increase in product salinity cannot be compensated for except with large scale membrane replacement.

Therefore, to maintain the product water quality within WHO standards, the designer of stand alone seawater RO plants has the option to replace membranes more frequently or install a two stage (seawater RO and brackish water RO) system.

In the case of hybrid systems, a single stage system can be specified while maintaining a long membrane life. This is made possible by blending the RO product water with the high purity distilled water.

Membrane Performance as a Function of Seawater Temperature

For all membranes, water production is a function of temperature, at constant pressure. Production will go up with temperature increase by 1.5% to 3% per degree Celsius for nearly all membranes, thereby reducing the required number of RO membrane modules.

For the fully integrated hybrid process, this advantage can be utilized by operating the RO plant at optimum temperature and pressure conditions by using cooling water from the reject section of the MSF plant.

El-Sayed et al. [13] conducted pilot study of MSF/RO hybrid systems in Kuwait and observed a significant increase in RO product water flow rate. It was demonstrated on basis of experimental data that 42-48% gain in the product water flow could be achieved for a temperature of 33°C, over that of an isolated RO plant operating at 15 °C during winter season. The results imply that the energy consumption of RO can be reduced without involving any form of energy recovery, to the level of 5.2 kWhr/m^3 using a simple integration of MSF/RO hybrid arrangement in which the RO plant is fed the preheated seawater rejected from the MSF heat rejection section.

Savings due to control of SWRO plant feed temperature

The use of all or some of the preheated cooling water discharge from a thermal desalination plant as feed to a SWRO plant enables elevating and controlling the SWRO plant's operating temperature at its optimal or any other higher desired value.

Feed water temperature affects the two main performance characteristics of a membrane: flux and salt rejection. Higher feed water temperatures increase not only flux but also salt passage. They also reduce membrane life, but, as there are no definite quantitative figures relating to this effect, we will not include it in our considerations.

The main results are:

1. Hybrid plants will increase the average annual membrane flux and reduce the required membrane surface in the SWRO plants from 10.5%, when only thermal plant cooling water is used as SWRO plant feed, to 4.6%, when the ratio of the outputs of the SWRO and thermal plants is 6:1.
2. The corresponding increases in salt passage and SWRO plant product salinity will range from 4% to 9%.

The US ¢0.6/m3 membrane cost saving figure will be compounded by the savings due to the reduced investment in a range of other items of equipment related to the number of membranes in the plant. These include the membrane pressure vessels, the stainless steel high-pressure connection pipes and fittings, membrane racks, etc. Hoffman estimated the investments in these items as US $90–100/m3/day, or about 10% of total plant investment. Reducing the number of membranes by 10.5% will, therefore, reduce total water costs by about another US ¢0.6/m3, for a total of US ¢1.2/m3, or about 2%.

Savings due to blending SWRO and distillation plants' products

The blending of SWRO and thermal plants' products makes it possible to use the low-salinity (less than 20 ppm TDS) distillation plant product to compensate for inferior SWRO plant product. This is important, since, with current SWRO membranes performance (initial salt rejections of 99.6–99.8%), it is possible to obtain a better than 500 ppm TDS product in only one pass, even with high-salinity Gulf and Red Sea seawater (rather than with two passes, as required ten years ago, when membrane salt rejections were only 99.2%).However, if the plants are designed to operate at the high conversion ratios used today in most modern SWRO plants, 45–50%, product salinity will exceed 500 ppm TDS after only four years, as a result of membrane performance degradation. Hoffman calculated that at a cost of US \$650 per membrane element with a 10 m3/day design output (after three years of operation), the added replacement costs, including labor and idling of equipment costs, are equivalent to about US ¢1.5–2.0/m3.Note that, at the costs and performance applicable 5–10 years ago these added replacement costs would have been about double.

The membrane replacement cost savings due to the blending of products within a 150,000 m3/day hybrid plant, within the above range of SWRO and thermal plants' output ratios, are shown at the optimal output ratio of 2:1. The savings in membrane replacement costs in the corresponding 100,000 m3/day hybrid SWRO plant, compared with its equivalent 100,000 m3/day non-hybrid plant, are about US \$1,172,000 per year, or about US ¢3.6/m3. Figure 19 shows the effect of SWRO to thermal plant output ratios and the blending of products on initial product salinity and SWRO plant feed temperature (with new membranes).

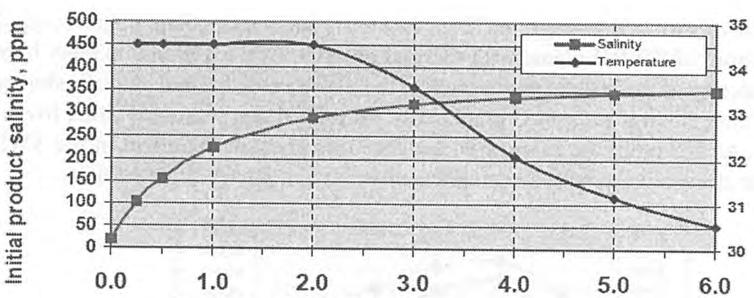


Figure 19. The effect on initial product salinity and feed temperature of the ratio of the outputs of the SWRO and thermal plants and the blending of products

Increased Recovery Ratio

Recovery ratio (conversion) is one of the key RO design parameters. It determines the size of the feedwater handling system (e.g., intake, pretreatment, high pressure pumping) for a given plant size. Higher recoveries decrease the cost of the feedwater handling system and the required electrical and chemical consumption while increasing the initial and replacement costs of the membrane system.

Some of the reasons why higher recovery ratios have not been used in the past are related to the performance characteristics of the membranes and the product water quality specifications. Due to the salt rejection property of available membranes, product water specifications (typically 500 ppm TDS and/or 250 ppm chloride) could not be easily met at higher recovery ratios. In view of the easier to achieve TDS/chloride specifications (e.g. 1000 TDS and/or 500 chlorides) for RO product water in a hybrid system, higher recovery ratios can be incorporated into the plant design while attaining the overall product water specifications.

Feedwater Deaeration

Most membranes require dechlorination of the feedwater as they are very sensitive to even very small concentrations of residual chlorine and/or bromine. This is typically done by the addition of large quantities of sodium bisulphite. As an alternative, this can also be accomplished by use of a deaerator, followed by significantly reduced quantities of sodium bisulphite. Some membranes are also sensitive to oxygen, in which case the use of a deaerator is essential.

Deaeration of the feed water also reduces corrosion significantly. In the case of hybrid systems, low pressure steam suitable to operate the deaerator is readily available from the MSF plant at low cost. Deaeration can reduce the specification for high pressure piping from SMO - 254, SS - 317L to lower grades and more economical SS 316L.

Hybrid plant cost savings – summary

Summary of the estimated savings, at OPEC fuel costs and intermediate intake costs, due to the combining of SWRO plants with thermal plants within a 150,000 m3/day hybrid plant, as a function of the ratio of the outputs of the SWRO and thermal plants is shown in Figure 20. These savings presented graphically, on annual and represent value (over 20 years) bases. At this point the savings do not take into account the effect of the SWRO plants' power consumption on the economics of water and power cogeneration.

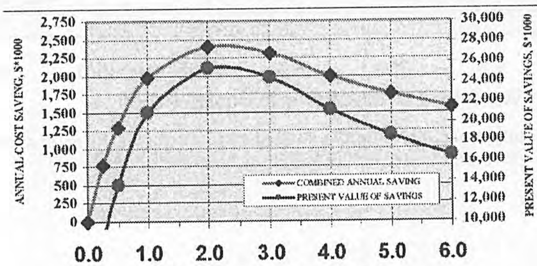


Figure 20. Annual and present-value of savings due to combining SWRO and thermal plants within a 150,000 m^3/day hybrid plant at different ratios of the outputs of the plants

Figure 21 breaks down the annual savings graph into its capital recovery, energy and treatments, and membrane cost savings components. We see that the optimal ratios of the outputs of the SWRO and thermal plants are in the range of 1.5–3.0, the peak being at a ratio of about 2:1 (100,000 m3/day SWRO and 50,000 m3/day thermal).

Hoffman et al. determined that the total savings (for the outputs of both SWRO and thermal plants) at this ratio will be about US ¢ 4.8/m3 or about US $2.4 million annually. The present value of this annual saving, at 7% interest and a 20-year depreciation period, is about US $25.3 million. To put these figures into perspective, Hoffman estimated, the costs of desalinated water from an equal-sized 100,000 m3/day non-hybrid SWRO plant, using the range of conditions and costs applicable to the MENA countries The absolute and relative magnitudes of the savings derived from the hybrid plant scheme, if they are all assigned to the 100,000 m3/day hybrid SWRO plant's product costs only (i.e. none to the MSF plant's product costs), compared to the costs of our estimated non-hybrid SWRO plant desalinated water shown that the hybrid plant scheme will reduce non-hybrid SWRO plant water costs by as much as US ¢6–9/m3, or 9–16%.

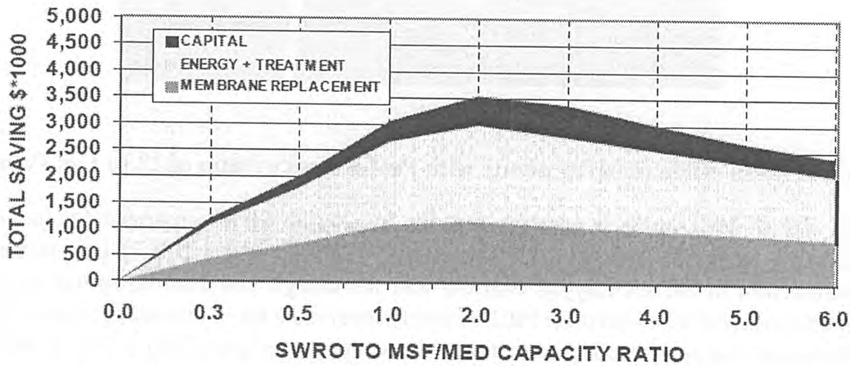


Figure 21. Breakdown of annual savings due to combining SWRO and thermal plants within a 150,000 m^3/day hybrid plant at different ratios of the outputs of the plants

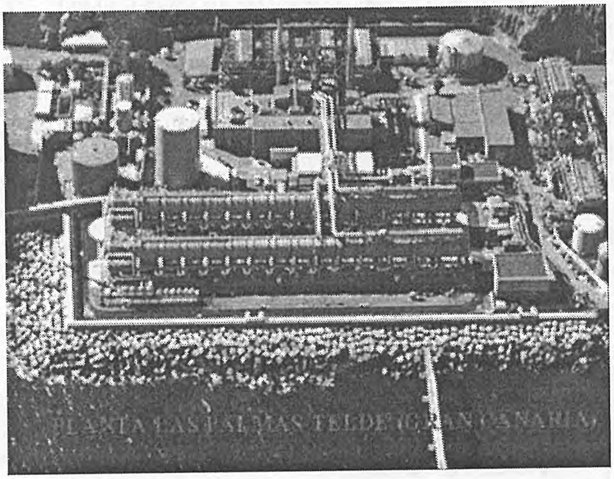
HYBRID VARIATIONS

As the concepts and applications of hybridization are accepted between distillation processes and RO, we believe that membrane manufactures will develop a new generation of membranes. This new generation of membrane [20] is characterized by a very high specific flux about double the flux of the current generation with small reduction in salt rejection. The current high flux membranes, developed for brackish water desalting demonstrated the ability to significantly reduce the cost of desalting and will be ideal for hybrid with distillation plants.

HYBRID SYSTEM USING MULTI-EFFECT DISTILLATION

Multi-effect distillation (MED) is in our opinion the most important large-scale evaporative process offering significant potential for water cost reduction.

The major potential advantage of MED process is the ability to produce significantly higher Performance Ratio (PR) in excess of 15 pounds of the product per pound of steam where MSF practical limits PR to 10.



One of more efficient MED plants with Performance Ratio of 12 in Las Palmas

The size of MED units is growing rapidly. In Sharjah SEWA operated for last two years the largest commercial MED units of 5 MIGD and 8 MIGD unit is under construction in SEWA Layyah Station, and the design and demonstration module already exist for 10-migd unit. MED recently received a lot of attention, as a result of numerous commercial successes of Thermocompression like MED for Al Taweelah A1 a 53 MIGD capacity plant.

In general MED capital cost today varies from US$ 4.50 to US$ 6.00 per IGPD capacity. The future calls for increasing top operating temperature, finding new ways to improve heat transfer performance to reduce heat exchange area to search for an increase in heat transfer performance by tube enhancement, and use of very thin wall in tubular materials. The critical challenge is to adopt Nanofiltration as means to dramatically increase output and increase efficiency

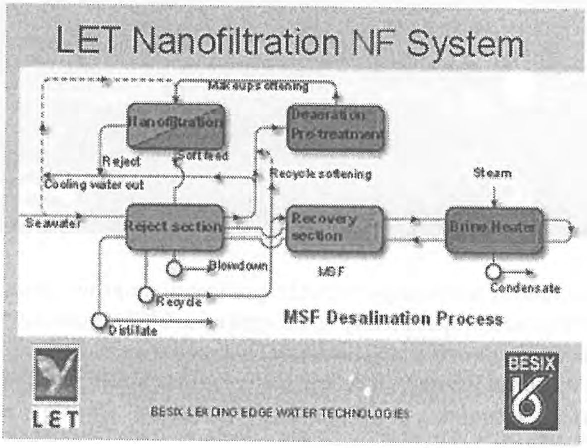
HYBRID USING NANOFILTRATION - MEMBRANE SOFTENING

Membrane softening technology adapted to hybrid with distillation processes could lead to significant increase in productivity of existing of future distillation plants as well as obviously cost reduction. Similar to reverse osmosis, nanofiltration (NF) is based on solution-diffusion as major transport mechanism; however, nanofiltration membranes contained fixed (negatively) charged functional groups. As a result, the selecting of NF for monovalent and bivalent anions is significantly different.

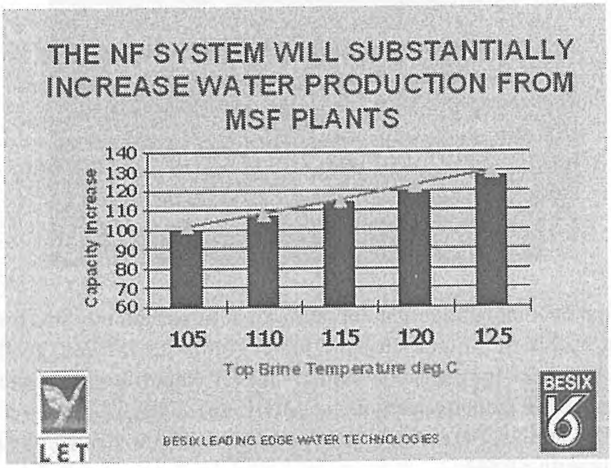
The development of low-cost nanofiltration membranes that will remove, economically, the scaling salts from the MSF and SWRO plant feeds, has been advocated by Awerbuch [11] and the Saudi Arabian Saline Water Conversion Corporation's (SWCC) R&D Center [32–34].

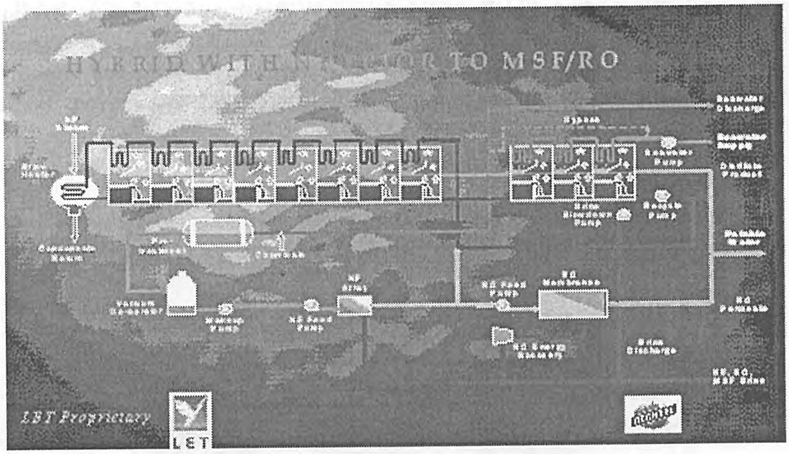
The great potential of Nanofiltration membrane softening technology was brought to focus by recent award by Sharjah Electricity and Water Authority (SEWA) to Besix Leading Edge Water Technologies for the first commercial LET Nanofiltration System to increase capacity of existing MSF plant from nominal 5 MIGD to 7.2 MIGD. This over 40% increase in capacity of MSF unit was a result of a two year demonstration and simulation program developed jointly with SEWA.

The additional capacity is achieved without building new intake structure or new power plant in a very limited space which would not allow construction of new desalination plant. The paper will present the results of the testing and basic feature of the system. The system involves construction of NF plant to provide partial membrane softening of feed to MSF as well as modifications to existing MSF plant to be capable to achieve the increased capacity.



The concentration of sulfate and calcium ions determines in the distillation process the top temperature and concentration factor. Even partial elimination of calcium and sulfate from the feed will dramatically improve the performance of distillation plants. By increasing top temperature from current 95-110°C to 120-125°C would increase water production from existing MSF plants by 25% to 45%. The partial removal of sulfate and calcium ions from the feed has a multiplying positive effect on reduction of scale potential. With the current high quality materials of construction the negative corrosion effects of higher temperature would be minimal.

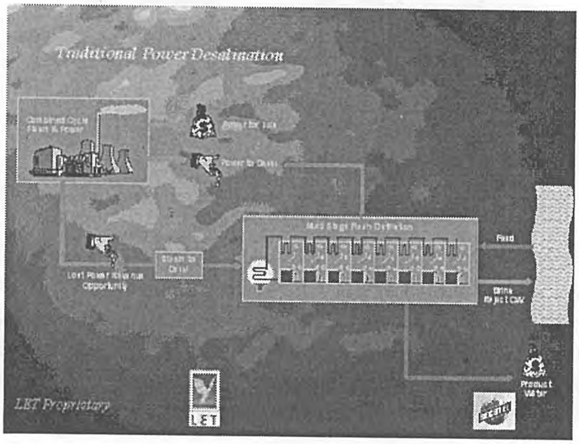




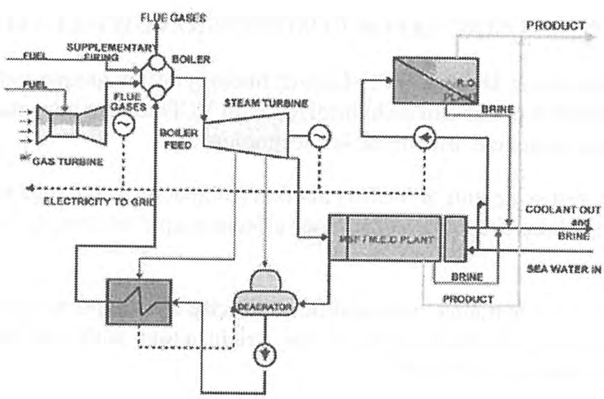
N F – membrane softening technology – could significantly improve operation and reduce the cost of the MED process, specifically when applied to MED processes using advanced heat transfer surfaces like double fluted tubes, by eliminating the risk of scaling and fouling. NF technology will permit increase in the top temperature resulting in significant increase in output and performance ratio.

The latest references to use of Nanofiltration to hybrid system can be found in papers presented by Awerbuch [47]

DUAL PURPOSE FACILITIES AND POTENTIAL FOR HYBRIDIZATION

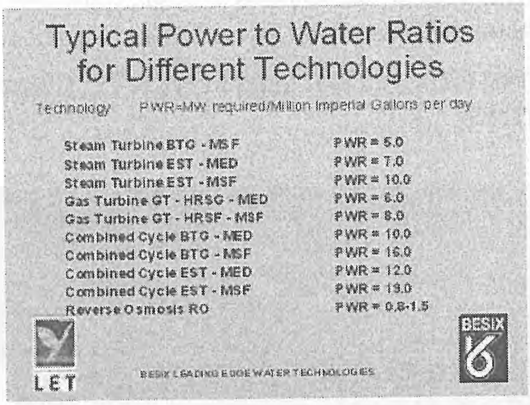


The importance of power to water ratio and potential for hybridization becomes more relevant, after sorting out the comparative advantages of currently available power and water technologies. In this presentation we will omit nuclear options for power generation.



Given that the water and power requirements of real utility varies seasonally by a significant amount, the optimal choice for power desalting plant is not obvious when operation extends beyond base load.

In order to simplify the initial selection the following are typical MW power required to be installed, in order to produce 1 million imperial gallons per day of desalinated water defined the ratio as PWR.



It is interesting to note, that the more efficient is the base load operation for generating electricity, the less effective is production of water and power in peaking and intermediate modes. The most advance combined cycle desalination plant has very high PWR, choice of which would provide a significant surplus of unused power capacity in winter time. An excellent analysis of the optimal power and water plants design and demand analysis was reported by Lennox et al. [23], Sommariva et al.[41]

The above data show that electrical driven desalination processes like RO and VCD, clearly requires minimum investment in the power plant. At the same time where seasonal and daily variations occur electrically driven technology can provide an excellent choice to be hybridized with more conventional dual purpose plants. The hybrid approach could achieve the lowest cost of total investment, flexibility in production and the lowest cost of power and water production.

HYBRID SYSTEMS USING VAPOR COMPRESSION DISTILLATION

The Vapor Compression Distillation (VCD) technology offers unique potential. Today power/MSF/MED/RO plants can be hybridized with VCD to take advantage of increase distillation output, using electrically driven technology.

Currently the largest scale unit of VCD is 3000 m^3/d capacity or 0.8 mgd in a single unit, which consists of three evaporator-condenser effects couple to a single high volumetric compressor.

This large scale VCD guarantee unit specific electricity consumption of 7.5-8.5 kWh/m^3 of product. (excluding sea water supply). They produce high purity 10-20 ppm distillate at high plant availability's of 94-96%.

In future vapor compression distillation units, will grow in capacity and number of effects. A single - effect 2.5 impd (11,000 ^{t}/day) vapor compression system with a conventional axial flow compressor and with an unconventional radial inflow compressor of a novel design was described by Yehia M. El-Sayed [14].

Design of VCD over four and more effects, staging compressor in series or parallel will allow effective hybridization of power with MED, MSF and Reverse Osmosis. This particularly will be important in cases where power to water ratio has to be minimized in favor of water production.

HYBRID SYSTEMS USING MSF-MED

In distillation processes there is no interaction between MSF and MED energy process streams. Substantial efficiency improvements could be obtained if process streams between MSF and MED are exchanged in order to take advantage of the different operating temperature conditions of each plant.

In particular due to the low MED operating temperature (61-67° C) this process could be thermally driven by process streams properly sourced by an adjacent MSF plant.

A number of novel technology options (LET –Mott McDonald patent pending) that have been studied and their possible implementation in a real scale plant should be available soon. The process was recently described by Sommariva et al [49] at IDA World Congress in Singapore.

The objective of the MSF-MED hybrid is to increase energy efficiency, distillate production and minimize operational costs.

HYBRID SYSTEMS AND AQUIFER STORAGE AND RECOVERY (ASR)

Cost-effective integration of three proven technologies, desalination, power and aquifer storage recovery (ASR) can secure a reliable, sustainable and high-quality fresh water supply for the Gulf States. LET pioneered in the Middle East the concept of strategic and economic storage and recovery of desalinated water (DASR) and waste water (WASR) to the security of its communities. The idea is covered in many papers [40, 42-45]

The seasonal surplus of unused idle power could be used by electrically driven desalination technologies RO and Hybrid Systems including NF/RO/ MSF process in combination with ASR creating a system of Desalination/ Aquifer Storage and Recovery (DASR). The ability to store and recover large volumes of water can contribute to the average downsizing of power and water facilities with substantial operational cost savings.

DASR provides strategic reserves of potable water, to prevent damage or depletion to existing oasis or aquifers, for controlling salt-water intrusion, or improvement in water quality .DASR is of strategic importance to the Middle East

DASR - Creating the Additional Water

- Electricity demand drops to 30-40% of peak during the winter months
- As a result, over 50% of power generation capacity of power-desalination plants is idle
- This idle power can be used to produce low-cost water (above normal demand) using nano-filtration and other membrane desalination technologies

LET

Schlumberger

Desalination plant will operate continuously with modulating its output depending on power demand. Typical water storage volumes for desalinated water are limited to providing less than one day of water supply, a highly vulnerable situation.

Aquifer Storage and Recovery, a unique technology for temporarily storing water in natural aquifer systems, can be effectively used to provide the additional capacity needed to store the surplus desalinated water.

One solution which should be given serious consideration is Aquifer Storage and Recovery (ASR). This technique involves storing desalinated water in existing aquifers, ready for recovery when water is in short supply or needed. It is already used at more than 30 locations in the US. In ASR, we would inject the surplus desalinated water into the aquifers, through suitable storage in semi-confined or confined zones.

Resources Conservation and Environmental Impacts of Various Hybrid Configurations

Resources conservation and environmental impact, too, are aspects that have to be considered when designing hybrid systems The reduced primary (fuel) energy consumption when coupling reverse osmosis with thermal processes in a hybrid configuration amounts to from 30% up to 40% against reference plants (boiler for heat and condensing turbine power plant for electricity). CO_2 emissions from a gas-fired combined cycle plant with a corresponding SWRO share are likewise substantially lower than for a conventional cycle, that is a condensing turbine power plant alone with SWRO for water production. There is a rise in heat dissipated to the atmosphere, but considering environmental impacts this is offset by less pollution emitted in the flue gases from a gas-fired power plant and the consequently reduced need for flue gas cleaning. Showing a substantial reduction is dissipation of heat from a hybrid plant to the sea as compared with the conventional heat cycle/SWRO configuration. In recent years the consideration of carbon dioxide tax will have a significant impact in justifying hybrid plants in the Gulf.

CONCLUSIONS

Combining thermal and membrane desalination processes and technologies within a single plant or in hybrid plant schemes can reduce desalinated water costs, and, as part of dual-purpose stations; add flexibility to the combined water and power production and reduce any existing water and power demand mismatch problems

It can be seen that applying hybrid solutions will reduce desalinated water costs, compared with non-hybrid schemes, from as little as 2–3% to as much 15%,

In large desalination plants, there should also be little loss of economies of scale due to the use of two or more different processes, in two or more smaller units, in lieu of one large, single-technology plant. Many such plants, at the same site, are based on the same process (MSF), but utilize different designs and have different performance figures. All the solutions whether stand-alone high-GOR plants (LT-MED/TVC, HT-MED) or hybrid schemes (MSF/SWRO, MVC/MED, MVC/TVC, etc.) requires use of the largest size plants available.

The Hybrid of power-desalination systems, from its early concept of power - MSF-RO to blend the products and minimize power generation, leads to many new ideas.

- Hybrid of MED - RO has many of the same advantages than the MSF - RO, but has the ability to cut significantly power water (PWR) ratio.
- Hybrid of MSF - MED with VCD has the potential of boosting water output through simple or full integration and at the same time reduce power to water (PWR) ratio.
- Hybrid with Nanofiltration - Softening Membrane will provide the ability to increase desalination output of distillation plants MSF and MED, by reducing scaling potential of the feed, increase the top brine temperature and provide significant better concentration factors and recovery for all distillation processes.
- Hybrid with electrically driven desalination technologies RO and VCD would allow use off peak power for water production, and minimize power capacity by shutting down RO or VCD daily during the peak.
- The seasonal surplus of unused idle power could be used by electrically driven desalination technologies RO and VCR in combination with acquifer storage and recovery to create effective DASR solutions.

All of the above ideas have a goal to maximize and optimize benefits of power and water generation in order to provide lower cost water the "Essence of Life".

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Recarbonational Strategy is an Innovative Technology in Kuwait

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RECARBONATIONAL STRATEGY IS AN INNOVATIVE TECHNOLOGY IN KUWAIT

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ABSTRACT

The production of drinking water from sea water is a very common practice in all areas of the world where fresh water is not available. The most suitable technology adopted in Kuwait is based on the Multi Flash System (MSF). Distilled water produced from MSF plants in Kuwait is a very soft water that has low buffer capacity, which is considered quite aggressive to the materials utilized in the water distribution system. Moreover, where there is a lack of fresh water, MSF proves to be an interesting technological achievement in the production of water for human consumption. There is however, a new problem paradoxically the opposite of what normally takes place in MSF plants involving the replacement of lost salinity to a level that is required and acceptable for humans. Therefore, a certain degree of hardness and alkalinity is required to make water pleasant to the palette and thirst-quenching. In Kuwait, the recarbonation process has been adopted as the remedial treatment. The recarbonation system is intended to increase the carbonate hardness in the potable water thereby minimizing corrosion in the water distribution system, thus eliminating to a great extent the "Red Water" problem. It also improves the palatability of the drinking water. To achieve these objectives the recarbonation system makes use of recovered carbon dioxide from the MSF Evaporators and adopts Limestone Dissolution with necessary pH control. This is regarded as reasonably effective and an economical solution as well.

Key words: MSF, Recarbonation, Red water, Hardness, Alkalinity, Lime stone.

1. INTRODUCTION

The process of removing hardness from water is called softening. Hardness is mainly caused by the presence of calcium and magnesium salts. These substances are dissolved from deposits through which the water percolates. The length of time that water is in contact with the hardness-producing material is one factor that determines how much hardness there is in the raw water. To soften water by the lime or soda-ash method, its degree of alkalinity has to be considered.

The alkalinity of a water sample is a measure of the water's capacity to neutralize acids. In natural and treated waters, alkalinity is the result of the presence of bicarbonates, carbonates, and hydroxides of calcium, magnesium, and sodium. Many of the chemicals used in water treatment, such as alum, chlorine, or lime, cause changes in alkalinity. Determining alkalinity is useful when calculating chemical dosages for the coagulation and water-softening processes. The alkalinity must also be known when calculating corrosivity of the water and to estimate its carbonate hardness. Alkalinity is usually expressed in terms of calcium carbonate.

Depending on the amount of lime or chemicals needed to reduce the amount of calcium or magnesium in the water, the treated water will generally have a pH greater than 10. It is necessary to lower the pH to stabilize the water and prevent the deposition of hard carbonate scale on filter sand and distribution piping. Recarbonation is the most common process used to reduce the pH. This procedure involves the addition of carbon dioxide to the water after the softening. Generally, enough carbon dioxide is added to reduce the pH of the water to less than 8.7. The actual amount of carbon dioxide to add must be determined by using a saturation index of some kind. The Langelier Index (LI) is by far the most common stabilization index used, but some plants instead use the Rizner Index, the reciprocal of the Langelier Index. The Langelier Index is expressed as the pH of stabilization (pHs) minus the actual pH measured (pHs-pH). When the Langelier Index is positive, the water will tend to coat the pipes, when it is negative, the water tends to be corrosive.

2. Red water problem in Kuwait

The history of the Multi Flash System (MSF) technology in Kuwait as well as the other Gulf Cooperation Council (GCC) countries reveals no cultural or social concern whatsoever on the part of the public regarding the extent which MSF product water is used. The fact that the process involves heating to high temperatures and results in distilled product water made this water fully acceptable without reservation. However, incomplete stabilization of the product water, to some degree in the past, as well as improper selection of piping network materials have resulted in significant corrosion residues being carried over into the domestic water supply. This is a problem known as the red water problem in Kuwait, and it requires extensive use of simple domestic filtration.

The distilled water produced by the MSF plants has very low concentrations of dissolved salts and gases, and a total alkalinity of less than 1 mg $CaCO_3$/l. The high purity renders the water chemically very aggressive towards almost all components in the water distribution system, resulting in very severe corrosion problems. One of the by-products of this chemical attack is ferric hydroxide, red-brown rust that results in what is known as "red water".

To improve the palatability of distilled water, it is usually blended with brackish water using different blending ratios, depending on the quality of the available brackish water. The blending ratio used in Kuwait is about 10 (distillate): 1 (brackish). The potable water being produced in Kuwait has a total alkalinity of 20-24 mg $CaCO_3$/l, which is less than the minimum value of 50mg $CaCO_3$/l necessary to depress the corrosion rate of the different materials involved in the water distribution system. It is, therefore, important to increase the pH and depress iron dissolution, and thus, prevent complete failure of pipes in the potable water distribution system.

During the period 1979-1981, the Water Resources Development Center (WRDC) of the MEW conducted a comprehensive study in order to find a solution to the red water problem. One of the main outcomes of this study was the recommendation to use ductile iron pipes lined internally with cement-mortar and seal-coated. Most of the water distribution system has been upgraded to this material of construction. Another main outcome of this study was that water recarbonation was best performed using bicarbonates. It was also recommended that phosphate inhibitors be used with a higher initial dose of 6-12mg/l of phosphate followed by a lower 2-6mg/l dosage.

The first recarbonation process was established in 1987 and incorporated into the Shuwaik MSF plant. The capacity of the recarbonation plant is 18MIGD and was designed to yield recarbonated water with total alkalinity of 60-80mg $CaCO_3$/l. Based on the experience gained and the success of the recarbonation process at the Shuwaik MSF plant, the MEW also adopted the recarbonation process in the MSF plants located at Doha West (90MIGD) and Az-Zour South (40MIGD), Fig. (1).

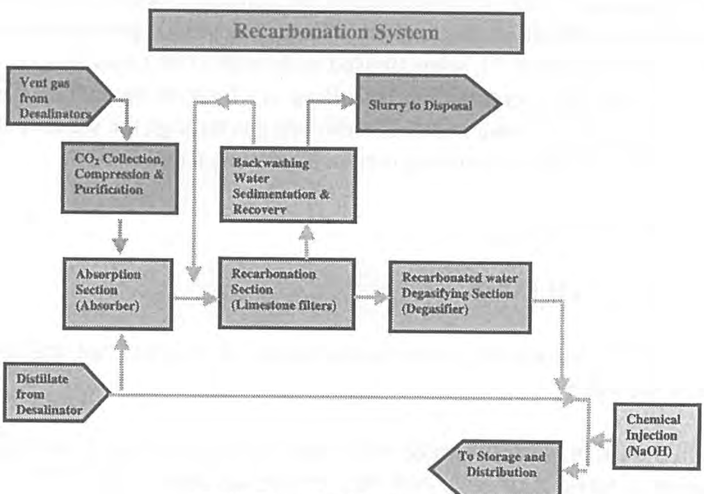


Fig (1): General Flow Diagram

3. Treating Water Hardness

The process of removing Ca^{2+} and Mg^{2+} from the water is known as water softening. Two minerals, lime ($Ca(OH)_2$) and soda ash (Na_2CO_3), are typically used to soften public water supplies.

When lime is added to water, it dissolves to give three aqueous (solvated) ions: one Ca^{2+} ion and two OH^- ions for each unit of $Ca(OH)_2$. Likewise, soda ash dissolves to give two Na^+ ions and one CO_3^{2-} ion for each unit of Na_2CO_3 that dissolves.

A number of reactions occur to generate the insoluble precipitates $CaCO_{3(s)}$ and $Mg(OH)_{2(s)}$ from the Ca^{2+} and Mg^{2+} ions. The most important reaction for the removal of Mg^{2+} is shown in Equation 1.

$$\underset{\text{From water}}{Mg^{2+}_{(aq)} + Ca^{2+}_{(aq)}} + \underset{\text{From lime}}{2OH^-_{(aq)}} \rightarrow \underset{\text{Precipitate}}{Mg(OH)_{2(s)}} + Ca^{2+} \qquad (1)$$

Notice that Ca^{2+} appears on both sides of Equation 1. The calcium ion from lime does not actually participate in the reaction to generate insoluble $Mg(OH)_2$. Hence, this ion is called a spectator ion and can be omitted from the equation. We can write the reaction more correctly with the net ionic equation, given by Equation 2.

$$Mg^{2+}_{(aq)} + 2OH^-_{(aq)} \rightarrow Mg(OH)_{2(s)} \qquad (2)$$

The important reaction for the removal of Ca^{2+} ions is given in Equation 3.

$$\underset{\text{From water}}{Ca^{2+}_{(aq)}} + \underset{\text{From lime}}{Ca^{2+}_{(aq)}} + \underset{\text{From soda ash}}{2CO_3^{2-}{}_{(aq)}} \rightarrow \underset{\text{Precipitate}}{2CaCO_{3(s)}} \qquad (3)$$

The solids generated by the water-softening precipitation reaction are then removed by sedimentation or filtration. If an excess of lime was used to precipitate magnesium ions in the water (Equation 1), some unused hydroxide (OH^-) ions will remain in the water after the calcium is precipitated, resulting in a high (or basic) pH. If necessary, the pH can be lowered by bubbling carbon dioxide gas through the water. The net ionic equations for this recarbonation are given in Equations 4 and 5.

$$CO_{2(g)} + H_2O_{(l)} \rightarrow H_2CO_{3(aq)} \qquad (4)$$

$$H_2CO_{3(aq)} + OH^-_{(aq)} \rightarrow H_2O + HCO3^-_{(aq)} \qquad (5)$$

Bicarbonate (HCO_3^-) remaining in the water is nontoxic and does not negatively affect the flavor of the water.

If iron is immersed in water saturated with dissolved oxygen, rust is developed due to the formation of ferric hydroxide $Fe(OH)_3$ or hydrated ferric oxide $Fe_2O_3.xH_2O$. The mechanism of this electrochemical reaction can be explained by the following equations:

$$Fe \rightarrow Fe^{2+} + 2e^- \quad (6)$$

$$2e^- + 2H_2O \rightarrow H_2 + 2OH^- \quad (7)$$

$$2e^- + H_2O + ½O_2 \rightarrow 2OH^- \quad (8)$$

Equation (6) represents the anodic reaction which causes ferrous ions and electrons to be released from iron atoms. The electrons released will be consumed according to the cathodic reactions represented by equations (7) and (8). In acidic solutions such as water saturated with CO_2, hydrogen gas is generated and hydroxyl ions are formed according to equation (7). In neutral solutions containing dissolved oxygen, the same amount of hydroxyl ions will be formed according to equation (8), and alkaline conditions will prevail. The ferrous ions are unstable under normal conditions and will be oxidized due to the presence of dissolved oxygen in neutral solutions according to the following reaction:

$$2Fe^{2+} + H_2O + ½O_2 \rightarrow 2Fe^{3+} + 2OH^- \quad (9)$$

The ferric ions are then hydrolyzed by water precipitating the insoluble ferric hydroxide (iron rust) according to:

$$Fe^{3+} + 3H_2O \rightarrow Fe(OH)_3 + 3H^+ \quad (10)$$

The overall reaction is the formation of hydrogen ions, which maintain a condition of acidity according to:

$$2Fe^{3+} + 5H_2O + ½O_2 \rightarrow 2Fe(OH)_3 + 4H^+ \quad (11)$$

Ferric hydroxide is a gelatinous precipitate which partially dehydrates resulting in the red-brown ferric oxide, which is the main constituent of "red water". It has no protective action and as water temperature increases, the pH decreases corresponding and the corrosion rate will therefore increase. It is therefore important to increase the pH and depress iron dissolution and thus prevent complete failure of the pipes in the potable water distribution system.

4. The Best Formula of Water

The optimum chemical balance of drinking water is a very complicated matter in both physiological and organoleptic terms. For this reason reference is normally made to the WHO Guidelines in the production of water with the best "formula". A certain degree of hardness and alkalinity are required to make water pleasant to the palette and thirst-quenching.

Water of high alkalinity and calcium content is stable water, and can produce a thin protective layer of calcium carbonate by careful increase in the pH. This is the concept of self-inhibition and can be described by the following reactions:

$$HCO^-_3 + OH^- \rightarrow CO^{2-}_3 + H_2O \quad (12)$$

$$CO^{2-}_3 + Ca^{2+} \rightarrow CaCO_3(S) \quad (13)$$

$$CaCO_3 + H_2CO_3 \rightarrow Ca\,(HCO_3)_2 \quad (14)$$

The hydroxyl ions necessary for reaction are produced during the reduction of dissolved oxygen, according to equation (8), Fig. (2).

The stiochiometry of the reactions indicate that for every mole of carbon dioxide, there is a mole of calcium carbonate, namely 1 ppm of converted CO_2 which gives origin to an increase in hardness as $CaCO_3$.

There are three conditions which are necessary for self-inhibition of water:

(1) The water must be free of CO_2. Carbonic acid, even at low concentrations, will neutralize the hydroxyl ions produced according to equation (8). It will also dissolve any protective calcium carbonate layer, and finally it will attack the metal. Free CO_2 can be degasified in a stripping tower using air, or by treatment with sodium hydroxide.
(2) The concentration of calcium and carbonate ions must satisfy the solubility product of calcium carbonate, in order to precipitate the inhibition film.
(3) The pH value of water must be carefully adjusted. The Langelier saturation index, LSI, is defined by the following equation:

$$LSI = pH - pHs \quad (15)$$

Where the pH of saturation, pHs, is defined as the pH value at which water containing bicarbonate and calcium is just saturated with $CaCO_3$. The pHs can be expressed as:

$$pHs = A + B - \log\,(Ca^{2+}) - \log\,(\text{total alkalinity}) \quad (16)$$

Where A is a constant which depends on the water temperature and B is another constant which depends on the water TDS content. Calcium ion concentration and total alkalinity are both expressed as mg $CaCO_3$/l.

A positive saturation index is associated with non-corrosive conditions, and thus indicates a tendency to deposit $CaCO_3$. A negative index indicates a tendency to dissolve $CaCO_3$ and is therefore associated with corrosive conditions. It can then be deduced that maintaining the pH value of water above its pHs value will result in deposition of the protective $CaCO_3$ layer and hence corrosion inhibition can be achieved.

Based on the recommendation of maintaining the same level of calcium ion concentration and total alkalinity, the following equation can be applied for a recarbonation process, operating at 40°C, a total alkalinity of 70 mg $CaCO_3$/l and calcium concentration of 70 mg $CaCO_3$/l.

$$pHs = 11.46 - 2 * \log(\text{total alkalinity}) \quad (17)$$

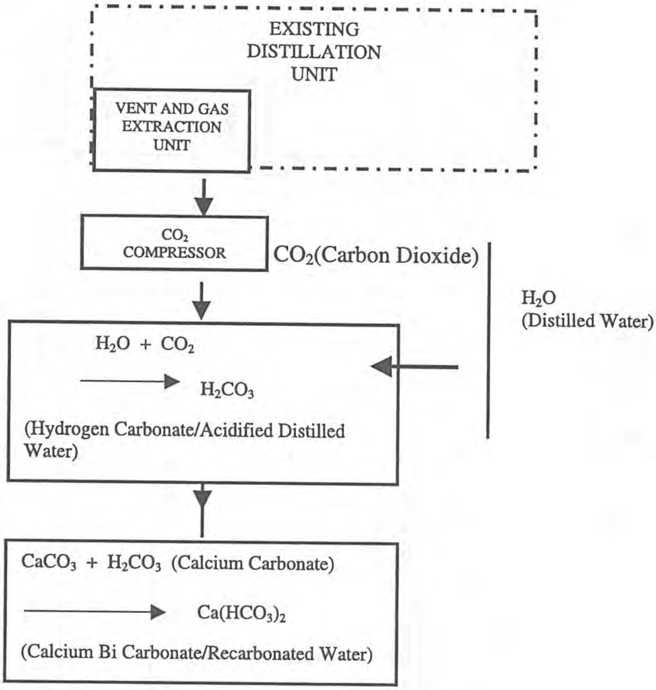


Fig. (2): Basic Process Block Diagram

5. The Recarbonation Process:

In Kuwait, a recarbonation process has been in operation at Shuwaikh Desalination Plant treating 18 MIGD of distilled water as the first step in a major scheme aimed at treating water produced by all MSF plants in Kuwait.

The major steps in the adopted recarbonation process include extraction of carbon dioxide from the vent gas system of the MSF plant, compression and purification of CO_2 – air gas stream, acidification of a pre-calculated distilled water stream in a gas absorption tower, limestone dissolution to augment the water with Ca^{2+} and HCO^-_3 necessary for water to be self-inhibiting, degasification of the excess CO_2, blending with by-passed distilled water and finally pH adjustment, Fig. (3).

Table 1 shows the chemical analysis of potable water produced in Kuwait before recarbonation. Table 2 is the chemical analysis of recarbonated distilled water produced in the distillation plant and the chemical analysis of distributed fresh water produced by blending the recarbonated distilled water with 5% brackish water.

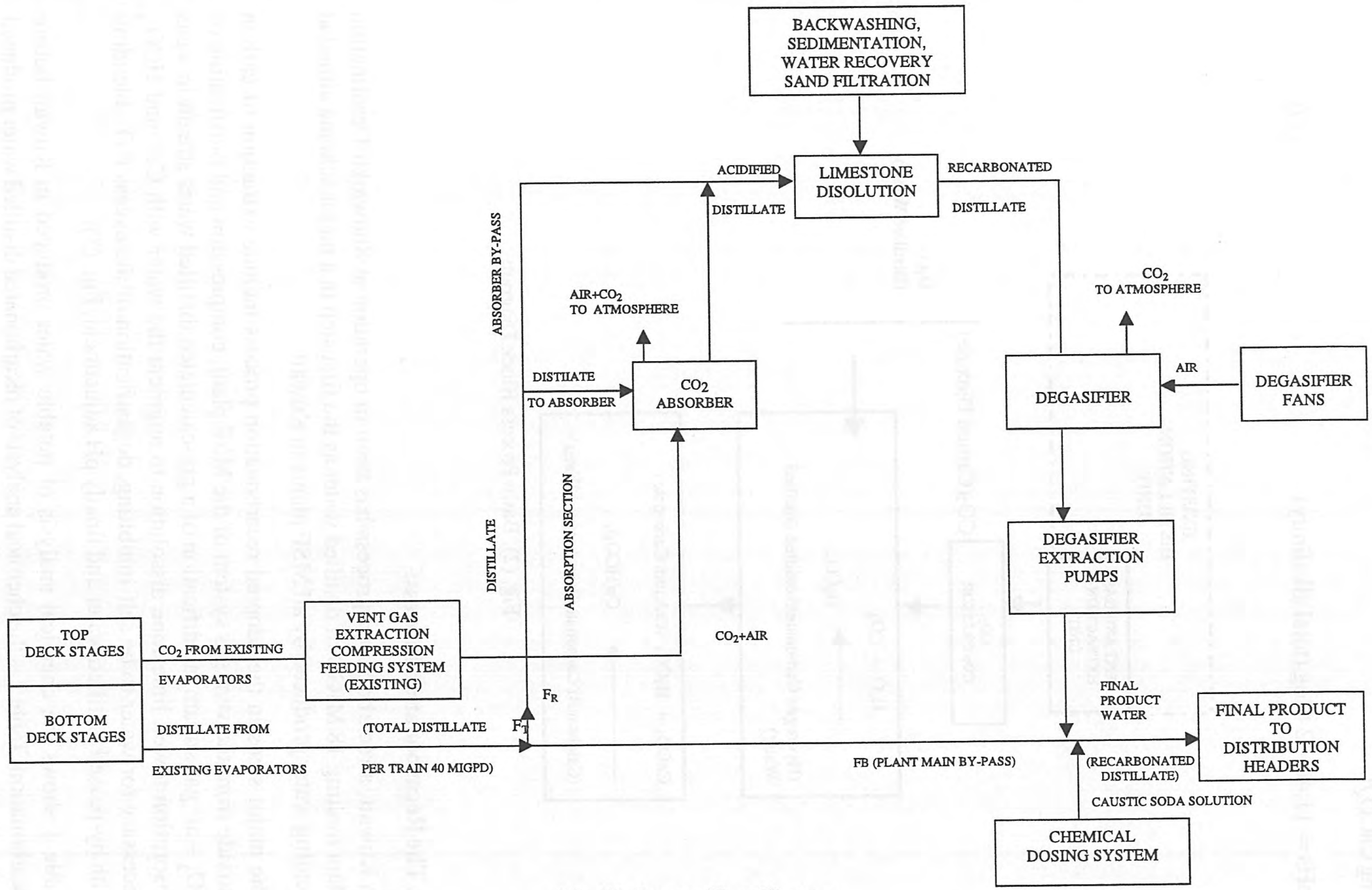


Fig. (3): Process Flow Diagram

Table 1: Average Chemical Analysis of Potable Water Produced from Shuwaikh Blending Complex

Parameters	Concentration mg/l
Total Dissolved Solids (TDS)	241
pH	7.8
Total Alkalinity (as $CaCO_3$)	11.5
Total Hardness (as $CaCO_3$)	96
HCO_3^-	13
SO_4^{--}	80
Cl^-	56
Ca^{++}	23
Mg^{++}	9.6
Na^+	39
K^+	1.2

Table 2: Average Chemical analysis of recarbonated distillate and recarbonated fresh water

Parameters (mg/l)	Recarbonated Dist.	Recarbonated Fresh
TDS	83	261
pH	8.05	7.86
Total Alkalinity	72	85
Cl^-	>0.2	40
SO_4^{--}	>0.5	66
Ca^{++}	28	38

Recarbonation plant at Doha West Power Station is divided into the following, Fig (3):

5.1. Vent Gas (CO_2) collection, CO_2 Compression and Feeding System.

5.2. CO_2 purification and absorption system.

5.3. Limestone dissolution system.

5.4. Backwashing and backwash water recovery systems.

5.5. Degasifying & Chemical Dosing System.

The Doha Recarbonation System is divided into two trains (Train A and Train B) having identical characteristics. Train A normally treats (45 MIGPD) distillate from a first set of eight evaporators (D1A to D4B) and Train B which treats (45 MIGPD) distillate from the second set of the remaining eight evaporators (D5A to D8B), Fig. (4), Table 3.

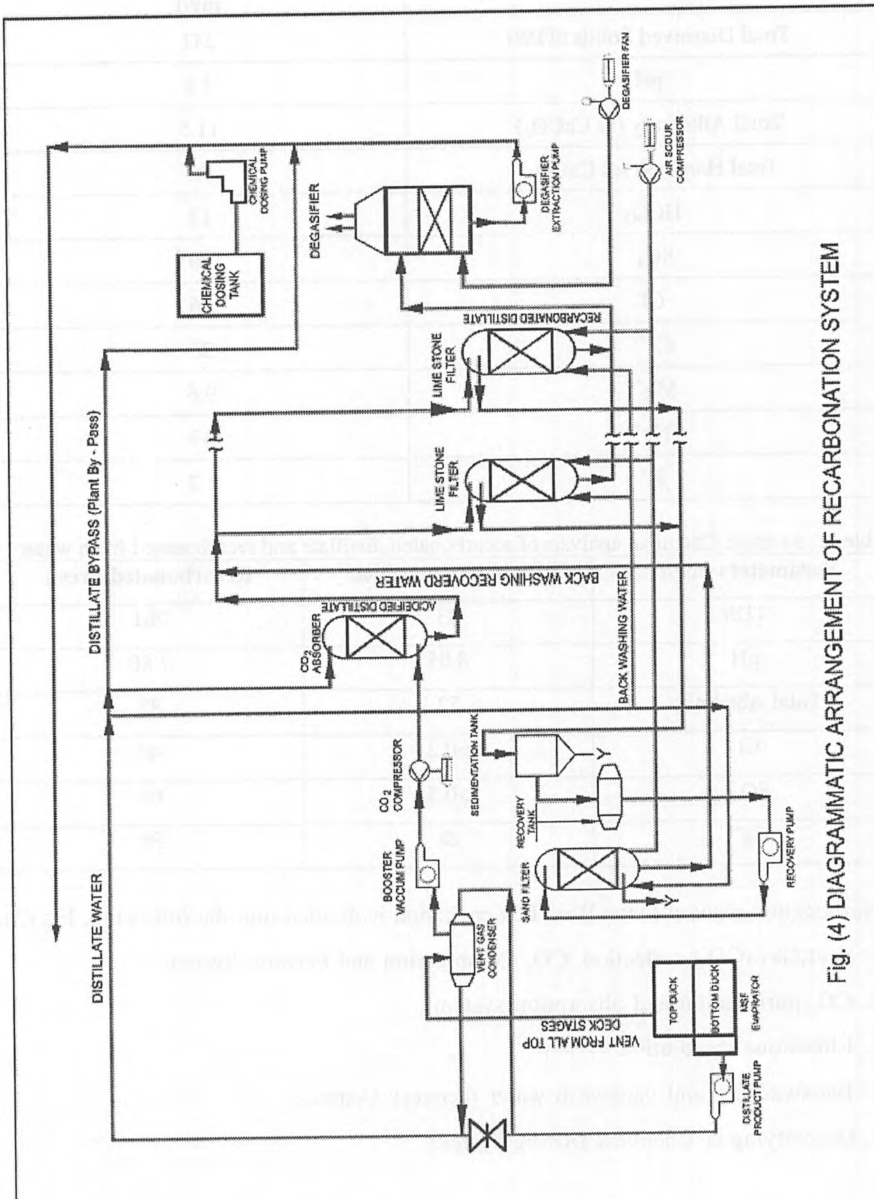


Fig. (4) DIAGRAMMATIC ARRANGEMENT OF RECARBONATION SYSTEM

Table (3): Major Equipments of Doha Recarbonation System

Major Equipments per Train		Total Number	Type
CO_2 Gas Compressor	3	6 (4 duty, 2 standby)	Reciprocating, 2-stage, Double action compressor
Snubber	2	Four	Vertical, Cylindrical with ellipsoidal head
CO_2 Gas Receiver	2	Four	Vertical, Cylindrical with ellipsoidal head
CO_2 Gas Absorber	2	4 (2 duty, 2 standby)	Vertical, Cylindrical Packed Tower
Gas Purification Filter	2	18 (14 duty, 4 standby)	Vertical, Cylindrical with ellipsoidal Head and Basket
Limestone Dissolution Filter	9	7 duty, 2 standby per train	Vertical, Cylindrical with Ellipsoidal Heads acidified distillate distributor and collection pipes
Degasifier	1	2	Vertical Cylindrical packed Tower with fan
Degasifier Extraction Pump	2	4 (2 duty, 2 standby)	Volute Type
Degasifier Fan	2	4 (2 duty, 2 standby)	Blower
Caustic Soda Pump	2	4 (2 duty, 2 standby)	Plunger type, reciprocating
Caustic Soda Tank	2	4 (2 duty, 2 standby)	Vertical, Cylindrical atmospheric tank
Sedimentation Tank	1	2	Vertical, Cylindrical with cone bottom

5.1.1. Vent Gas Collection

Necessary CO_2 to recarbonate the distillate produced is recovered from the same evaporators. The mixture of CO_2, vapour and other gases (mainly oxygen and nitrogen) is collected from the vent pipes of all top stages of evaporators and passed to the Vent Gas Condensers and then to the liquid ring Booster Vacuum Pumps where the pressure will be increased to the required level for the suction side of the CO_2 compressor.

The Cooling Water for the condenser and sealing ring of Booster Pumps will be the same distillate produced from the respective evaporators.

From the booster vacuum pump the vent gas mixture (together with sealing ring water) will be discharged to the drain separator where gas and water will be separated. The arrangement of the drain separator is able to recover all the CO_2 entering the gathering section except the quantity dissolved in the drain water of the vent gas condenser. The CO_2 gas thus recovered is then sent to the snubber at the suction side of the main CO_2 compressor.

5.1.2. CO_2 Compressions & Feeding System

The CO_2 compression and feeding system mainly consists of CO_2 inlet header, snubbers which act also as pulsation dampers, CO_2 compressors, CO_2 receivers and header to activated carbon filter.

The CO_2 gas recovered from each evaporator by the vacuum pump is collected in the snubbers and is passed to the suction side of the CO_2 main compressors. There are three main compressors per train; two on duty while the third one is on standby. The CO_2 compressors increase the pressure to about 8 bars in order to pass the mixture of gases to the CO_2 receivers and then to the headers leading to activated carbon filters.

5.2.1. CO_2 Gas Purification

In order to ensure the carbon dioxide used in the recarbonation system is food grade type, the compressed gas will be passed through an activated carbon filter where volatile organic compounds, odor etc., if any, are eliminated.

Duplicate activated carbon filters for each train one in use and the other as standby are provided. The compressed carbon dioxide passed through these filters is then sent to the absorbers.

5.2.2. Carbon Dioxide Absorption

The compressed gas after purification is passed to the absorbers where carbon dioxide is dissolved into the water.

For each train the absorber will be provided in duplicate one on service whiles the other as standby.

The absorbers are of counter current packed type equipped with distillate and gas distribution facilities. The system will assure a recovery ratio of not less than 95% of the total inlet carbon dioxide.

Due to the very high solubility of carbon dioxide into water only a limited portion of the distillate is admitted into the absorber to have high CO_2 concentration and after the tower the acidified water is mixed with untreated distillate by-passed and this total then passed to the limestone filters.

5.3. Limestone Dissolution

The limestone is a chemical compound normally found abundantly in nature (usually not 100% pure). In our case it should contain not lass than 90% of $CaCO_3$ (90% purity).

The limestone dissolution is achieved by passing the distillate containing CO_2 acidified distillate (effluent from absorber) into the limestone beds.

The reaction rate, at which the limestone dissolves, depends mainly on several parameters such as:

- limestone granulometry
- distillate temperature
- CO_2 concentration of the distillate incoming
- residence time/linear velocity

It is obvious that smaller particle size of limestone to some extent achieves a faster dissolution into the water, and therefore small particle size could be preferred as filter media. However, the small particles tend to create dust and to be mechanically destroyed, and therefore a compromise must be selected for this parameter. Particular size of 1-5 mm is used to improve dissolution, while limiting the dust and mechanical degradation of it avoiding a faster pressure drop.

The residence time of acidified distillate in the limestone filters can be calculated as:

$$R_T = A \times H/Q = H/LV \qquad (17)$$

Where:

Q – Is the distillate flow through limestone filters (m^3/h)

A – Limestone filter cross area (m^2)

H – Height of limestone layer (m)

LV – Distillate linear velocity (m/h)

The rate of dissolution is also proportional to the concentration of the CO_2 into the distillate, because, the reaction ends when the CO_2 becomes in equilibrium with the limestone. Obviously, the complete equilibrium cannot be reached if not at infinite contact time, hypothetically. Therefore a certain excess of CO_2 is required to achieve the best results in the limestone dissolution filters.

For the Doha Recarbonation System there are nine numbers of limestone filters per train connected in parallel. In normal operation having 45 MIGPD capacities seven numbers of limestone filters will be in service while two numbers of filters will be on standby.

The Limestone Filters are to be back filled periodically with Limestone chips in order to make-up for the limestone consumption and to maintain the residence time. After every refilling operation the limestone filters are to be backwashed to remove insolubles dust etc. and reduce the pressure drop.

5.4. Backwashing & Water Recovery

Air scouring of the limestone filters, as well as periodical backwashing must be done in order to avoid clogging of the limestone beds and also whenever the limestone filter is back filled with limestone chips.

The backwashing is made by distillate water, and this water also contains large amount of calcium carbonate. This is recovered by a settling tank where the slurry is separated from the water and discharged.

The recovered water is passed from the settling tank to the recovery tank, and after pumping and sand filtration, it is returned back to the system. The sand filter also will be backwashed periodically after every backfilling operation to avoid clogging.

This Water Recovery System saves a large amount of distillate water, because the filter backwashing is required periodically in order to avoid great filter clogging which requires considerable amount of distillate.

However a certain amount of limestone is lost as dust or slurry, apart from that dissolved into the water. The filters must therefore be periodically refilled by fresh limestone using filling devices.

5.5. Degasifying and Chemical Injection

5.5.1. Degasifying

The Recarbonated Distillate coming out of the limestone filters contains higher alkalinity so that this when mixed with Plant By-pass Distillate will give the required alkalinity. The effluent from the filters will have higher CO_2 excess. In order to minimize caustic soda consumption the CO_2 excess present to be decreased by using the degasifier (desorption tower) where excess CO_2 is removed by bubbling air into the atmospheric tower. This increases the capital cost, but decreases the running cost of the plant considerably.

The degasifier is a packed counter current flow type stripping tower. At the bottom of the tower air is distributed to remove the CO_2 gas. The recarbonated water is distributed through the specially designed distributor at the top of the packing. The degasifier fans supply the required stripping air by adjusting the air flow rate from degasifier fan, the pH value can be slightly controlled.

The sizing of the degasifier is designed taking into consideration that at the bottom of the tower the recarbonated distillate is almost CO_2 free. Provision is also made to by-pass the degasifier when this desorption tower is taken for maintenance.

5.5.2. Chemical Injection

After degasifying the recarbonated distillate, it is mixed with the plant by-pass distillate. Caustic soda solution is injected to this to neutralize and increase its saturation index and maintain the product at the required pH.

The chemical dosing system mainly consists of duplicate sodium hydroxide dosing tanks duplicate dosing pumps and duplicate agitators and normally one will be in service while the other is on standby.

The caustic soda solution is prepared in the chemical dosing tanks and pumped by means of reciprocating pumps into the recarbonated distillate product. At the end of the process the recarbonated distillate product will have a slightly positive Langelier Index so that this product water tends to precipitate making the same more or less passive and forming a protective film, as well as avoiding corrosion problems in the distribution piping and increasing the palatability.

Results

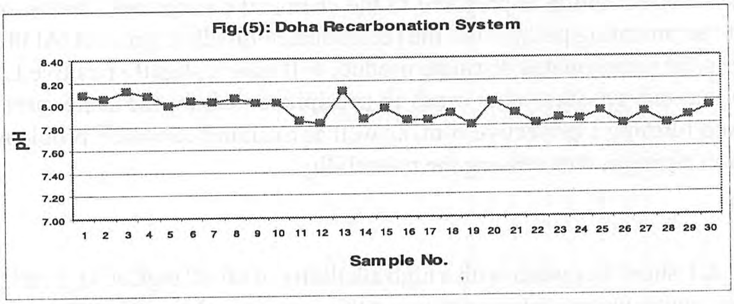
Figures 5 & 6 show that water with a high alkalinity of 60-80 mg$CaCO_3$/l and calcium content is a stable water, and can produce a thin protective layer of calcium carbonate by careful increase in the pH of 7.8-8.2. The pH at which a water containing bicarbonate and calcium is just saturated with calcium carbonate ($CaCO_3$) is known as the pH of saturation or pH. The Langelier saturation index (LSI) is defined as the actual pH minus pH. A negative index indicates a tendency to dissolve $CaCO_3$. This index is not related directly to corrosion, but deposition of a thin, coherent carbonate scale may be protective. Thus a slight positive index frequently is associated with noncorrosive conditions, whereas a negative index indicates the possibility of corrosion, Fig. (7).

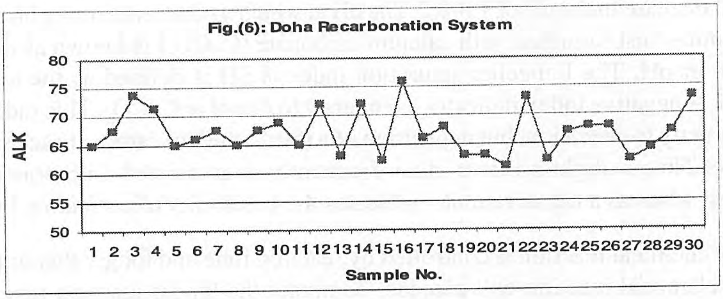
Generally chemical reaction is controlled by reaction time and longer this time is, the more the chemical reaction will precede. To obtain the longer reaction time, higher residence time is required. Hence with reference to the calculated residence limestone layer height, the variable that must be controlled to get the required alkalinity is the distillate flow through limestone filters. Similarly, once the residence time is fixed controlling the distillate flow to each filter, the outlet alkalinity can be modified acting on CO_2 concentration according to the function shown in Fig. 2.

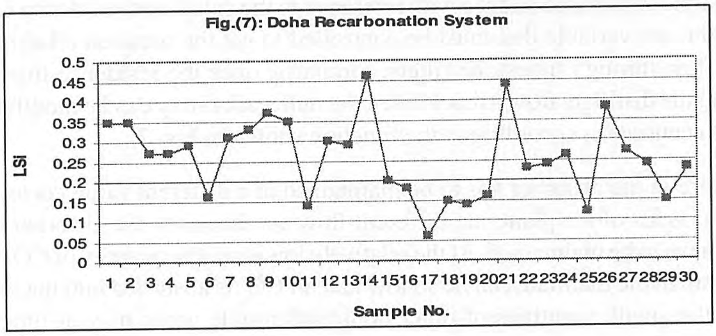
The pressure in the absorber has to be maintained at a different value corresponding to different loads of the plant, i.e. different flow conditions to the absorber different pressures have to be maintained. At the relatively low load, the quantity of CO_2 required to remineralize the distillate can be so low that all CO_2 is absorbed into the liquid and therefore the small quantities of other non-condensable gases may at times not be enough to maintain the tower pressure. To overcome this, the gas load is increased by admitting air into the tower.

CO_2 gas flow to the absorber is controlled to obtain the necessary CO_2 concentration in the acidified distillate. Acidified distillate flow into each limestone dissolution filter is controlled to obtain the required linear velocity and uniform distribution inside each device. Backwash water supply flow to the limestone filter is controlled to obtain the design value of backwash water velocity through the limestone layer.

The linear velocity must be optimized since the hold-up time will determine the total bed height, diameter and number of filters. As the linear velocity decreases the diameter or the number of filters must be increased. For high linear velocities, a large bed height is needed which will result in a higher pressure drop. For an optimum linear velocity, the alkalinity can therefore be controlled only by the CO_2 concentration in the entering water stream.







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Solar ponds utilization for seawater desalination

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SOLAR PONDS UTILIZATION FOR SEAWATER DESALINATION

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ABSTRACT

As many as 2 billion people in the World still have no access to acceptable drinking water, because natural sources of fresh water are limited. Most of them live in countries located in tropical and subtropical areas, with greater availability of renewable resources than advanced countries. Many of these countries are located along seacoasts and have higher levels of solar energy. The most favorable belt (15-35° N) encompasses such countries in the northern parts of Africa and the southern parts of Asia. Libya is one of the countries in Africa situated along the Mediterranean Sea shoreline (seawater salinity is 39.5 x 10^3 ppm). The member countries of the Cooperation Council for the Arab States of the Gulf (GCC) are located along the Arabian Gulf (water salinity is 45 x 10^3 ppm) and the Red Sea shorelines (seawater salinity is 43 x 10^3 ppm). In general, the accepted quality standards for drinking water and other domestic uses are $5x10^2$ ppm for TDS and $2x10^2$ ppm for NaCl. The thermal energy required for desalination processes can be obtained, at no cost, from the Sun by using the salinity-gradient solar pond technologies. The cost of developing this technology in GCC and Libya, which possess abundant sunshine (solar energy) throughout the year, large amounts of salts, abundant saline water, and several natural lakes, is usually much lower compared to that in advanced countries. In this paper the solar radiation conditions in GCC and Libya, and the course of technology advancement in solar ponds have been analyzed and studied. This paper summarizes the efforts needed on solar pond development in GCC and Libya. This paper also outlines the various technical problems that have to be solved during the construction and operation period of solar ponds and gives suggestions for future course of development.

Keywords: Seawater desalination, Salinity – gradient solar pond technologies, the sun, Middle East, North Africa, Solar distillation, Multi - effect distillation, Arabian Gulf states.

1. Introduction

Energy constitutes a crucial component in the development of a nation as it not only influences the economic front but also has a vital bearing on the social, environmental and other allied fronts. The cost of desalting by distillation processes is very high. Distillation processes requires a source of thermal energy (Kudish, 1991, Kumar and Kishore, 1998). The need for higher thermal energy inputs in the water sector becomes imperative in order to achieve the target of increasing the production of desalted water in the next ten years to attain the growth rate of their population. Solar ponds can have a significant role in meeting the domestic drinking water requirements in GCC. The cost of desalination has generally decreased from more than 3 US$/m^3 to as low as 0.5 US$/m^3 over time as a result of both technological advances and market processes. The big advantage to desalination is that it extracts water that is unusable, leaving fresh water available for other uses. However, there is some question about how to dispose the concentrated brine by-product of desalination. The very highly concentrated brine from distilling plants poses a major problem to be solved including a sufficient analysis of the potential impact on the marine environment.

Solar ponds are studied for its relevance in seawater desalination from the viewpoints of process applications and operational parameters. Solar ponds can provide that source of heat, which can be obtained using a free available natural source, the Sun, as seen in Fig. 1 (Hussain, 2004a).

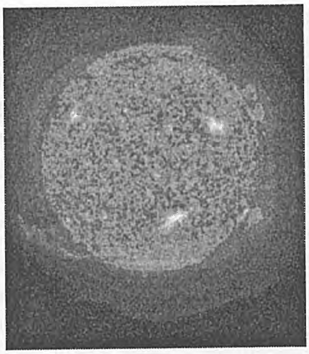

Figure 1: Solar Energy (The Sun)

2. Water Scarcity

Less than 1% of the World's fresh water is accessible for direct human use. Several arid countries in the World, such as GCC are technically in a situation of water scarcity. Water shortage is predicted to increase significantly mainly as a result of increase in population. To meet its needs of fresh water, humanity exploits all viable sources, such as, burning fossil fuels to desalinate seawater. The total amount of sea water in the World amounts to about 4 x 10^{20} m^3 and would constitute unlimited resources of raw water for coastal areas (Hussain, 2001a). The salinity of seawater in the World is classified according to the total quantity of dissolved salts (TDS) which varies from place to place as shown in Table 1. The generally accepted quality standards for drinking water and other domestic uses are 5 x 10^2 ppm for TDS and 2x10^2 ppm for NaCl (Viessman et al, 1993, and Hussain, 2001b). Table 2 gives an example of seawater chemical composition.

Many dry areas are situated along the seacoast. Other arid regions have large quantities of salty groundwater. If saline water could be converted to fresh water at acceptable costs, development of these regions would no more be restricted by water shortages and the situation might become favorable. Naturally occurring salinity-gradient solar ponds or lakes located in arid regions may be excellent sources of energy for water desalination, which can also be employed to solve drinking water problems in such areas where fresh water is scarce.

Table 1: Salinity of Seawater (TDS) in the World. (Hussain, 2001a)

Name	Water Salinity (TDS) (ppm)
Red Sea	43×10^3
Mediterranean Sea	39.4×10^3
Caspian Sea	13×10^3
Arabian Gulf	45×10^3
Pacific Ocean	33.6×10^3
Atlantic Ocean	36×10^3
Dead Sea	1×10^5
Baltic Sea	7×10^3

Table 2: Typical Chemical Composition of Seawater (Hussain, 2001b)

Element Name	Symbol or Formula	Concentration (ppm)
Chlorine	Cl	18980
Sodium	Na	10561
Magnesium	Mg	1272
Sulphur	S	884
Calcium	Ca	400
Potassium	K	380
Bromine	Br	65
Carbon	C	28
Strontium	Sr	13
Boron	B	4.6
Fluorine	F	1.4

3. World Seawater Desalination

Desalination means the conversion of saline water (seawater) to fresh water for drinking and other domestic uses. This process reduces the concentration of total dissolved solids (TDS) in water from levels shown in Tables 1 and 2 to 5×10^2 ppm or less. Selection of the type of desalination process depends upon the TD.S in raw salty water. The most common desalination processes used today are distillation and membrane processes. Historically, distillation technologies have dominated the seawater desalination market, because they generate water and electrical power (World Resources Institute, 2003).

The typical range of TDS in the feed water (seawater) for distillation is between 3×10^4 and 10×10^4 ppm. The arid region, with its very limited freshwater potential, has generally used high-salinity waters (seawater) as major water supply sources. The World desalination capacity is around 30×10^6 m^3/day.

There are some significant environmental impacts of desalination processes during the operating phase of the plants. One major impact is the discharge of brine, a concentrated salt solution, hot and may contain various chemicals, into the sea. Desalinated water from plants using distillation technologies is completely de-mineralized and needs to be re-mineralized before human consumption in order to include minerals that are essential for human nutrition.

Furthermore, if post-treatment is inadequate, desalinated water from thermal processes can corrode pipes and corrosion products may contaminate the drinking water. In addition, practically all desalination processes require chemical pre-treatment and residual chemicals need to be removed from product water through appropriate post-treatment. Any chemicals added to the desalination process for scale and fouling prevention, corrosion reduction and corrosion products flow back into the sea.

4. Case Study – Seawater Desalination in the Arabian Gulf Countries and in Libya

In some of the drier parts of the Middle East, in particular the Arabian Gulf states, where conventional good - quality water resources (fresh surface water and renewable groundwater) are not available or is extremely limited, desalination of seawater has been commonly used to solve problems of water supply arising from increasing demand for municipal and industrial uses. More than two-thirds of the World's desalting plants are located in the arid, oil-rich states of the Middle East, such as GCC and the northern parts of Africa, such as Libya. The Cooperation Council for the Arabian Gulf Countries (GCC) includes Saudi Arabia, Kuwait, the United Arab Emirates, Qatar, Bahrain, and Oman. In GCC, desalination of seawater has been used since the 1960s. In these countries, distillation of seawater is the main process being used. In spite of the high cost of seawater desalination, with unit water costs 5 to 10 times as high as those of conventional water-resources development, a vast quantity has been produced to meet the increasing demand for domestic water in GCC and in Libya. These countries have significant domestic fossil energy sources. They usually subsidize the provision of natural gas to power plants and thus subsidize indirectly the cost of electricity and steam used for desalination, which can afford the price of massive quantities of desalting equipments.

Kuwait was the first state and was one of the world's leaders in the production of fresh water from salty seawater using the Multi Stage Flush (MSF) process to distill seawater. Co-generation stations, using the MSF distillation plants, as it is called, reuses low-pressure steam from the generator to provide energy for the desalination process, were developed in the early 1950s and have been in use since then. As a result, both energy and costs have been minimized. Kuwait began desalinated water production in 1957, when the capacity was 3.1 x 10^6 m^3/year (0.1 m^3/s). Between 1965 and 1982, distillation plants were installed in response to a rapid increase in the demand for fresh water. By 1987 the figure had risen from 3.1 x 10^6 m^3/year to 184 x 10^6 m^3/year (5.835 m^3/s). The largest distillation plant in Kuwait is on the Arabian Gulf where over 0.2 m^3/s of fresh water (TDS = 25 to 50 ppm.) is produced by MSF distillation. The feed water is assumed to be of seawater quality in the Arabian Gulf. Seawater is an unlimited water source for Kuwait. It has a long coastline along the Arabian Gulf, which covers an area of 3.683 x 10^9 km^2 and holds 10.07 x 10^{12} m^3 of water. The problem with seawater distillation is the high cost of the MSF evaporation process. The cost of the thermal process is largely dependent on the rate of energy (fuel) consumption for operating the system, which can account for as much as about 50% of the plant water cost and is sensitive to the unstable world market price of crude oil. In Kuwait, there is increasing government concern about the production cost of desalinated water, and every effort is being made to ensure that water use is as efficient as possible. Qatar began desalinated water production in 1972. By 2000, its production had risen to 150 x 10^6 m^3/year. This was about 3/4 of its total water demand at that time.

Saudi Arabia began desalinated water production. in 1970. The construction of desalination plants was on both the Red Sea and Gulf coasts. The Kingdom had installed 30 desalination plant projects by the end of the 1980s. The total production of desalinated water is estimated to be 2.16 x 10^6 m^3/day including a plant at Al-Jubail producing 1 x 10^6 m^3/day (11.6 m^3/s), which is currently the world's largest distillation plant.

The first MSF distillation plant in Bahrain began desalinated water production in 1984. The total installed capacity of this plant was 23 x 10^3 m^3/day in 1981, which was 15% of the total demand of 154 x 10^3 m^3/day. The present installed capacity of desalination plants in Bahrain is 20.5 x 10^3 m^3/day, including 16 x 10^3 m^3/day of seawater distillation by MSF.

5. Installed Capacity of Desalination Plants in the Arabian Gulf Countries

There are about 1483 desalination units operating in GCC, which account for 57.9% of the worldwide desalting plant capacity. World desalination capacity is around 30 x 10^6 m^3/day and growing. The installed capacity of desalination plants in GCC is estimated at 5.76 x 10^6 m^3/day in total. The capacity of desalination plants in Saudi Arabia is approximately 1/2 of the total desalination capacity of GCC. The MSF process is the most commonly used technique to desalt seawater, representing 97% of the total installed capacity. It has served very well during the past ten years. The MSF desalting has proved to be the simplest, most reliable, and most commonly used seawater system in large capacities. It has reached maturity with very little improvement in sight. This maturity is expressed in reliable designs of large units up to 38 x 10^3 m^3/day, long operation experience with high on-line stream factors (up to 95%), confidence in material selection, and very satisfactory water pre-treatment.

6. Solar Energy in the Arabian Gulf Countries

The sun generates an enormous amount of energy, approximately 1.1 x 10^{20} KWh/s (A kilowatt-hour is the amount of energy needed to power a 100 watt light bulb for ten hours.), approximately 7 x 10^7 KWh/d, reaches the surface of the earth. The amounts of solar energy arriving at the earth's surface vary over the year. The availability of solar energy varies with geographical location of site and is the highest in regions closest to the equator.

Solar energy forms an important constituent of renewable energy sources in GCC with an average incident solar radiation of 5.7 KWh/m^2/d during the year. This energy, if fully utilized, is 2 x 10^4 times the current electrical energy consumption. Solar energy is clean, safe and abundant in these countries. Solar ponds, essentially large area solar energy collectors, can provide cheap means of collection and storage of energy.

In the summer, the degree of temperature ranges between 30° C in coastal regions and 50° C in the desert of these countries. The Arabian Gulf Countries are arid regions. They have over 3 x 10^3 h/year of sunshine duration and limited cloud coverage. More than 90% of the incident solar radiation comes as direct radiation. The quantity of solar energy received on the ground surface is about 13 x 10^9 cal/km^2 on a sunny day, thermally equivalent to the recovered energy in 15 x 10^3 barrels of petroleum. The costs of desalted water produced by large-scale desalination plants in different regions of the World are around 0.7 US\$/m^3 and in the oil-rich states of the Middle East (GCC) and the northern parts of Africa, such as Libya, which can afford the price of massive quantities of desalting equipments, are in the order of 0.45 US\$/m^3 to 0.52 US\$/m^3. The unit costs for smaller plants are significantly higher. The main reason of these costs is the source of energy used for MSF distillation plants operation. The water problem in this region can be solved with solar water distillation. Renewable energy, such as solar energy, provides clean and reliable energy sources, which can be used in GCC in many applications, such as, seawater desalination. Table 3 shows solar distillation plants, in different regions of the World, in operation since 1987.

Table 3: Solar Distillation Plants in Operation (Garcia-Rodriguez et al, 2001).

Location of Solar Distillation Plant	**Desalination Process**	**Capacity (m^3/d)**	**Type of Solar Collectors**
La Desired Island, French Caribbean	ME, 14 effects	40	Evacuated tube
Abu Dhabi, UAE	ME, 18 effects	120	Evacuated tube
Kuwait	MSF autoregulated	100	Parabolic trough
La Paz, Mexico	MSF,10 stages	10	Flat plate + parabolic trough
Arabian Gulf	ME	6000	Parabolic trough
Al-Ain, UAE	ME, 55 stages MSF, 75 stages	500	Parabolic trough
Takami Island, Japan	ME, 16 effects	16	Flat plate
Margarita de Savoya,Italy	MSF	50 – 60	Solar pond
Berken, Germany	MSF	20	------
Islands of Cape Verde	Atlantis Autoflash	300	Solar pond
University of Ancona, Italy)	ME–TC	30	Solar pond
PSA, Almería, Spain	ME, heat pump	72	Parabolic trough

7. General Description of Salinity-Gradient Solar Pond Technology

A solar pond is a large reservoir of saline water, with the exception that a salt concentration profile is artificially achieved and maintained in the pond, that collects and stores solar energy. Solar energy will warm a body of water that is exposed to the Sun. Salinity-gradient solar ponds technologies are essentially low cost per unit area of solar collectors with integrated storage, inherent storage capacity, and are easily built over large areas. They operate at moderately high temperatures. The solar pond can be a repository for concentrated waste and hot brine. A typical salinity-gradient solar pond consists of three main layers of water different in salt concentration, degree of temperature and the depth as seen in Figure 2. The specifications of salinity-gradient solar ponds are shown in Table 4. Depth of water in the pond is from 2.8 to 4.2 m. The lower layer is a homogeneous and very salty solution which is called the hot brine zone (LCZ). This layer can be a reliable heat source at temperature levels of 71° C to 100° C. The middle layer is the important zone. Here salt concentration increases gradually with depth, thereby forming a linear salinity gradient. Water in the gradient zone cannot rise up, because the water above it has less salt content and is lighter. The stable gradient zone suppresses convection and acts as a thermal-insulating layer (a transparent insulator) for the lower convective zone. It permits sunlight (solar radiation) which reaches the hot bottom layer of the pond to be trapped there (from which useful heat is withdrawn), or stored in the form of hot brine and may be used as thermal energy required for desalination process, as seen in Figure 3. This is because the salt gradient, which increases the brine density with depth, counteracts the buoyancy effect of the warmer water below (which would otherwise rise to the surface and lose its heat to the air). The thermal conductivity of water is moderately low, and if the gradient zone has substantial thickness, heat escapes upward from the lower zone very slowly. The insulating properties of the gradient zone, combined with the high heat capacity of water and large volume of water make the solar pond both a thermal collector and a long-term storage device. To control the wind, plastic fencing material can be used.

A solar pond multi-effect distillation (MED) system comprises a set of evaporative condensers and heat exchanger extracting heat from the solar pond as shown in Fig.3. The fresh water is produced through repetitive cycles of evaporation and condensation, using low temperature heat from the solar ponds. A pond of 1/3 km^2 surface area could operate a MED unit, with an annual mean output of 4 x 10^3 m^3/day.

Table 4: Specifications of the Salinity Gradient Solar Pond Technology (Hussain et al, 2004a).

Water Zone No.	Water Zone Location	Water Zone Name	Water Salinity (ppm)	Min. Zone Depth (m)	Max. Zone Depth (m
1st	Surface (Cold) Zone	UCZ	1 x 10^3	0.8	1
2nd	Middle Zone	NCZ or GZ	82 x 10^3	1	2
3rd	Lower or Bottom (hot brine) Zone	LCZ or SZ	2 x 10^5	1	1.2

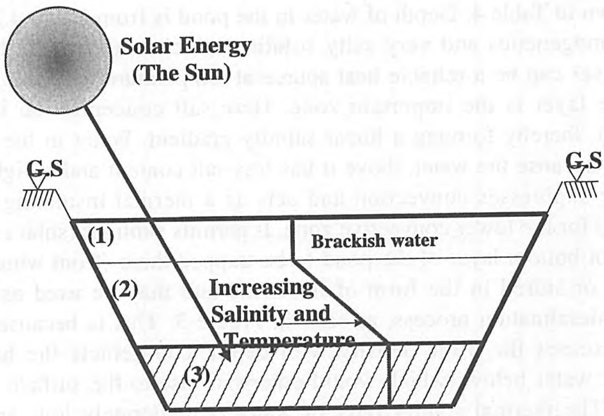

Figure 2: Schematic Presentation of Salt- Gradient Solar pond.
(1) Surface Zone or Upper Convective Zone (UCZ).
(2) Non-convective Zone (NCZ) or Gradient Zone (GZ).
(3) Lower Convective Zone (LCZ) or Storage Zone (SZ).

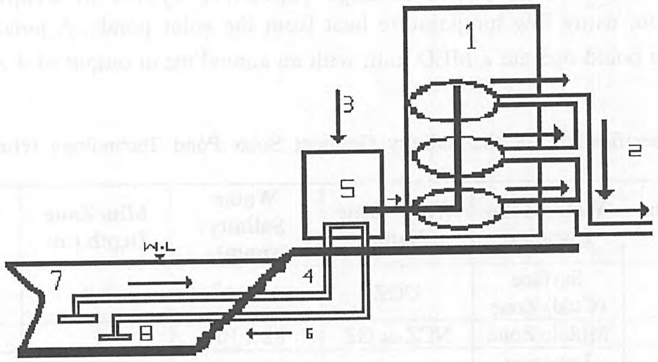

Figure 3: Schematic Diagram of Hot Brine withdrawn from Salt- Gradient Solar Pond for Desalination Plant. (1) Multi - Stage Flash Pan. (2) Fresh Water. (3) Brackish Water. (4) Hot Brine. (5) Heat Exchanger. (6) Cold Brine. (7) Solar Pond. (8) Diffuser.

8. Design and Construction of Solar Pond

Salinity-gradient solar ponds require large areas, large quantities of salt and water, and trained persons for operation and maintenance. Manpower requirements are not proportional to the size of the pond, and decrease rapidly on a unit area basis as the area increases. The requirements of land, salt and salty water suggest that solar ponds are better constructed on wastelands or desert lands close to salt works, and in GCC context, several of such sites can be identified in the coastal and desert regions. Solar ponds have a low capital cost owing to the fact that they are based on low cost materials like clay, plastic and salt. Cost of solar pond construction in GCC is about 20 US$/m^2.

The lining scheme (for the bed and side slopes of the pond) should be impermeable to hot brine. The coefficient of hydraulic conductivity K of lining layer is 1 x 10^{-4} m/day (0.1 mm/day). The lining scheme can be done by using clay and the high oven life of special polyethylene membranes like XR-5 (Hussain et al, 2004a), Hypalon and polypropylene (Folchitto, 1990). The membranes should be over lapped. The purpose of the lining is to stop the leakage (seepage) of hot brine and to avoid the failures in plastic material. In fact, the tear resistance of tensile strength of any polymer is dependent on the thickness, nature and extent of additives, exposure to ultraviolet light and temperature. It has been well established that the oven life decreases rapidly with temperature. When the time of usage exceeds the useful life of the material, any small disturbance, such as, settlement of embankments can cause ruptures. Usage of recycled plastic has to be avoided. A lining scheme consisting of two layers of 0.25 mm low oven life polyethylene membrane LDPE film sandwiched between three clay layers (thickness of each layer is 0.3 m) can be employed for the pond (Raman et al., 1990). This lining scheme is adequate. The types of clay should be either bentonite or Kaolinite. The advantages of such lining scheme are that the polymer film will be buried; shielding it from exposure to high temperature, and the sandwiching clay between plastic membranes will make the lining scheme quite insensitive to blow holes and minor defects in the film. In general, side slope (V:H) of the pond depends on the type of soil in that area. Because the type of soil in GCC is sand a side slope (V:H) for the solar pond to be used is (1:4) in order to achieve structural stability of the side slopes of the pond, ease of soil compaction with rollers and transport of material to and from the pond. Head of water (depth of water) in the pond can be from 2.8 to 4.2 with lining thickness of about 1 m. The level drop of water (head loss) in the pond can be reduced by sandwiching to 0.03 mm/day. The large mass of saline water in the lower zone of the pond gets transformed into a large thermal storage, from which heat can be extracted for useful purposes.

9. Achievement of Salinity-Gradient Solar Pond Technology

A salt-gradient solar pond is achieved artificially by dissolving salt in water at different concentrations along the depth of the solar pond. Quantity of salt required for ponds depends upon the volume of water stored in it. For a large pond with a surface area of 6 x 10^3 m^2, 3.2 x 103 tons of salt are required. To dissolve the required quantity of NaCl salt to get the hot brine, the time required is 6 months (using manual mixing) and 1 month (using mechanical mixing). A mixing tank is provided with a mechanical stirrer in the centre of the pond to ensure lateral uniformity of the salinity gradient. It is driven by a 15 hp motor.

The salinity gradient is establishment takes a week. The pond is heated up within 2 to 3 months after establishing the salinity gradient. Once the pond is heated up, it is very important to maintain the clarity of the pond. The pond is made as transparent as possible by periodically treating it for algae and dust control. For this reason, hydrochloric acid and copper sulphate are fed to the bottom of the pond.

10. Heat Extraction from the Salinity-gradient Solar Pond

For small solar ponds, heat extraction from its lower zone, can be done by submerged heat exchangers (copper pipe structures) in that zone. For large solar ponds (water surface area is greater than 1.2 x 10^3 m^2), the most common method of heat extraction is to pass hot brine from the low convective zone through an external heat exchanger, placed outside the pond. A suction diffuser (mild steel, 1.5 m in diameter) is used for taking out the hot brine from the pond to the external heat exchanger. A return diffuser is used for returning the cooler brine from the external heat exchanger, by pumping, to the storage zone of the pond after its circulation through the external heat exchanger. The suction diffuser is placed 0.8 m above the bottom of the pond, and the return diffuser is placed 0.2 m below the suction diffuser. Both the diffusers are on the same side of the pond. They are separated by a horizontal distance of 3 m. Brine flow rate of 70 m^3/hr in the suction diffuser is required. Heat extraction takes 7 to 10 hrs/day.

11. Case study- Operation of the El Paso solar pond

The El Paso Solar Pond is a research, development and demonstration project operated by the University of Texas at El Paso. It was the first in the world to deliver industrial process heat to a commercial manufacturer since 1985, the first solar pond electric power generating facility in the US in 1986, and the nation's first experimental solar-pond-powered desalting facility in 1987 (Xu et al, 1993). The El Paso solar pond has a surface area of 3 x 10^3 m^2 and a depth of about 3.25 m. The UCZ, NCZ, and LCZ are approximately 0.7 m, 1.2 m and 1.35 m, respectively. The pond uses an aqueous solution of predominantly sodium chloride (NaCl). The LCZ contains saturated or nearly saturated brine with a concentration of about 26% by weight (2.6 x 10^5 ppm). The bottom temperature increased at a rate of about 1°C per day during start-up, while the temperature fluctuation in the LCZ is (1 to 3)°C between day and night due to the thermal storage capacity in the pond if no heat is extracted. This is a very important factor for providing a steady heat supply to the thermal desalination processes. During a typical day in the summer, the storage zone temperature starts to increase at about 8 a.m. and stops increasing at about 8 p.m.. The operation temperature of the pond ranged from 70°C in winter to 90°C. The highest temperature observed at the El Paso solar pond during these years was 93°C, and the maximum temperature difference between the LCZ and UCZ was well above 70°C. The observed temperatures in the storage zone at the bottom of the pond were influenced by ambient conditions and periodic heat removal to operate testing equipment and generate electricity. During the summer months heat is specifically removed from the solar pond, usually by generating electricity in order to maintain the stability of the gradient zone and to prevent boiling. The temperature of the LCZ varies seasonally. More generally, the rate of heating of the storage zone is proportional to the incoming solar radiation and inversely proportional to the thickness of the storage zone. The thickness of the storage zone can be increased to increase the storage capacity or decreased to increase the temperature response.

The salinity gradient was built by utilizing the scanning injection technology which was developed by the El Paso Solar Pond Project. The procedure consists of partially filling the pond with saturated brine and injecting fresh water in a scanning step-by-step fashion through a diffuser that is immersed within the existing solution. With these new techniques, the salinity gradient was built with great ease, was less labor-intensive and less time-consuming. Most importantly, the achieved salinity profile was much smoother and matched well with the desired profile. It took 4 days to build the gradient. A transient temperature gradient became established immediately after the salinity gradient was established, and the bottom temperature of the pond increased at an average rate of about 1°C per day. In about 2 months, the bottom temperature reached 80°C (Lu et al, 1996). At the El Paso solar pond, brine withdrawal is the method used for heat extraction. The method is to pump the hot brine from the storage zone of the pond to a heat exchanger located near the pond. Hot brine is pumped from the storage zone by means of a diffuser (extraction diffuser) mounted in the storage zone, passed through an external heat exchanger, then returned to the bottom of the pond through another diffuser (return diffuser). The extraction diffuser can be moved to the height of maximum temperature in the storage zone and the return diffuser is placed at the pond bottom. This method allows placement for both the extraction and return diffusers near the point of use, reducing pipe cost. Also, this method insures that the cooler brine is returned to the bottom, reducing ground losses, and that the piping can be easily removed for inspection and repair. Both suction and return diffusers are double plate diffusers. The opening of the suction diffuser is covered with stainless steel screen to prevent the piping system from sucking in debris. The return diffuser is placed at the pond bottom on a gravel bed. The gravel bed is about 10 cm thick, and below the gravel lies a piece of 10 mm polypropylene which covers the sand and prevents it from being washed away by the brine exiting the diffuser. The maximum withdrawal flow rate for this design is 2.3 m^3/min, and at this flow rate the exiting velocity is less than 7 cm/s.

12. Conclusion

From this study, it is concluded that the salinity-gradient solar pond technology seems to be appropriate, relevant and promising for arid and dry regions, and developed countries located in Middle East specially the Cooperation Council for the Arab States of the Gulf (GCC) which are located along the Arabian Gulf shorelines (water salinity is 45 x 10^3 ppm) and the Red Sea shorelines (water salinity is 43 x 10^3 ppm). The cost of developing the technology in these countries is usually much lower compared to that in advanced countries. Fresh water shortage problem in GCC and Libya for municipal demand (drinking and other domestic uses) could be solved and the situation in the cities located along the coastlines be improved and becomes more favorable if desalting seawater processes be achieved at an acceptable cost. Price of produced desalted water depends upon the type of energy source used in desalination process. To reduce the cost of desalted water produced by the desalting units in GCC and Libya they have to increase the efforts to promote renewable energy utilization by providing a financial support for the solar pond research. GCC possesses abundant sunshine (solar energy) throughout the year, large amounts of salts, abundant saline water, wide area and several natural lakes. The requirements of land, salt and salty water suggest that solar ponds are better constructed on wastelands or desert lands close to salt works, and in the GCC and Libya context several such sites can be identified in the coastal and desert regions. By using salinity-gradient solar pond technology in GCC and Libya to produce cheap energy to drive desalting units. Large quantities of desalted water from seawater produced will be sufficient for municipal water demand (drinking and other domestic uses) in the large coastal cities and the other important regions.

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Premeate flux enhancement in cross-flow ultrafiltration by turbulence promoter inserts in tubular membranes

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PERMEATE FLUX ENHANCEMENT IN CROSS-FLOW ULTRAFILTRATION BY TURBULENCE PROMOTER INSERTS IN TUBULAR MEMBRANES

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ABSTRACT

The development of the membrane processes is hampered by the relatively low flux performances due to concentration polarization and fouling, which has very serious operational, economic and environmental implications and is one of the factors that affect the lifetime of the membrane, reduce the permeate flux and limit the performances of the filtration processes. The deposit formed on the membrane surface, which causes blockage of flow passage and lower quality of the permeate, can be reduced with different techniques by increasing the shear stress in the vicinity of the membrane surface without increasing the flow rate. The use of turbulence promoter inside the membrane element is one of such techniques. The selection of an appropriate static turbulence promoter is vital to get improved permeation flux with a minimum pressure drop for cross-flow feed. In this investigation we selected the helical baffle as the static turbulence promoter since it gave higher permeation flux by reducing the membrane fouling. The number of helices per unit length has a considerable influence on the selected helical baffle. An optimum permeation flux was found for stems having 3 helices every 40 mm. Studies were carried out on the treatment of suspensions containing biological and suspended solids using inorganic tubular membranes. We found 1 bar as an optimal pressure, above this pressure the permeation flux decreases, contrarily to several works, which observe a plateau after a certain value of pressure. Progressive fouling can be limited by use of helical baffles in the filtration element operated at low pressures and low cross-flow velocities and the flocculation of particles is reduced. At the same operating parameters, the insertion of static helical baffles inside the membrane tube caused improvement of the permeate flux of more than 50 %. Moreover, the helical baffles provided the similar flux value at about 3 times lower cross-flow velocity, which means large energy saving.

Keywords: Ultrafiltration; Flux enhancement; Shear stress; Cake layer; Turbulence promoter

1. Introduction

One of the most important problems in applying membrane technology is the usual concentration polarization and membrane fouling, which has very serious operational, economic and environmental implications and is one of the factors that affect the lifetime of the membrane, reduce permeate fluxes and limit the performances of the filtration processes. The deposit formed on the membrane surface, which causes blockage of flow passages, can be removed with different techniques, mainly by using a static turbulence promoter inside the membrane tube which increases the shear stress near the membrane surface. The significant disadvantage of the turbulence promoters is the increase of the pressure drop which disturbs the ultrafiltration (UF) operation.

Several authors used different techniques to reduce the concentration polarization and membrane fouling such as use of baffles [1,2,3,4,5,6] and spacers [7], increasing the cross-flow velocity or back-flushing [8,9], feed pulsation in a baffles tubular membrane system [10] or backpulsing [11,12], use of rotational membranes [13] or corkscrew vortices formed in helical flow passage by the interaction of dean vortices and axial flow component [14] and air or gas sparging during filtration [15,16,17,18,19,20,21,22]. All these authors observed a significant increase in the permeation flux with improvement up to 700%.

Gupta et al. [2] reported that the permeate flux increased with increase in number of helices by unit length. They found an improvement of flux up to 50% at the same operating conditions. Moreover, visualizing by video camera revealed that flow was rotational around the baffle axis that was responsible for enhanced mixing leading to migration of suspended solids away from the membrane surface. Metal grate type of helical baffles (crimped lozenges meshes type) are also used by Sebbane [5]. He found that the permeate flow-rate decreases when the thickness of the fluid vein increases. Other researchers, Bennasar [1], Maubois and Mocqout [4], using similar helical baffles, found that the ultrafiltration of milk gave better results when the hydraulic diameter D_h was reduced. The latter believes that D_h plays an essential role on the formed layer on the membrane surface and on his internal fouling.

Sebbane [5] showed that the gel resistance decreases with hydraulic diameter. The shear stress can influence internal fouling while acting on the gel layer, which lead to an increase of the permeate flux more important than foreseen. We can assume also that the gel concentration varies with hydraulic diameter.

Different Kenics static mixers were used as a turbulence promoter by several authors. Krstic et al. [6] used three Kenics static mixers with different material and characteristics in MF of skim milk. They observed an improvement of permeate flux of more than 700% at the same operating conditions. Moreover, they found that using a static mixer provided the similar flux value at about 5 times lower cross-flow velocity, which corresponds to the reduction of 80% in energy consumption. They reported that the use of the kenics static mixer results in the formation of a helical component of the flow, which enhances radial mixing and creation of secondary flows. Vatai and Tekic [23] used Kenics static mixer in UF of aqueous solution of pectin and sodium carboxymethylcellulose. They observed an increase of permeate flux of about 4 times by decreasing significantly the gel layer concentration. Sugimoto et al. [24] used a twisted tape and static mixer in UF of Dextran T500 aqueous solution. The permeate flux was improved 4 times with twisted tape and more than 7 times with the use of static mixer.

Ahmad and Mariadas [3] used different geometries of helical baffles with different number of turns per unit length and different baffle structures in microfiltration of TiO2 particle. They found an improvement of flux up to 104.9% using optimum baffles geometries of 4 turns per 50 mm baffle length.

Yeh et al. [25,26,27] investigated the effect of hydraulic behavior on UF membrane with a steel rod wrapped by a wire spiral of varied angles inserted concentrically in a tubular module at a uniform rate along the flow channel. They reported that increasing the wire spiral angle reduced the cross section of flow channel, as well as increased the fluid velocity in the tubular membrane module which reduce concentration polarization layer, resulting in improved performance. On the other hand, the insert of wire-rod decreases the average transmembrane pressure due to increase in frictional pressure loss.

All these authors reported that the flux improvement was attributed to a reduction of the cake or the gel layer formed on the membrane surface by increasing the cross-flow velocity, thus the shear stress.

The aim of this work was to investigate the flux improvement by using static helical baffles inside the membrane tube. After preliminary screening, we selected the helical baffle since it gave higher permeate flux. The number of helices per unit length has a big influence on the selected helical baffle. All experiments have been conducted with an inorganic tubular membrane manufactured by TechSep. The second step was the study of the influence of the pressure, the flow rate and the shear stress on the flux performance.

2. Experimental

2.1. Helical baffle

The helical baffles in this study come from suggestions of several authors [1,2,4,5]. These are wound stems in circular helices, which can consist of a variable number of helices per unit length, as shown in Figure 1. To the difference of the systems proposed by these authors, their installation in the filtration element implies the existence of contact points between the helix and the membrane surface to avoid any moving or vibrations due to fluid flow.

Fig. 1: Schematic of the used helical baffle fabricated using stainless steel.

The geometric parameters of the used helical baffle are gathered in Table 1, knowing that the hydraulic diameter is defined by:

$$D_h = \frac{4\Omega'}{P_h} \tag{1}$$

Table 1: Geometric parameters linked to the used helical baffle.

D, mm	Membrane tube diameter (internal)	6
δ, mm	Helical baffle thickness	1
Ω, mm^2	Membrane cross section	28,3
Ω', mm^2	Cross section with helical baffle	23,3
P_h, mm	Wetted perimeter	30,8
D_h, mm	Hydraulic diameter	3

2.2. Unit and membrane

The experimental unit used is shown schematically in Figure 2. Temperature and feed concentration were maintained at 30 °C and 5 g/l respectively. The ultrafiltration module operates in a closed loop where the permeate and the retentate were recirculated in the tank and mixed. Temperature is maintained homogenous using a helical coil heat exchanger immersed in a 10 liter feed tank. The selected Carbosep membrane, manufactured by TechSep, was an inorganic composite membrane whose zirconia-active layer was deposited on a carbon support. This tubular membrane has an internal diameter of 6 mm and a length of 40 cm. The membrane cut-off as given by the manufacturer is 300 000 Daltons. All experiments have been carried out at the optimum values of the operational parameters obtained without helical baffles.

The suspensions were made with suspended solids collected from the settler of an activated sludge plant. The membrane was cleaned chemically with a 3% sodium hydroxide and 3% nitric acid solutions after each experiment until the membrane permeability was regained.

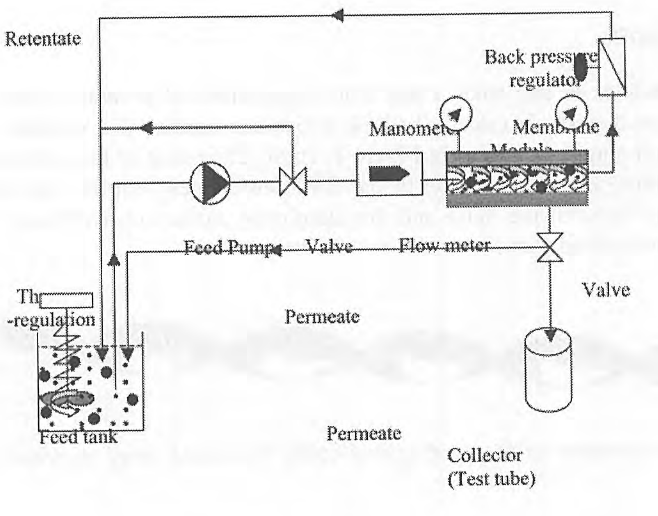

Fig. 2: Experimental unit.

3. Results and discussion

3.1. Flux improvement

The general form of the curves, permeate flux against time, is not changed by changing the pitch but it affected the time necessary to reach the steady state and its value (Fig. 3). At 0.5 bar the flux remains noticeably constant and the fouling is almost avoided and it therefore could be convenient to operate an industrial membrane at such low pressure. It is probably the critical flux similar to that defined by Howell et al. [28]. It is remarkable to know that the critical flux is close to the steady values at high pressures. At the optimal operating parameters the steady state flux is improved by about 50% when the helical baffle was inserted.

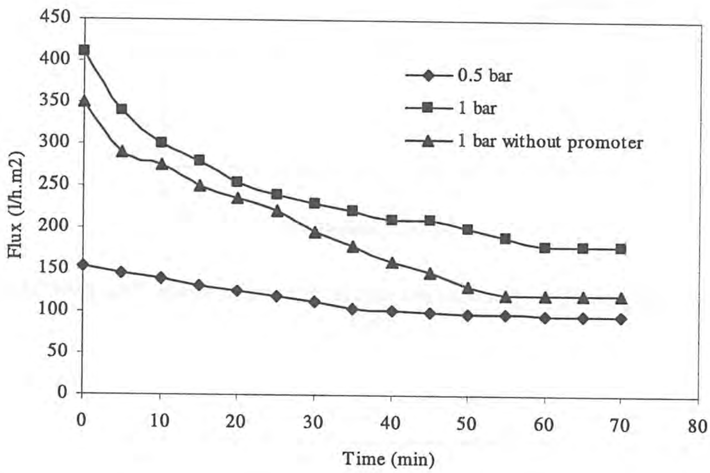

Fig. 3: Permeation flux against time for different pressures, Q=1.25 l/min.

3.2. Influence of the helices pitch and driving pressure

An optimum flux is obtained when the baffle has 3 helices every 40 mm (Fig. 4). Similar results were obtained by several authors. Ahmad and Mariadas [3] found an optimal flux with helical baffle having 4 turns per 50 mm. The additional pressure drop is negligible for low pressures but the permeation flux decreased when the driving pressure is higher than 1 bar (Fig. 5), whereas without helical baffle the flux reaches a plateau at about 2.5 bars. This is essentially due to the type of baffle used, which brushes against the internal surface of the membrane, helping the disruption of the deposit layer at high pressures. The helical baffle does not do this. It occupies an important volume of the membrane tube and creates an obstruction to flow. In the subsequent experiments, the helical baffle with 3 helices every 40 mm that gave maximum permeation flux was used.

The main restriction of the use of helical baffle is the increase of pressure drop leading to large energy consumption but it is more than compensated by the flux improvement achieved [6]. Moreover, the helical provided the operation at low velocity and pressure, thus minimum pressure drop.

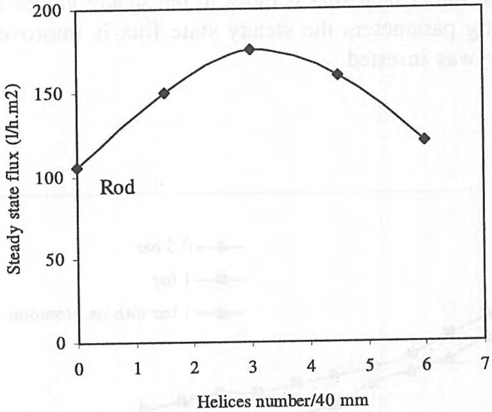

Fig. 4: Effect of the number of helices per unit length on the steady flux, Q=1.25 l/min, P=1 bar.

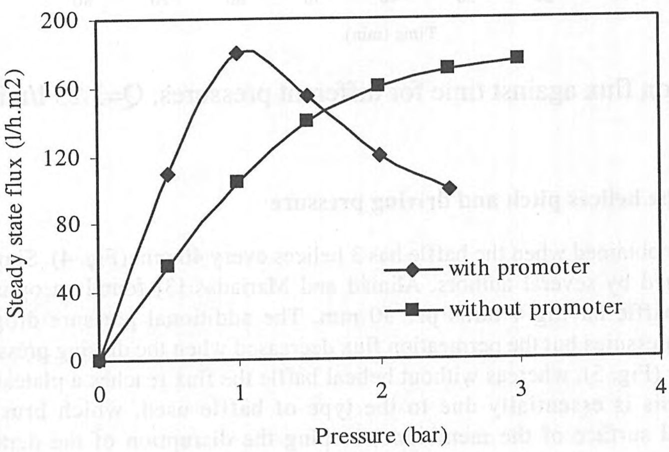

Fig. 5: Influence of the pressure on the permeate flux, Q=1.25 l/min.

3.3. Feed flow-rate

The increase of the feed flow-rate has also a beneficial effect on the permeate flux (Fig. 6). Without the promoter, the flow-rate has no influence on the flux beyond 1.25 l/min. On the other hand, the flux is proportional to the flow-rate when the helical is inserted. The presence of helical baffle increases the cross-flow velocity without increasing the flow-rate. Thus, the feed flows faster leading to migration of particles away from the membrane surface and avoid the formation of the cake layer.

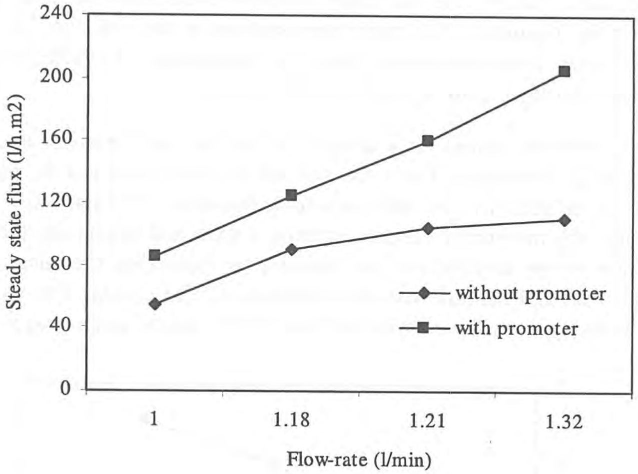

Fig. 6: Influence of the feed flow-rate on the permeate flux, P=1 bar.

The specific cake resistance is inversely proportional to the feed flow-rate (Fig. 7), this phenomenon is not observed without the helical baffle, which suggests that using this type of baffles eliminates the flocculation and the agglomeration of the deposited particles due to shear stress.

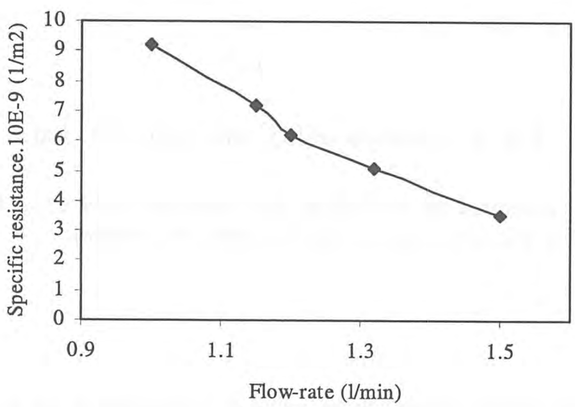

Fig. 7: Variation of the specific cake resistance with the flow rate, P=1 bar.

3.4. Cross-flow velocity and shear stress

The variations of the steady state flux against the cross-flow velocity in the studied conditions are linear with a slope equal to 0.68 corresponding to the system of the flow (Fig. 8). The cross-flow velocity is calculated using free cross sectional area. Several authors, using different helical baffles, found different slope values. Gekas and Hallstrom [29], Aimar et al. [30] explained this difference since the diffusion coefficient, the viscosity and the density (D, ì and ñ) depend on the gel concentration and not on the bulk concentration. Indeed, it is necessary to know as Jaffrin et al. [31] found that the D and ì affect the friction at the membrane surface or the velocity. The suspensions could have, as milk, a pseudo-plastic behavior. In addition, the influence of the shear should be much stronger than when the layer is thick.

When the rod without helices is inserted, the flux is not improved even though the cross-flow velocity increases. The cake already formed could not be pulled out. The fouling is then reduced by the helices which produce a helical flow and generate turbulences near the membrane surface creating vortex and enhancing scouring leading to the reduction of the thickness of the deposit by increasing the shear stress against the membrane wall and increase the permeation flux. The similar flux value at about 3 times lower velocity is achieved with helical, which means large energy saving.

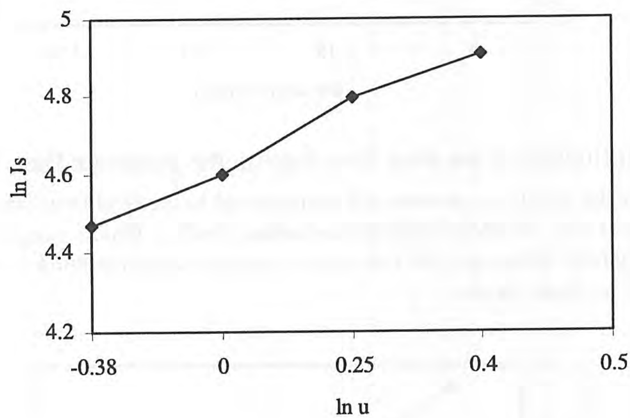

Fig. 8: Variation of $\ln J_s$ with $\ln U_c$, P=1 bar.

The shear stress ô against the membrane wall depends on the cross-flow velocity of feed solution [29]; it is calculated by the following relationship:

$$\frac{f}{2} = \frac{\tau}{\rho u^2} \qquad (2)$$

Where f/2 is the friction factor whose value is a function of the Reynolds number R_e=ñud/ì, ñ is the liquid density, u is the cross-flow velocity, d is the filtration element internal diameter and ì is the liquid viscosity. The friction factor is then obtained by one of the following relationships:

$$\frac{f}{2} = \frac{8}{R_e} \qquad \text{For } R_e \text{ £ } 4000 \qquad (3)$$

$$\frac{f}{2} = 0.023 R_e^{-0.2} \qquad \text{For } 5000 \text{ £ } R_e \text{ £ } 200\,000 \qquad (4)$$

The experimental data plotted in Figure 9 for the steady state flux versus shear stress shows that a plateau is reached at shear stress of 7 Pascal but when the helical baffle is introduced the flux is proportional to the shear stress and its value is higher, which confirm our approach.

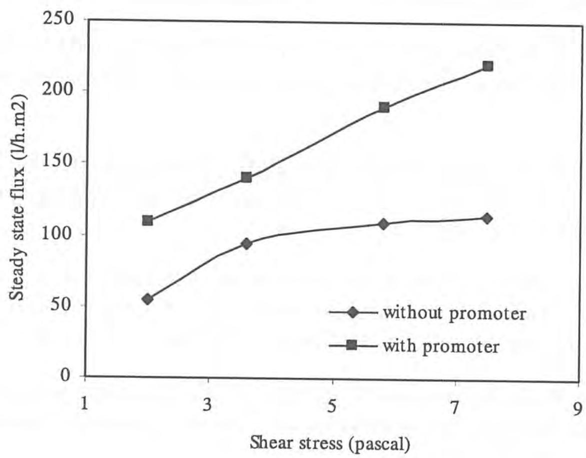

Fig. 9: Permeate steady flux against shear stress, P=1 bar.

4. Conclusion

Membrane processes often rely on the presence of a shear stress at the membrane surface to reduce the accumulation of foulants. The helical baffles allow the permeate flux to increase significantly without changing the process limiting the transfer flux. This is due to increasing the shear stress in the vicinity of the membrane surface. At the optimal conditions, 1 bar and 1.25 l/min, the flux is improved by about 50%.

The permeate flux depends, also, on the number of helices per unit length. The maximum was found for stems behaving 3 helices for every 40 mm at a pressure of 1 bar. At 0.5 bar, the permeate flux remains noticeably constant and the progressive fouling is almost avoided. Moreover, the whishes permeate flux can be achieved at lower cross-flow velocities; 3 times lower for the same flux without a promoter. It could be convenient to operate an industrial membrane at such low pressure and cross-flow velocity.

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Disinfection and disinfection by-products: A nuisance in desalination technology

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DISINFECTION AND DISINFECTION BY-PRODUCTS: A NUISANCE IN DESALINATION TECHNOLOGY

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ABSTRACT

To overcome the problem of water scarcity and facilitate social development of the nation, the Saudi Arabian Government was unprecedented in pioneering, and unparalleled in expanding, seawater desalination industry. One of the diverse utilities of desalinated water is its use for drinking and other related domestic purposes. In the latter case, it is obviously of dire importance to ensure a safe water supply. For this reason, desalinated water has to be treated with some suitable disinfectant before dispensing to the consumer. Almost all desalination plants in the Kingdom of Saudi Arabia use chlorine as the disinfectant of choice. Most of these plants produce their own chlorinating agent by electrolysis of seawater to generate chlorine, which is allowed to react with co-generated sodium hydroxide to form sodium hypochlorite. This seawater generated sodium hypochlorite, NaOCl (SW), is dosed to product water in the form of a crude chlorinating agent contained in the parent seawater matrix. Thus, there is always a fear that the carrier seawater matrix becomes charged with disinfection byproducts, DBPs, which may contaminate product water. The DBPs of concern are, in particular, halo-acetic acids (HAAs) and trihalomethanes (THMs). This fear stems from the fact that a seawater matrix contains all the precursors necessary for formation of HAAs and THMs byproducts. Apart from hypo-chlorinating agents, several alternative disinfectants are available. Ozone, chlorine dioxide, chloramines, peroxone, Cu II ion, and UV electromagnetic radiation, are examples of disinfectants considered or evaluated as alternatives for chlorine. Each disinfectant of these has some advantages and some disadvantages. In many cases, however, disadvantages far outweigh the advantages. This paper brushes only briefly over the DBPs of these alternative disinfectants. The main objective of the paper is to review, in some detail, the available literature relevant to the formation potential of chlorine generated DBPs, their constituent precursors, and the conditions that promote or suppress their formation reaction. In fact, there is a mounting literature, published in a large number of international journals, in relation to the topic of chloro-disinfectants and their by-products. Surveying such massive literature indiscriminately in this paper may not be very constructive or recommendable, not only because it may overwhelm the main purpose of the paper, but also because of the fact that the most important aquatic precursors for DBPs formation, i.e. dissolved natural humic, and anthropogenically introduced, organic compounds, are indigenous and site specific to the locality where they belong. Even bromide, the second most important precursor for THMs and HAAS

DBPs formation, varies significantly in concentration depending on the local marine environment, or groundwater aquifers, where it occurs. Thus, it was envisaged more appropriate, in relation to these compounds, to focus mainly on home-acquired literature rather than expanding far-out into regional or worldwide literature; except, of course, for brief comparative, or elucidative basic chemical information.

1.INTRODUCTION

Disinfections are inevitable for seawater desalination processes in order to combat pathogenic microbial growth in product water and also to suppress biofouling of the desalination plant components. Apart from being a mandatory step in the pretreatment sequence, disinfection is an equally important post-treatment step. Not only that, but it may also prove necessary even within the course of the desalination stream. However, disinfectants almost invariably have some problems associated with them. Toxicity of the disinfectant or its by-products (DBPs), cost, stability, potency, ease of handling, etc, are examples of those problems. Athough the chlorine-derived group of disinfectants posses an almost unbeatable germicidal effect, they are not an exception with respect to associated problems. One major problem, is the high toxicity of their disinfection by-products. To date, and over the last century, hypochlorinating agents have been the most popular disinfectants in water treatment. Different forms of hypochlorinating agents which have been most widely used, include chlorine gas, sodium hypocchlorite solutions, and calcium hypochloride, all of which are commercially manufactured. In addition to these, some major desalination plants synthesize their own hypochlorite disinfectant on-site. This is usually done by electrolysis either of seawater or brine (Figure 1a and 1.b). However, because raw seawater may contain a myriad of contaminants, there is a fear that some of these contaminants may persist in the generated hypochlorite and, on treatment, find their way to the product water. Even with other commercial hypochlorinating agents, there is still a possibility that the strongly oxidizing agent, when added to raw seawater will react with dissolved natural organic matter (NOM) and other oxidizable entities, resulting in undesirable, potentially toxic, chlorinated and unchlorinated organic contaminants which may carry over to product water [1]. These phenomena lead to very tight restrictions on the allowable level of chlorine residual in potable water, reducing it to no more than 0.1 – 0.2 mg/L. These low level restrictions have somewhat limited the effectiveness of the disinfectant.

Carry-over of volatile DBP contaminats doesn't constitute a problem in MSF desalination technique due to repeated venting of these compounds during the process. Thus, MSF distillates do not contain more than trace amounts of THMs, if at all. This is a fact that had been verified and confirmed through extensive studies as well as routine monitoring. However, the situation is different in the case of SWRO. Using the somewhat hydrophobic membranes, e.g. cellulose acetates, SWRO product water may be contaminated with DBPs such as THMs, HAA and the like. In contrast to the situation with MSF, analogous studies regarding DBPs removal by SWRO membranes are very sparse. Even the few available studies, inspite of their importance, are mostly sporadic and incomprehensive. However, a comprehensive project to study formation potential and fate of various DBPs within the course of SWRO and SW NF/RO processes is already in preparation in the chemistry department of the R&D Centre. The project will focus on monitoring the levels, identifying the nature, and tracking the fate of the DBPs and their precursors under varied conditions, varied disinfectants and varied application points.The study is particularly intended to investigate the impact of DBPs on different RO and NF membranes. A follow up study is also pending not only within the SWRO stream, but also beyond the RO permeates to include investigation of post-treatment disinfection impact on storage tanks, in simulated ground water blending, a simulated distribution system, and also along transmission lines of RO product water.

2. CHEMICAL BEHAVIOUR OF DIFFERENT FORMS OF HYPOCHLORINATING AGENTS

As a water disinfectant, chlorine can be applied in different forms of hypochlorinating agents. These forms include compressed chlorine gas, and solutions of sodium hypochlorite and calcium hypochlorite. The latter is usually marketed in solid form (powder or tablets). Chlorine can be obtained commercially for disinfection in any of these three forms or locally generated on site, as mentioned before. Economically, considering a per unit mass of active chlorine cost, calcium hypochlorite is more expensive than the other forms, followed by sodium hypochlorite. The use of liquified chlorine on a large scale is the least expensive, had it not been shrouded with risks of accidental leakage or explosion of the pressurized toxic gas. Chemically, the three commercially available forms of hypochlorinating agents as well as the on-site generated form, are equivalent (eqns. i-iii below). This equivalance is due to the rapid equilibrium established between chlorinating agents and the hydrolysis reaction products of hypochlorite compounds (hypochlorous acid in each case). In fact, this reaction has been reported to reach completion in about 100 ms [2-4].

Chlorine gas:

$(Cl_2)_{aq} + H_2O$ ® $HOCl + H^+ + Cl^-$ *(i)*

Sodium hypochlorite:

$NaOCl + H_2O$ ® $HOCl + NaOH$ *(ii)*

Calcium hypochlorite:

$Ca(OCl)_2 + 2\,H_2O$ ® $2\,HOCl + Ca(OH)_2$ *(iii)*

As can be seen from equation (iv), one mole of oxidising elemental chlorine is capable of reacting with two moles of electrons (abstracted from an oxidizable reactant) to form inert chloride. Similarly, equation (v) shows that one mole of oxidising hypochlorite also reacts with two moles of electrons resulting in the reduced form of inert chloride. Hence one mole of hypochlorite is electrochemically equivalent to one mole of molecular chlorine, and may be said to contain 70.91 g of available chlorine (identical to the molecular weight of chlorine) [2]

Thus, application of either of the hypochlorite salts of sodium or calcium in potable water achieves the same results as does chlorine gas,

Cl2 + 2 e- '! 2 Cl- **(iv)**

$OCl^- + 2\,H^+ +$ 2ç $Cl^- + H_2O$ *(v)*

The active ingredient in both of these cases is the hypochlorite ion which hydrolises to hypochlorous acid. The extent of the above hydrolysis reactions, i.e. the reactive amounts of chlorine species formed in water, is strongly dependent on pH and also on temperature, ionic strength as well as the total (initial) concentration of chlorine present.

Figure (2) illustrates the effect of pH on the relative amounts of the three species of chlorine (Cl2) , OCl- and HOCl in water. At 25°C HOCl is the predominant species between pH 1 and 7.5 whereas OCl- is predominant at pH greater than 7.5 [5].

Hypochlorous acid is a weak acid (pKa 7.5 at 25°C), which dissociates in accordance with equation (vi):

$$\mathbf{HOCl \;\circledR\; OCl^- + H^+} \qquad (vi)$$

Based on the above pKa value and the hypochlorous acid dissociation equilibrium, the relative concentration ratio of hypochlorite to hypochlorous species may be obtained from the following expression at 25°C:

$$\mathbf{\log ([OCl-]/[HOCl]) = 7.5 - pH} \qquad \mathbf{(vii)}$$

Tables (1a) and (1b) show, respectively, the effect of temperature on HOCl per cent concentration at different pH values and the per cent concentration of Cl2, HOCl, and OCl- at different buffered pH values as a function of initial solution concentration.

It has been postulated that an aqueous chlorine gas solution at pH 2 to 3 will always be somewhat more effective than a solution of sodium hypochlorite at pH 11 to 12 at the immediate vicinity of the application point. This is simply because there is more active HOCl species at the lower pH range and possibly because of the presence of some extremely active molecular chlorine on account of the low pH of the chlorine gas solution. It is a well known fact that at pH 11 to 12 the HOCl is almost completely dissociated to the ineffective hypochlorite ion [6]. However, though reported as a less effective disinfectant than the hypochlorous molecule, the hypochlorite ion posseses a higher standard potential (E°red = 1.64 v) than that of the hypochlorous molecule (E°red = 1. 48 v) [5]. The negative charge on the hypochlorite ion has been reported to retard its penetration of the cell wall of a micro-organism. This kinetic- related phenomenon may be one reason for the apparent ineffictiveness of the hypochlorite ion.

An empirical correlation between pKa of hypochlorous acid and temperature has been provided as shown belows [7,8]:

$$\mathbf{\ln pKa = 3.184 - 0.0583T - 6908/T} \qquad \mathbf{(viii)}$$

Table (2) shows the effect of temprature on the disociation constant of hypochloeous acid.

Despite the fact that the different forms of commercial and on-site generated hypochlorinating agents are chemically equivalent as far as chlorination products are concerned,, they do differ in the side reactions of their end products. The hydrolysis reaction with the hypochlorite raises the pH and alkalinity through generation of hydroxyl groups as sodium hydroxide and calcium hydroxide. On the other hand, addition of chlorine to water reduces the pH and alkalinity by increasing the H+ ion concentration (reaction v); whereas addition of calcium hypochlorite contributes further to increease the hardness of the aqueous solution and thus promotes scale formation. In the case of on-site generation, the hypochlorite differs in that it is obtained in a solution matrix of a high ionic strength.The pH, and other side effects, take place in accordance with the same reactions illustrating chlorinaion equivalence of the different hypochlorinators as represented by equations (i), (ii) & (iii), above.These equations are reproduced (with primes) below for convenience:

$$(Cl2)aq + H2O \circledR HOCl + H+ + Cl- \quad (i)'$$

$$NaOCl + H2O \circledR HOCl + Na+ + OH- \quad (ii)''$$

$$Ca(OCl)2 + 2H2O \circledR 2HOCl + Ca +2 OH- \quad (iii)'''$$

2.1 Available chlorine

The term available chlorine is commercially used to denote the relative amount of chlorine present in a chlorine gas or a hypochlorite salt. The concentration of hypochlorite (or any other oxidizing disinfectant) may be expressed as available chlorine by determining the electrochemical equivalent amount of Cl2 to that compound. The term free available chlorine is used to refer to the sum of the concentrations of molecular chlorine (Cl2), hypochlorous acid (HOCl), and hypochlorite ion (OCl-), each expressed as available chlorine.

3. DISINFECTION BY-PRODUCTS OF HYPOCHLORINATION

Disinfection by-products include a large number of organic and some inorganic compound generated as a result of application of disinfectant to different types of water. At the present time, however, there is more concern about a somewhat limited number of classes of organic compounds. These are actually the compounds identified by the United States Environmental Protection Agency (USEPA) as potentially toxic, mutagenic, or carcinogenic compounds. They are included in what became known as the priority list of pollutants. These classes of organic compounds, result mainly from the use of chlorine-derived disinfectants for potable water. They include trihalomethanes (THMs), halogenated acetic acids (HAAs), haloacetonitriles (HANs), haloketones (HKs), and miscellaneous other chlorinated and brominated organic compounds such as chloral hydarate, chloropicrin (CP), cyanogen chloride and bromide (CC & CB). The list also include some as well as inorganic oxohalides like bromate and chlorate. The first two classes of these compounds are the ones that are currently receiving most concern and attention. The potential of formation of these organochlorine compounds, apart from conditions of disinfectant concentration, contact time, temperature, pH, etc, depends mainly on the concentrations of precursors present. The most important precursors loads are the humic substance which are ubiquitous in natural aquatic reservoirs as dissolved organic carbon. Also, another contribution to the aquatic organic carbon load, is that originating from exudates of different aquatic biota, both fauna and flora. In addition, there is frequently an anthropogenic contribution aquatic organic carbon load through transport of organic pollutants from point or non-point sources, to various locations of water resources. The second most important precursor for formation of these DBP's is the bromide ion. The hypochlorinating agent, along with its disinfection action, oxidises both of these precursors. The humic portion of dissolved aquatic organic carbon constitutes the bulk of the total organic carbon. It consists of a composite mixture of highly refractory non-biodegradable polymeric macro-molecules of highly variable molecular weights. Each of these macromolecules incorpoates assortments of different organic moieties and functionalities. As such, they are not suitable for nutrition of micro-organisms. Chlorine disinfectants oxidize these molecules and break them down into simpler assimilable molecules. The products of chlorine oxidation of these organic compounds include both chlorinated and unchlorinated organic derivatives [1]. Chlorine also oxidizes Bromide ions to molecular bromine, which in aquous solution, behaves in much the same way as molecular chlorine. Analogous to chlorine, bromine equilibrates into hypobromous acid, hypobromite ion and molecular bromine forms. In these forms, it competes with their counter-parts of chlorine species for reaction with organic entities in solution. The result is the formation of a spectrum of volatile and non-volatile chloro-, bromo-, and mixed bromo-chloro- derivatives of organic compounds. THMs and HAAs, are examples of such a spectrum of halogenated products. THMs include bromodichloro- and chlorodibromo- methanes, in addition to chloroform and bromoform; whereas HAAs include monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, dibromoacetic acid and bromochloroacetic acid. These are abreviated, respectively, as MCAA, DCAA, TCAA, MBAA, DBAA and BCAA. All of these halogenated compounds are considered to be toxic, carcinogenic and/or mutagenic, in addition to being environmentally hostile because of their non-biodegradability and their bio-accumulation capability.

4. DISINFECTANT-DISINFECTION BY-PRODUCTS (D-DBP) RULE

The Disinfectant-disinfections by-products (D-DBP) Rule addresses complex and interrelated issues relating to disinfectants [9]. The risk of microbial disease outbreaks must be balanced against the risk associated with disinfectants and their by-products. Little is known about the occurrence of most DBPs, and the lack of knowledge regarding treatment effectiveness for DBP control complicates and inhibits regulatory decision-making. For these reasons the rule would be developped in two stages. The regulatory-negotiation (reg-neg) committee had set for stage I a maximum contaminant level (MCL) for total trihalomethanes THMs as 0.080 mg/L whereas an MCL for the total of five haloacetic acids (HAA5) was set at 0.060 mg/L. The five HAAs are monochloroacetic acid, dichloroacetic aacid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid. Stage I was promulgated in 1996. Stage II, which will reduce the above MCLs for THM's and HAA5 to 0.040 mg/L and 0.030 mg/L respectively. These lower MCL's were scheduled to be effective as of January 2002.

4.1 Hypochlorinating Agents Behaviour

4.1.1 Multistage Flash Desalination (MSF) Process

One of the most comprehensive studies that has been carried out on DBP's in SWCC plants of the eastern district was sponsored by SWCC and carried out as early as 1985 in coordination between RDC and RI (then known as the Research Institute and was affliated to UPM, now KFUPM) on MSF desalination streams of Al-Jubail, Al-Khobar and Al-Khafji plants [10]. In this study, more than 500 samples were collected from almost every stage in the desalination stream and analyzed for THM's and various kinds of DBPs. Sampling locations (Fig's 3. a, b and c), included the intake bay, chlorinated and unchlorinated intake fee, make-up seawater, brine recycle, brine blow down, desalinated product water, post-lime-treatment chlorinated product water, post-storage-tank chlorinated product water, as well as line-transmitted product water up to Riyadh HPT. In addition, several samples of on-site generated sodium hypochlorite, which was then used only by Al-Khobar plant as the chlorinating agents were also analysed. The study focused more emphasis on Al-Khobar (Al–Azizia) and Al-Jubail desalinaion plants.Within the main desalination streams of the former, starting from the intake feed to the unchlorinated product water, the study identified three groups of organic compounds at certain stages of both plant streams.

The first group consisted of two of the volatile trihalomethanes, namely, bromoform and chlrodibromomethane. However, these compounds, produced as a result of chlorine reaction with the organic constituents naturally occuring in the water were detected in all stages, only at concentrations well below the maximum contaminant level (MCL) set by the United States Environmental Protection Agency (USEPA).

The second group consisted of pthalate esters. Theese compounds were found in several product and other upstream water samples. Phthalate esters are extensively used as plasticizer in the plastic industry and also in the preparation of certain types of antifoaming agents. Although it is most likely that they might have emanated from one or more of these sources, the possibility of their formation as nonchlorinated by-produccts of chlorine oxidation of NOM may not be ruled out completely. The study identified two phthalate esters. These were benzyl butyl phthalate and bis (2-ethylhexyl) phthalate.

The third group comprised several chlorinated organic compounds. The source of these compounds was found to be the sodium hypochlorite used for chlorination of seawater. However these compounds which were detectable in seawater were completely absent from product water. Thus, they were not able to scape and carry over into product water, indicating that MSF is a very effective desalinating technique in removal of both volatile and nonvolatile organic contaminants that may form in the intake seawater.

Furthermore, the study spotted four additional halogenated organic compounds which were shown to have formed as a result of product water chlorination. The four organic contaminants were identified to be: a) 2- chlorocyclohexanol b) 2-bromocyclohexanol c) 1-bromo,2-chlorocyclohexane, d) 1,2-dichlorocyclohexane

Although, toxicity data of this last group of compounds are not available in the literature, yet on a structural basis, they may be anticipated to pose health hazards in a way similar to other halogenated hydrocarbon. The report stressed that the concentration of these compounds in Al-Azizia treated water were at all times higher than that of Al-Jubail water. They attributed this phenomenon to the fact that sodium hypochlorite was used for disinfecting the water at Al-Azizia plant, while the disinfectant then used in Al-Jubail plant was chlorine gas [10]. Based on their experimental results, the on-site generated sodium hypochlorite was found to contain many halogenated organic compounds in addition to the four cyclohexane derivatives. These latter findings were also published by Fayad and Iqbal elsewhere [11]. However, Jolley et. al.[12] discussed the presence of these latter compounds and considered them to be, most probably; artifacts attributable to cyclohexene preservatives in the methylene chloride used in the analytical process. Obviously, Jolley's interpretation doesn't seem to be founded since these compounds neither showed up in the extracts of the sampling stages preceding the product water chlorination stage, nor did they show up in the extract of analytical reagent grade sodium hypochlorite; even though the extractant used in all of these cases was the same methylene chloride. Moreover, in another study, Kutty et. al. [13] identified, among several other compounds, two of the above four compounds (namely c and d) in the hypochlorite header of all of Al-Jubail, Al-Khobar and Al-Khafji plants. In the latter plant, although compound (d) was identified alone, it was among several related compounds, e.g. monochlorocyclohexane. These findings of Kutty et. al. support the results of the RI studies. [10,11] However, none of these cyclohexane related compounds were identified in chlorinated product water in any of the stations of the latter study. In the study of Kutty et. al,. bromoform was the only THM which was common to all of the chlorinated seawater samples, as well as in the hypochlrite headers of all of the three stations as well. With the exception of Al-Khafji, traces of bromoform were also spotted in the chlorinated product water of Al-Jubail and Al-Khobar. However, in the Al-Khafji plant, low traces of bromoform were observed in both chlorinated and unchlorinated product water.

The second group of phthalate esters (PE's) of the RI investigation were identified in Kutty's study, only in the chlorinated seawater of Al-Khobar (dibutyl PE) and Al-Khafji plants (butyl dimethyl-hexyl PE) but not in Al-Jubail. Moreover, in contrast to what had been revealed in the Fayad and Iqbal study, these compounds were completely absent from product water. They were also absent from the hypochlorite solution headers of the three plants according to Kutty et. al. It should be noted here, that by the time of Kutty's study, all of the three plants had on-sit chlorine producing units installed.

4.1.2 Sea Water Rerverse Osmosis Desalination (SWRO) Process

Apart from some sporadic investigations here and there, very little systematic studies were reported in the literature with respect to DBP's formation potential, and precursors behaviour, in different stages of SWRO membrane processes. This is probably because in seawater reverse osmosis processes, rejection of DBPs, as exemplified by THMs behaviour, differs greatly with the nature, type and operation conditions of membranes used. In a Japanese study along the coast of Japan, a typical seawater, taken from a depth of 3 m, was treated with chlorine as sodium hypochlorite and allowed a 2 –4 hours contact time [14]. When this prechlorinated water was used as feed for a cellulose triacetate membrane, the THM's concentration in the feed water was 15 – 25 mg/L. As the feed seawater contained finite amounts of organic matter and relatively high levels of bromide, bromoform was the main THM generated with the addition of chlorine, as expected. Surprisingly, the THM's concentration in the permeate was 1.2 – 1.5 as much as that in the feed water. Thus, THM's were not only unremoved but, on the contrary, their concentration was even enriched on the permeate side of the membrane. On the other hand, when polyamide membranes were used instead, after dechlorinating the feed with SBS, THM's concentration in the permeate dropped to 0.1 – 0.2 of that of the feed water. That is, about 80 – 90% of the THM's were separated by the polyamide membranes and withheld in the brine side of the membrane. These THM's were neither found retained within, nor adsorbed to, the surface of the polyamide membrane [14].

Alternatively, when chloramine was used as the disinfectant, THMs were scarcely generated. In the chloramine-treated feed water, the THM concentration levels were found to be sometimes less than a microgram per liter (<1 mg/L), whereas their corresponding concentrations in the permeate of the cellulose triacetate membrane were obtained at a higher level of about a microgram per liter (~1 mg/L), [15]. Precursor levels in the feed seawater were reported [14] to be 69 mg/L for Br- ion, 0.5 mg/L as COD, and a chlorine residual of 0.3 mg/L for cellulose triacetate versus 0 mg/L for polyamide membrane (residual of 0.3 mg/L SBS dechlorinated ahead of cartridge filter). However, for RO permeate, the corresponding values of THM contaminant levels were not reported; probably indicating low concentration values, since their slots were left vacant in the report.

The above noted behaviour of THMs towards enriching the permeate side of cellulose triacetate membrane would probably find support in the studies carried out at the Yuma Desalination Test facility, where it quoted that THMs were present in RO feed, and that they are considered candidate RO membrane foulants [16].

Data obtained from other representative studies carried out in the R&D Center [17,18] had consistently showed extremely low levels of these organo-chlorine by-products. Similarly, data of years of periodic monitoring of DBP levels in MSF and RO product water from plants in the Kingdom, carried out by the chemical laboratories of the R&D Center, had also consistently confirmed those frequently undetectable organo-chlorine DBP levels. Representative samples of data obtained from the above mentioned R&D Center studies together with sample data of periodic monitoring of DBPs levels in MSF and RO product water of some plants in the Kingdom are shown respectively for THMs and HAAs, in Tables (3) & (4).

4.2 Alternative Disinfectants & DBPs

There is a large number of disinfectants that are salternative to chlorine. These alternatives include ozone, chlorine dioxide, chloramines, peroxone (hydrogen peroxide and ozone), CuSO4 salt, UV irradiation, to name just a few. Each disinfectant of these has, to a variable extent, some merits and some limitations. In some cases, the limitations outweigh the merits. Ozone has been in use for several decades in Europe, the late USSR Republics and USA. The inadequacy of ozone was mainly the interference by manganese and iron and, particularly, excessive formation of bromate and also bromoform in the presence of high levels of bromide. All of these drawbacks are in addition to difficulties in operating and maintaining the associated ozone generating equipment. These factors, coupled with lower cost, greater flexibility, and better reliability of chlorination versus ozonation, contributed to the somewhat limited use of ozone worldwide. In terms of DBP's, the use of ozone leads to excessive formation of the bromate oxohalide ion. This has been assigned an MCL of only 10 ppb by the USEPA.

Kutty and Al-Jarrah [19] considered the advantages and limitations of four disinfectants that they considered relatively safe for use with drinking water. Apart from chlorine, these disinfectants included chloramine, chlorine dioxide and ozone. These authors, and others, considered that although ozone is a very strong oxidizing and disinfecting agent, producing few THM's, removing taste and odour from treated water, its main drawbacks are high capital cost, incapability of deep penetration, as well as its inability to impart a residual protection against microbial aftergrowth in distribution systems [19,20,21]. Fig. (4) shows an example of an experimental UV unit installed downstream of an NF/RO train in Al-Jubail pilot plant [21].

However, Kutty et. al. [22] concluded that both ozone and chloramine as alternatives to chlorine may not be helpful in reducing THMs in bromide rich seawater. They reported that even chloramine which normally produces very low concentration of THMs in surface water samples was found to generate substantial amounts of THMS in the presence of high bromine in the source water. The study also revealed that MSF distillates do not contain appreciable amounts of THMs, but inferred that SWRO using less hydrophilic membranes such as cellulose triacetate, product permeate may be contaminated with THMs. The absence of THMs in MSF distillate was attributed to losses of both THMs and inorganics present in seawater during vacuum distillation [22]. Similarly, concentration levels of HAAs monitored in the product water of Al-Jubail MSF plants were found to range very much below MCL of 60 ppb [17]. These much lower HAAs levels, compared to their counterparts normally spotted in raw seawater, are apparently due to a significant loss of small halogenated and nonhalogenated organics during flash distillation [1,23].

Chlorine dioxide, ClO2, is a strong oxidizing compound which reacts rapidly with naturally occurring oxidizable organic and inorganic species. As a result, it becomes reduced to the more or less undesirable by-producs of chlorate, ClO3-, chlorite, ClO2-, and chloride, Cl-. In addition, like ozone it oxidizes bromide when preesent generating bromine, Br2, and bromate ion, BrO3-; and furthermore, in presence of humic material, it forms THMs, particularly bromoform and HAAs [24].

Some heavy metal ions have also been used as disinfectants. Examples include copper and silver ions. Nada et. al. reported on the application of copper sulfate as a disinfectant in a regional satellite SWCC plant [25]. The application of silver as a disinfectant, which is claimed to be a success, was also reported in the literature [1,26]. The claimed success of silver is due to the effective disinfection achieved at a concentration level of 10, which is well below the 80 ppb MCL limit of USEPA for Ag+.

The R&D Center has a long history of investigations regarding DBPs. This lead to an accumulated wealth of information and acquired experience. Comparative studies of different disinfectant levels, different precursors levels, different conditions affecting formation potential and other related optimization studies, were all carried out in the Center. This is reflected in the standadization, optimization and perfection of chemical methodologies pertinent to DBPs determinations at ultra trace levels, and frequently, in complex and difficult matrices. Research achievements as regards to DBPs can also be observed in several developments of creative modifications of known chemical methods to suit of the otherwise cumbersome chemical analysis, or in adapting already existing methods for some locally unique analytical situations.

Investigation of DBP loads were occasionally carried out at various points in desalination streams, in plants situated at various environmentally different sites. These included testing various possibilities of dose application points in relation to various sequential pre- and post-treatment stages. The results obtained from these studies further evolved into an adminstrative decision of weekly and monthly routine monitoring of these DBP's so as to ensure continued safety of the product water. The data that will eventually accumulate can, in the long run, serve as a useful database for evaluation, and decision making processes relating to the performance of relevant disinfecants.

5. CONCLUSIONS

1. Chlorination, at least at the present time, and possibly for quite sometime to come, remains the disinfection practice of choice. Different forms of hypochlorinating agent appear, on the whole, to be adequate for doing the job in most cases, despite minor privilges of some of them versus the others in certain applications.

2. None of the numerous alternatives seem to be a candidate possessing the desirable universal properties of cost, efficiency, etc. to currently replace chlorine.

6. RECOMMENDATIONS

1. *Although, a relatively massive work have been carried out on DBP's in MSF desalination processes, very few studies appear to have been carried out with respect to its analog SWRO processes. Further detailed studies in this respected are recommended.*

2. Also, the membrane industry is required to develop new chlorine-resistant membranes of equal, or even better performance levels and in suitable configurations, to replace current membranes that are sensitive to chlorine disinfectants.

3. One of the logical recommendations that the RI study proposed was the elimination of the chlorination step of product water preceding blending. Even though, THM levels are well below MCLs, they justified their recommendation pointing out that the step may elevate the low levels of DBPs unnecessarily since it will, anyway, be chlorinated again after blending.

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Table 1a. **Effect of temperature on HOCl % concentration at different pH**

	Percent HOCl						
pH	0°C	5°C	10°C	15°C	20°C	25°C	30°C
5.0	99.85	99.82	99.80	99.79	99.74	99.71	99.68
5.5	99.53	99.45	99.36	99.27	99.18	99.09	99.00
6.0	98.53	98.28	98.00	97.73	97.45	97.18	96.92
6.1	98.16	97.84	97.50	97.16	96.82	96.48	96.15
6.2	97.69	97.29	96.88	96.45	96.02	95.60	95.20
6.3	97.11	96.62	96.10	95.57	95.05	94.53	94.04
6.4	96.39	95.78	95.14	94.49	93.84	93.21	92.61
6.5	95.50	94.75	93.96	93.16	92.37	91.60	90.87
6.6	94.40	93.47	92.51	91.54	90.58	89.65	88.78
6.7	93.05	91.92	90.75	89.58	88.43	87.32	86.27
6.8	91.41	90.03	88.63	87.23	85.85	84.54	83.31
6.9	89.42	87.77	86.10	84.43	82.82	81.29	79.86
7.0	87.04	85.08	83.10	81.16	79.29	77.53	75.90
7.1	84.22	81.92	79.63	77.39	75.26	73.27	71.44
7.2	80.91	78.25	75.64	73.11	70.73	68.52	66.52
7.3	77.10	74.08	71.15	68.35	65.75	63.36	61.22
7.4	72.78	69.42	66.20	63.18	60.39	57.87	55.63
7.5	67.99	64.53	60.88	57.68	54.77	52.18	49.90
7.6	62.79	58.89	55.27	51.98	49.03	46.43	44.17
7.7	57.27	53.23	49.54	46.23	43.32	40.77	38.59
7.8	51.57	47.48	43.81	40.58	37.77	35.35	33.30
7.9	45.82	41.79	38.25	35.17	32.53	30.28	28.39
8.0	40.18	36.32	32.98	30.12	27.69	25.65	23.95
8.1	34.79	31.18	28.10	25.50	23.32	21.51	20.01
8.2	29.77	26.46	23.69	21.38	19.46	17.88	16.58
8.3	25.19	22.23	19.78	17.76	16.10	14.74	13.63
8.4	21.18	18.50	16.38	14.64	13.23	12.07	11.14
8.5	17.52	15.28	13.46	11.99	10.80	9.84	9.06
8.6	14.44	12.53	11.00	9.77	8.77	7.97	7.33
8.7	11.82	10.22	8.94	7.92	7.10	6.44	5.91
8.8	9.62	8.29	7.23	6.39	5.72	5.18	4.75
8.9	7.80	6.70	5.83	5.15	4.60	4.16	3.81
9.0	6.29	5.39	4.68	4.13	3.69	3.33	3.05
9.5	2.08	1.77	1.53	1.34	1.19	1.08	0.98
10.0	0.67	0.57	0.49	0.43	0.38	0.34	0.31
10.5	0.21	0.18	0.15	0.14	0.12	0.11	0.10
11.0	0.07	0.06	0.05	0.04	0.04	0.03	0.03
11.5	0.02	0.02	0.015	0.013	0.012	0.01	0.01
11.7	0.01	0.01	0.01	0.01	0.007	0.007	0.006

*Computer printout courtesy D. S. Cherry, N.C. State Univ., Raleigh, N.C.

Table 1b: Percent Molecular Chlorine, Hypochlorous Acid, and OCl-ion in a Water Solution Buffered from pH 6.5– 9.0 at 20 °C

	Solution Concentration (mg/liter)								
	5000			7000			10000		
pH	**Cl_2**	**HOCl**	**OCl^-**	**Cl_2**	**HOCl**	**OCl^-**	**Cl_2**	**HOCl**	**OCl^-**
6.5	.0063	92.28	7.71	.0088	92.28	7.71	.0126	92.28	7.71
7.0	.0017	79.10	20.89	.0024	79.10	20.89	.0034	79.10	20.89
7.5	.0004	54.84	45.51	.0005	54.49	49.51	.0007	54.49	45.51
8.0	.0001	27.46	72.54	.0001	27.46	72.54	.0001	27.46	72.54
8.5	.0000	10.69	89.31	.0000	10.69	89.30	.0000	10.69	89.30
9.0	.0000	3.65	96.35	.0000	3.65	96.35	.0000	3.65	96.35

Table 2: Effect of temperature on dissociation constant of hypochlorous acid

Temperature (°C)	0	5	10	15	20	25	30
$K_i \times 10^{-8.1}$ (moles/liter)	1.488	1.753	2.032	2.320	2.621	2.898	3.175

Table 3a: Individual and total THMs in blended potable water samples (mg/L)

SL	$CHCl_3$		$CHBrCl_2$		$CHBr_2Cl$		$CHBr_3$		Tot. THM
	Avg	Range	Avg	Range	Avg	Range	Avg	Range	Avg
1.	0.02	0-0.07	0.36	0-0.5	0.76	.7-0.8	2.42	2.4-2.5	3.56
2.	0.03	0-0.06	0.31	.3-0.4	0.85	.8-0.9	1.89	1.8-2.	3.08
3.	0.01	0-0.01	0.36	0-0.48	0.86	0-1	4.66	0-5.9	5.89
4.	0.02	0-0.05	0.29	.2-0.3	0.99	.9-1.1	7.2	5.6-8.5	8.52
5.	0.02	0-0.04	0.35	.3-.47	1.4	1.-1.9	9.1	7.8-9.7	10.8
6.	0.01	0-0.03	0.25	.2-.28	1.07	.7-1.22	8.54	7.7-9.7	9.8
7.	0	-	0.29	.25-.3	1.32	1.1-1.5	10.67	9.-12.	12.3
8.	0.24	.060.92	0.41	.1-0.74	1	.8-1.14	1.69	1.5-1.7	3.3
9.	0.45	0-0.9	0.82	.75-0.9	0.9	.86-.92	1.77	1.6-2.	3.9
10	0	-	1.51	-	1.63	-	2.3	-	5.4
11	0.03	0-0.09	0.46	.4-0.5	1.31	1.2-1.4	2.31	2.-2.5	4.1
12	0.03	0-0.1	0.44	.39-.5	1.3	1.2-1.4	2.29	2.1-2.5	4.06
13	0.04	0-0.1	0.42	.34-0.5	1.28	1.2-1.4	2.42	2.3-2.7	4.16

SL= Sampling Locations - Avg: Average

1=Jubail BS#1, 2=Jubail #2, 3=Jubail BS#3, 4 = Khobar BS, 5 = Dammam BS
6=Qatif BS, 7= Rahima, 8 = Khafji T#1, 9=Khafji#2, 10=Khafji City Tank
11= Riyadh TG-1, 12= Riyadh TG-2, 13=Riyadh TG-3

Table 3b: THM levels in some SWCC plants

S.No	Chemical Name	Unit	MPL	Sample Name			
				Shuqeaqe-1 (Akku)	Shuqeaqe-2 (Apha)	Farasan	Albirk
1	Chloroform	(ug/L)	200	ND	ND	ND	ND
2	Dichlorobormomethane	(ug/L)	60	0.2	0.2	0.3	1.6
3	Dibromochloromethane	(ug/L)	100	0.8	0.7	1.5	0.4
4	Bromoform	(ug/L)	100	1.7	1.4	12.4	6.5
5	TOTAL (THMs)	(ug/L)	*	2.7	2.3	14.2	8.5

MPL : Maximum permissible limit
ND : Not Detected (Less than 0.1 ppb)

* Note:

The sum of the ratios of the concentration of each THM to its respective guidelines value should not exceed1.

Table 4. Individual and total THMs in chlorinated well water used for Blending (µg/L)

Sampling location	$CHCl_3$	$CHBrCl_2$	$CHBr_2Cl$	$CHBr_3$	Total THM
Al-Jubail Area					
BS#1 WWST (a)	0.91	0.15	0.11	0.77	1.94
BS#2WWST	0.50	0.16	0.28	1.97	2.90
BS#3WWST	0.71	0.16	0.09	0.50	1.46
Al-Khobar Area					
Fowzia	1.62	0.32	0.41	2.92	5.27
Aqrabia (W)	1.84	0.41	1.12	6.17	9.54
Aqravia (E)	1.42	0.96	0.80	7.39	10.57
Tuqba (E)	1.27	0.34	0.40	5.60	7.61
Tuqba(W)	1.44	0.49	0.83	7.17	9.93
S.pmp station	1.08	0.27	0.64	4.75	6.74
N. pump station	1.27	0.32	1.25	8.13	10.97
Bandaria	1.17	0.29	0.61	5.76	7.83
Green belt	2.18	0.30	0.46	4.71	7.65
Al-Khobar tank	1.85	0.26	0.35	4.50	6.96
Oneza	1.94	0.18	0.76	16.41	19.29
Al-Saudia	1.28	0.26	0.19	3.74	5.47
Al-Raka	1.154	0.15	0.31	8.59	10.59
Dammam Area					
Well No. 78	4.66	0.0	0.33	3.27	8.26
Well No. 37	2.40	0.0	0.21	1.44	4.05
Well No. 76	2.84	0.0	0.93	5.22	8.99
Well No.91	3.51	0.0	0.07	2.65	6.23
Al-Jameen	4.55	0.16	4.16	13.60	22.47
Dammam WWS	0.0	1.02	3.51	11.10	15.62
Qatif WWST	0.0	0.0	0.02	0.18	0.20
Rahima	0.0	0.0	0.40	5.50	5.90
Riyadh					
HPT composite	0.03	0.53	1.14	2.98	4.68
Wasia composite	2.36	0.42	0.60	6.30	9.68

(a) WWST = Well water storage tank From Ref. 18

Table 5. Analysis of water samples for Haloacetic acid from Al-Jubail plant

HAAs	Product Water		Product water after lime CO_2 treatment		Blended water		Seawater*	
	Range (ppb)	Mean (ppb)	Range (ppb)	Mean (ppb)	Range (ppb)	Mean (ppb)	Range (ppb)	Mean (ppb)
CAA	ND	ND	ND	ND	ND	ND	ND	ND
BAA	ND	ND	ND	ND	ND	ND	ND to 3.1	1.1
BCAA	ND to 2	0.6	ND to 2.0	0.5	ND to 1.5	0.5	1.4 to 5.3	2.0
TCAA	ND to 3	0.8	ND to 2.6	0.8	ND to 2.1	0.7	1.6 to 2.6	2.1
BCAA	ND to 0.5	0.15	ND to 0.8	0.2	1.0 to 1.5	1.2	ND to 11	1.8
DBAA	ND	ND	ND	ND	ND	ND	ND	ND
Total HAA		1.55		1.5		2.4		7.0

* R-Cl_2 – 0.1 ppm, from Ref. 17.

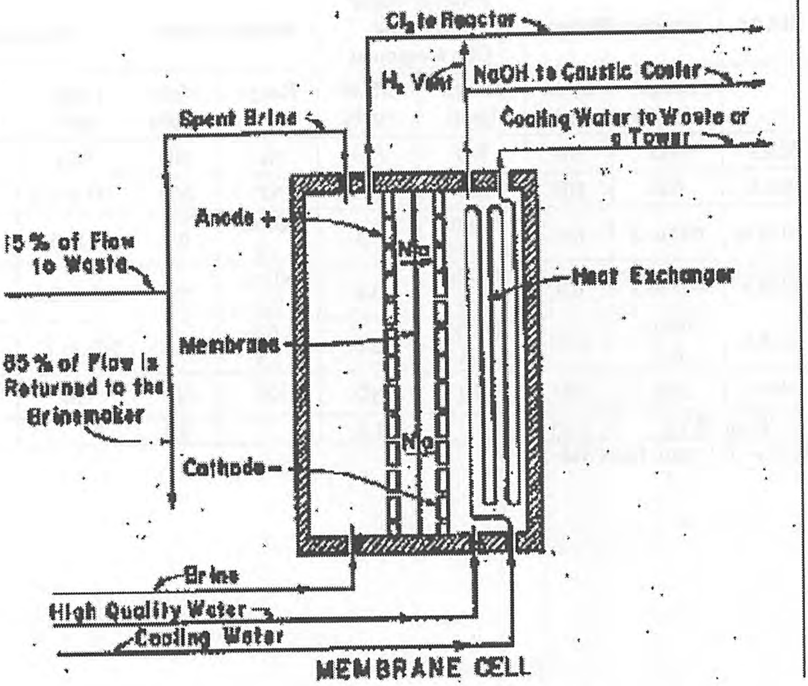

Figure 1a. Cloromat membrane cell with expanded electrodes

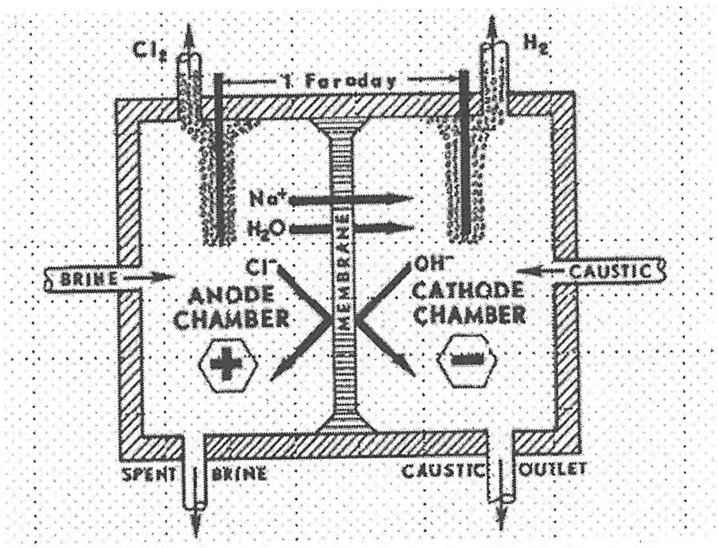

Figure 1b. The ideal membrane cell

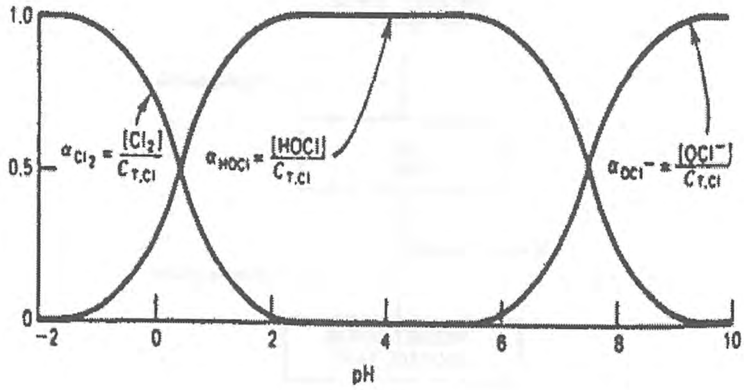

Figure 2. Distribution diagram for molecular chlorine, hypochlorous acid, and hypochlorite ion in water as a function of pH ($[Cl^-]=10^{-3}M$). (Source: Snoeyink and Jenkins, 1980)

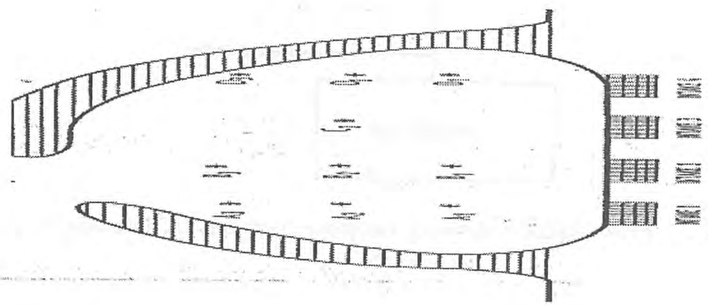

Figure 3a. Schematic diagram showing the sampling locations in Al-Jubail desalination plant intake basin

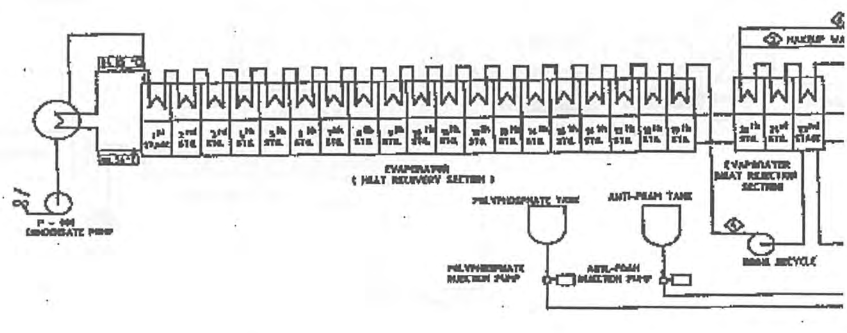

Figure 3b. Schematic diagram of a MSF desalination Plant showing the sampling locations ◊ 1to 5

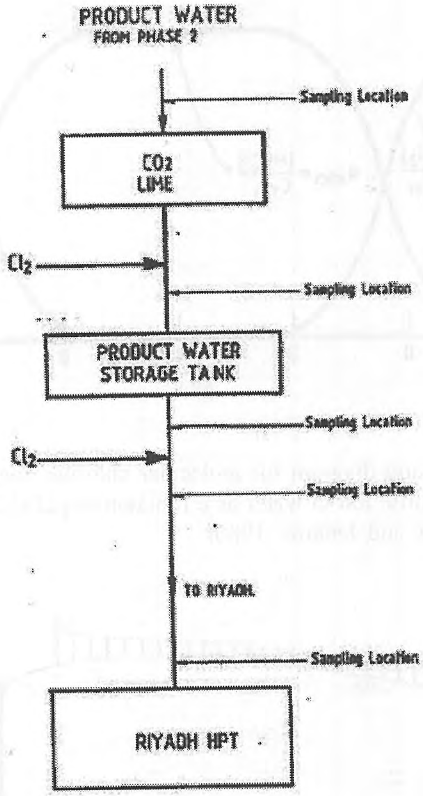

Figure 3c. Flow diagram showing sampling locations of desalination product water

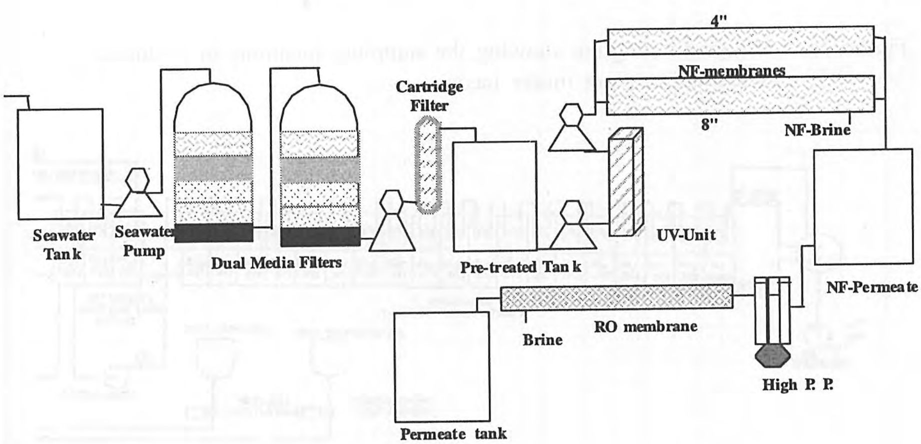

Figure 4. Applying UV-radiation at RSW Inlet and Before NF Membrane (from Ref. 21)

Prevention of losses through corrosion control-some case studies

Ismaeel N. Andijani and A. U. Malik

PREVENTION OF LOSSES THROUGH CORROSION CONTROL-SOME CASE STUDIES

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ABSTRACT

Annual losses resulting from corrosion in the desalination plants are estimated to be in the order of many millions of riyals. Corrosion can be broadly defined as reaction of an engineering constructional metallic material with its environment, leading to its loss or deterioration in properties of the metallic material. If potential corrosion problems are not resolved during early stages, they will arise later in the forms of corroded units and unexpected shutdowns. In such cases, high costs for repair or replacement of units as well as the cost incurred by the suspension of production during shutdowns will result. Additional costs are incurred due to the loss of efficiency of heat transfer surfaces and accidents resulting from component failure. In general, to minimize the losses, the material of construction must be carefully selected from a corrosion resistance standpoint. The design details should preserve the corrosion resistance of the materials. The material needed should be accurately ordered. The unit should be fabricated properly and adequately inspected to prove compliance with the specifications. The unit must be operated and maintained properly. Focusing on the above mentioned factors related to material selection and corrosion control in desalination and power plants, this paper presents some case studies which suggest solutions to corrosion problems based on type of corrosion. The case studies include failures of different components in SWCC desalination and power plants.

Keywords: failure analyses, weldment corrosion, SS 317L, galvanic compatibility, cupronickel, wear and tear.

INTRODUCTION

In desalination plants, the components used are subjected to the strong nature of different environments existing in the process, creating a number of corrosion related problems [1]. Losses resulting from these corrosion problems are estimated to be in the order of millions of riyals. The type of corrosion depends on both the material and the specific environment and operating condition to which the component is exposed. Therefore, to prevent the losses, the materials used must have adequate corrosion resistance and should be resistant to the corrosion environment. The design and environmental factors such as chloride ion concentration, electrical conductivity, and dissolved oxygen, velocity of flow, temperature and pH should be considered in the selection of materials [2]. The unit must be operated and maintained properly. There are many alternatives in solving corrosion problems. Whether the choice is corrosion-resistant materials or less expensive materials using cathodic protection, coatings, inhibition, etc., for protection, the basis for the selection requires the recognition and assessment of economic factors, as well as an understanding of corrosion technology. The cost of an anticorrosion alternative is not only the original installed cost, but it is all costs, including operating, maintenance, overhead, and various interim costs.

The failure of welded components in chemical, power and desalination plants and other engineering structures is a common problem. A number of factors are responsible for weld failures which include improper choice of welding technique, inadequate post weld treatment, use of unsuitable filler, incompatibility of weld and substrate materials, etc. [3]. In the welding of conventional stainless steels, one of the major problems is the precipitation of chromium carbide along the grain boundaries in the heat affected zone (HAZ). As a result, the zones around grain boundaries are depleted in chromium and are likely sites for corrosion attack in oxidizing and chloride bearing solution. Besides the precipitation effect, there are other welding defects such as adhesion defects, slag inclusion, and macro and micro fissures which are initiation points for corrosion attacks on weld of stainless steels, particularly in chloride containing environments [4].

In SWCC desalination and power plants weldment corrosion is directly or indirectly responsible for substantial cases of failures [5-9]. A 316L pipeline in a multi stage flash (MSF) plant was found to be corroded; the corrosion appeared quite prominent at the junction of two pipes joined by seam welds. The pipeline carrying product water remained stagnant for some time. Under the conditions of high chloride concentration and stagnancy, the pits were developed inside the pipe more at the vicinity of weld zone [5]. Leakage was observed from weld joints in a seawater reverse osmosis (SWRO) plant cleaning line made of 316L which remained idle for about 5 months after hydraulic testing [6]. Radiographic testing revealed the presence of cavities or voids in the metal and weldment/parent metal interface. Localized corrosion initiated as a result of the presence of a small stagnant pool of water containing chloride ions which served to open these cavities leading to final failure and leakage.

When dissimilar metals or alloys in a common electrolyte are electrically coupled (i.e. galvanic coupling) with each other, they experience galvanic corrosion. The extent of corrosion resulting from galvanic coupling is affected by (i) the potential difference between the metals or alloys, (ii) the nature of environment, (iii) the polarization behavior of metals or alloys and (iv) the geometric relationship of the component metals or alloys. The galvanic compatibility of different metals or alloys can be established by immersion testing of the galvanic couples in the environment of interest. Electrochemical tests like potential measurements, current measurements, and polarization measurements can provide information about the behavior of member metals or alloys in the galvanic couple.

Wear and corrosion are not easy to avoid in the majority of the industrial or manufacturing processes as they run at elevated temperature and increased heat can exacerbate both wear and corrosion. A variety of wear mechanisms can operate including adhesion (or sliding), abrasion, erosion, and fretting. Together wear and corrosion can lead to high economic costs and productivity losses, as worn and/or corroded equipment requires increased repair and maintenance [10].

This paper presents a few interesting case studies dealing with the failure analysis of components used in SWCC desalination plants. A case study investigating the failure of water rejection pipes of an RO plant has revealed long preservation in high salinity water containing formalin which made the water acidic. The results of the study confirmed that the failure in weldment is due to localized corrosion which resulted in the formation of pits and subsequently initiation and propagation of cracks. The second case study concerns the investigation of the compatibility of 90/10 Cu-Ni and 70/30 Cu-Ni tubes in a desal unit. The results showed that the use of 70/30 with 90/10 Cu-Ni is found to be acceptable in deaerated seawater. Another case study of the failure of the anode and diaphragm plate in an electrolyzer also indicated that the failure in the form of corrosion and erosion is basically wear and tear problems created by prolonged use of the equipment without sufficient maintenance. The methodology adopted and the analysis of material and corrosion products carried out is detailed. The issues that must be addressed in order to control corrosion are discussed.

CASE - I

FAILURE OF WATER REJECTION PIPE, SWRO PLANT

Mitsubishi Heavy Industries (MHI) installed SWCC Medina-Yanbu Seawater Reverse Osmosis (SWRO) plant in the year 1995. The plant has a total capacity of producing 5325 m^3/h water from its 15 trays. SWCC postponed the operation of the plant as the water transmission system was not ready. The SWRO plant and membranes were preserved using formalin as recommended by MHI. It started producing water in 1998.

The SWRO Plant started facing problems when leakages were noted at the weld joints of tubes at different locations. Leakages were observed in brine rejection, the drain pipe of the energy recovery pump (ERT) and other locations. Tubes were made of AISI 317 L SS. The Medina-Yanbu Plant engineers tried to stop the leakages by rewelding the pipe joints by using 317 materials, but in vain. MHI were called to find the solution to stop leakages. The MHI studied the problem and recommended tig welding using Inconel 625 as the new welding material.

PHYSICAL EXAMINATION

The following test pieces from RO pipes (marked # 1 to 4) were received:

1. Test pieces 1A and 1B (two samples)
2. Test pieces 2 (one sample)
3. Test pieces 3A and 3B (two samples)
4. Test pieces 4 (one sample)

The pipe # 1 leaked and was not repaired. The pipe # 2 leaked but was repaired temporarily by arc welding. The pipe # 3 did not leak and pipe # 4 has a new welding joint welded by tig welding using Inconel 625 welding rod. The photographs of the pipe samples are given in Figures 1 to 3.

The pipe samples were subjected to visual inspection. Some samples were found corroded at the weld and showed small and big pits and in some cases, cracks were also noted.

RESULTS AND DISCUSSION

The failure investigation of the welded pipe specimens from pipe # 1 to pipe # 4 was carried out by visual inspection, metallographic studies employing optical and scanning electron microscopy and EDX techniques.

Physical examination of specimens 1A and 1B from pipe # 1 which leaked but was not repaired revealed that sample 1A had corrosion at welding and there are few pits at locations near the weld. There is emanation of cracks from and near the pits. There is segregation of white constituents which are rich in chloride. The weld surface appeared to be severely corroded. It appears that long preservation in high salinity water containing formalin which made the water acidic resulted in the deterioration of the weld joints by initiation of localized corrosion. The localized corrosion resulted in the formation of pits and subsequently, initiation and propagation of cracks at the weldment. EDX studies indicated pit initiation occurring in the welding areas at locations wherever there is considerable concentration of chloride. Sample 1B from pipe # 1 was from a non-welded area and it did not show any corrosion or pitting. Sample from pipe # 2, (which was leaking and subsequently repaired temporarily by arc welding) on visual examination showed corrosion in welding along with some pitting. This is manifestation of corrosion in pipes rewelded by arc welding indicating the incapability of rewelds to withstand attack by seawater. The metallographic studies of the location near the weld indicated the presence of black or dark gray colored inclusions in the form of streaks or stringers. Closer study of the scanning electron microstructure indicated the presence of a microcrack emanating from the inclusions (Fig. 4). EDX studies indicated that these inclusions were rich in molybdenum and might be the secondary phases formed during the welding (Fig 5). SEM studies showed some evidence of the presence of cracks at or near the inclusions. In general, after repair, the weld appeared to function normally although there is some evidence of corrosion attack. Two samples 3A and 3B from pipe #3 (no leakage) were taken for investigation. Sample 3A showed indication of corrosion at the weld surface and there was a lot of pitting. This led to the conclusion that although there was no leakage, the pipe is likely to leak as the process of corrosion had already been initiated as evident by the presence of number of pits and corrosion at the surface. The sample 3B appeared to have sound welding and there was no evidence of pitting or corrosion by visual examination and metallographic studies.

The new weld joint point was welded by tig welding using an Inconel 625 welding rod (Sample 4). The visual examination of the welding surface and subsequent metallographic and EDX studies indicated that welding was sound and without any defect. The welding surface and the matrix showed typical dendrite and austenite structures, respectively.

CONCLUSIONS

The conclusions derived from the results of investigations carried out on pipes samples (No. 1 to 4) received from Medina-Yanbu SWRO plants are stated as follows:

1. Sample 1A from pipe 1 (leaking but not repaired) showed severe corrosion at the weld joint. The corrosion failure appeared to occur due to long preservation in high salinity water containing formalin which made the water acidic. The onset of localized corrosion resulted in the formation of pits and subsequently initiation and propagation of cracks.
2. Sample 2 which had leakage and was temporarily repaired by arc welding showed corrosion marks and pitting at the weldment, thus indicating the inadequacy of the weld to withstand high salinity water attack.
3. Sample 3 which did not leak showed indication of onset of corrosion and ultimately deterioration of the weld. This could lead to the possibility of leakage at the weld in due course of time.
4. In sample # 4, the joint welded by tig welding using an Inconel 625 welding rod was found to be sound without any visible defect.
5. The new joint welded by tig welding and an Inconel 625 weld appeared to be most appropriate for brine sub-header material for a SWRO plant.

RECOMMENDATIONS

It is advised to replace the existing weld joints of header pipes which showed poor performance due to inadequate corrosion resistance to brine by tig weld using Inconel 625 welding rods. This new welding type appeared to provide a sound weldment.

In the future, during preservation of SWRO pipelines, the contact of the water with formalin should be avoided as the mixture being acidic may corrode the metallic components including welds of the pipe line.

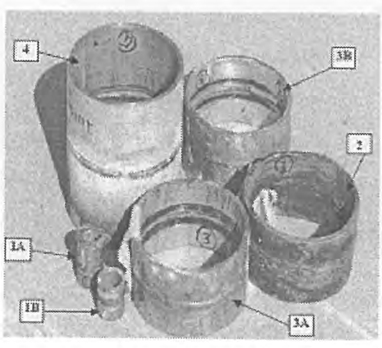


Figure 1: Photographs of the RO pipe samples from Medina-Yanbu RO Plant in as received condition

Figure 2: Photograph showing pipe # 2 (rewelded from outside)

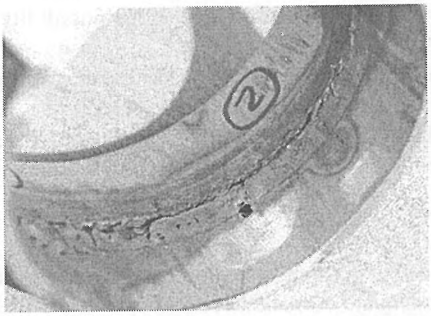


Figure 3: Photograph of the pipe # 2 (internal view) showing corrosion at the welding holes. Pits and cleavages can be seen

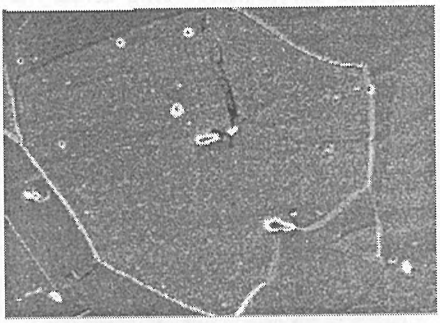


Figure 4: Scanning electron micrograph of the metal matrix near the weld in sample # 2 showing inclusions and microcrack emanating from inclusion 750X

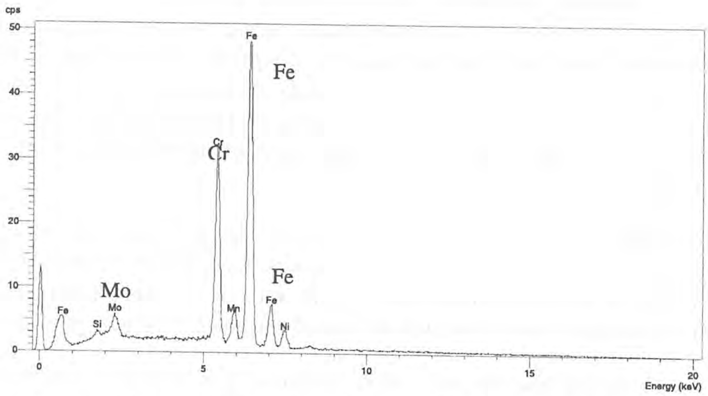


Figure 5: EDX profile of the inclusions from sample #2 showing high Mo content

CASE - II
COMPATIBILITY DETERMINATION OF 70/30 AND 90/10 CU/NI

The failures in the heat exchanger tubes of desalination Unit # 26 of Al-Jubail plant were determined by the Eddy Current Test. Failures were on the shell side at the tube sheet at both ends. A tube insert demonstration for approximately 157 tubes was held, and a concern rose due to the material used in this Demo (i.e. 70/30 Cu-Ni comparing with existing 90/10 Cu-Ni tubes). The performance of the inserted material could be observed after running the unit for some time for any chance of galvanic attack between both materials. This study is carried out to investigate the compatibility of 90/10 Cu-Ni and 70/30 Cu-Ni tubes in the Desalination unit # 26 of Al-Jubail Plant.

RESULTS AND DISCUSSION

The inserted 70/30 Cu-Ni and the existing 90/10 Cu-Ni tubes were analyzed by wet chemical method and Energy Dispersion Spectroscopy for their chemical composition. Their chemical composition is given below:

Elements	70/30 Cu-Ni	90/10 Cu-Ni
Cu	67.7	87.9
Ni	28.2	9.31
Fe	1.4	1.55
Mn	0.54	0.67

Free corrosion potential (vs. SCE) measurements with time were measured for both alloys in deaerated seawater at 85°C under dynamic conditions. The measured results are shown in Figure 6. The value of free corrosion potential for 70/30 Cu-Ni was in the range of -520 to -560 mV, whereas for 90/10 Cu-Ni it was -480 to -505 mV. This indicates that 90/10 Cu-Ni has a quite more noble potential than 70/30 Cu-Ni.

In general, when two dissimilar metals are coupled in an aqueous environment, noble metal does not corrode, but the active one suffers with an increase in its corrosion rate. While in this case the potential difference between 90/10 Cu-Ni and 70/30 Cu-Ni is small, in order to find out the possibility of any galvanic action between two alloys under these conditions, uncoupled potentiodynamic polarization curves were measured simultaneously for alloys in a single experiment. Curves were plotted as shown in Figure 7. The polarization curves can be used to predict the galvanic current passing when the two alloys are galvanically coupled. Intersection of the anodic polarization curve of 70/30 Cu-Ni and the cathodic polarization curve of 90/10 Cu-Ni gives the galvanic current value on the horizontal axis. The area ratio of 90/10 Cu-Ni to 70/30 Cu-Ni was 4: 1 and the galvanic current approximately 0.4mA/cm^2, as shown in Figure 8, which is taken from Figure 7.

Another experiment was carried out to determine the galvanic current of coupled alloys in deaerated seawater at 85° C under dynamic conditions with time (Fig. 9). The surface area ratio of 90/10 Cu-Ni and 70/30 Cu-Ni was 4:1. A maximum current of about 2.5 mA was measured which indicated the possibility of a very low galvanic action.

It is also worth noting that the tube sheet is also of Cu-Ni 90/10 cladding which is normally used for 90/10 as well as for 70/30 Cu-Ni tubes. Based on this, one can ascertain that the use of 70/30 inserts is quite safe.

CONCLUSIONS

(1) Free corrosion potential measurements of two alloys in deaerated seawater at 85° C under dynamic conditions indicate that 90/10 Cu-Ni has more noble potential than 70/30 Cu-Ni.

(2) Prediction of galvanic current from uncoupled potentiodynamic polarization curves was very low.

(3) The result of galvanic corrosion test showed low galvanic current for coupled samples of large 90/10 Cu-Ni (4 cm^2) and small 70/30 Cu-Ni (1 cm^2), indicating that the galvanic action, at these conditions, is not serious due to the small potential difference between them.

RECOMMENDATION

It is preferred that future MSF units' tube inserts are to be of the same material as the base tube material, yet the use of 70/30 with 90/10 Cu-Ni is found to be acceptable. Moreover, it is suggested to carry out chemical analysis for new tube inserts before their application in commercial plants.

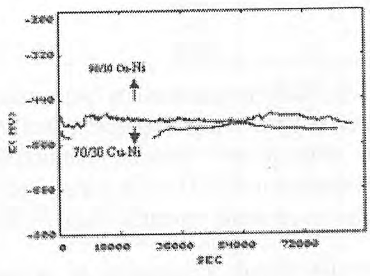


Figure 6: Free corrosion potential measurements with time for 90/10 Cu-Ni and 70/30 Cu-Ni in deaerated seawater at 85° C under dynamic condition

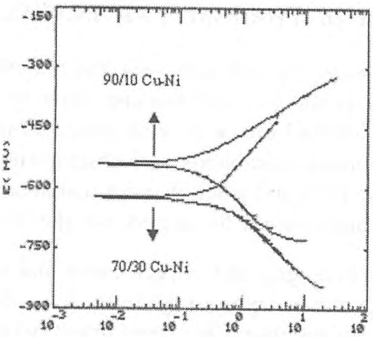


Figure 7: Potentiodynamic polarization curves of cupronickel alloys in deaerated seawater at 85° C under dynamic condition

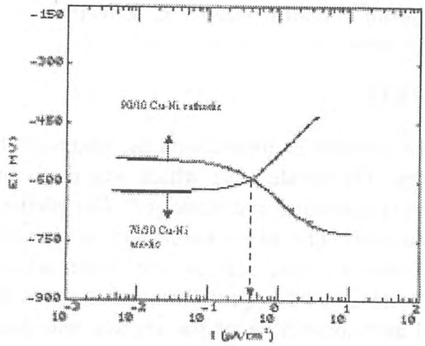


Figure 8: Polarization curves for cupronickel alloys (taken from Fig. 6 predicting galvanic corrosion current when 90/10 Cu-Ni is coupled with 70/30 Cu-Ni)

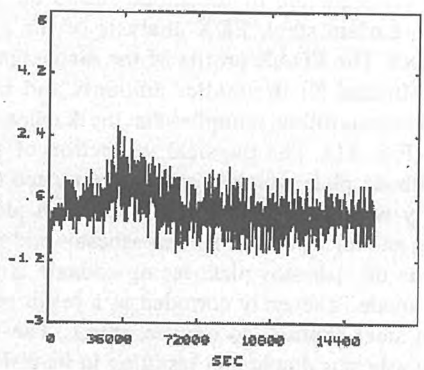


Figure 9: Galvanic current vs time for 90/10 Cu-Ni coupled with 70/30 Cu-Ni (4:1 area ratio) in deaerated seawater at 85°C under dynamic condition

CASE - III
FAILURE OF ANODE AND DIAPHRAGM PLATE IN AN ELECTROLYZER

Al-Jubail plants have two water electrolyzers, unit X01 and X02. Each unit has 10 cells which are connected in series and each cell has one diaphragm. The Anode plate in the unit X01 failed. The units have been in service since the inception of the Al-Jubail plant in the 1980's. Some corrosion problems were observed in 1990 and the maintenance of units was carried out in 1991 and since then no maintenance was done. As per the manufacturer, the maintenance should be carried out after 8-10 years.

The electrolyzer produces hydrogen and oxygen gases and each unit operates at 21V and 730 mA. As per design, the purity of oxygen gas should be 99.5 ± 0.2% and that of hydrogen gas 99.8%. Oxygen gas always has some impurity of hydrogen. If the impurity level exceeds 0.1%, the unit automatically gets tripped, but the safety detectors on both the product lines of oxygen and hydrogen gases were showing normal even though the impurity exceeded the set value.

Anode and cathode electrode materials contain 0.044 and 0.073% carbon respectively, where as diaphragm material contains 0.056% C. Except iron, no other element was found in a significant amount.

PHYSICAL EXAMINATION

The electrode plates were visually examined and the photographs of the damaged and sound portions were taken. The anode plate which was made of Nickel plated carbon steel was found extensively corroded and damaged. The photographs of the corroded anode are shown in Figure 10. The net- like carbon steel surface appeared after the nickel plating detached and the bare carbon steel plate was exposed. The cathode plate was found to be corrosion free apart from some spots due to deposition. The diaphragm was ruptured and about 50% of the surface was damaged. The rest of the diaphragm appeared to be in good condition. The diaphragm was ruptured at the top covering about 25% of the area.

DISCUSSION

The EDX profile of a cross section of the anode shows the presence of an outside layer of Ni coating on carbon steel, EDX analysis of the cathode shows that the electrode material is steel. The EDAX profile of the diaphragm material indicates the presence of Mg, Si, Mn and Ni in smaller amounts and Fe, K and O in higher concentrations. From the composition, it implies that the diaphragm material is a complex silicate like asbestos (Fig. 11). The physical inspection of the cathode and anode plates show that the cathode plate which is made of activated carbon steel has largely maintained its integrity whilst the anode and diaphragm plates are damaged. The diaphragm plate (facing anode) which is made of asbestos and fixed on a steel frame is largely torn out whereas the asbestos plate facing cathode is partially damaged. The Ni plated carbon steel anode is severely corroded as a result of wearing off the nickel plating leaving carbon steel exposed to oxygen attack. The corrosion of the anode also affected the facing asbestos diaphragm resulting in its disbondment. The damaged areas of anode and the facing diaphragm are nearly at the same locations. The cathode plate is largely intact and only appears to be affected slightly whereas the facing diaphragm is considerably damaged.

CONCLUSIONS

The corrosion of the anode plate is due to wearing off the nickel plating resulting in the direct exposure of oxygen gas to the bare carbon steel plate. The CS plate corroded in the presence of oxygen.

(i) Damage to the asbestos diaphragm is due to wear and tear with time and wearing off the nickel plating on the anode could also be partially responsible.

(ii) The cathode plate made of activated carbon steel by and large remained unaffected by corrosion which is due to the integrity of the activated carbon steel material.

RECOMMENDATION

The failure of anode and diaphragm plates in the form of corrosion and erosion is basically wear and tear problems created by prolonged use of the equipment without sufficient maintenance. It is therefore recommended that the existing unit be replaced with a new water electrolyzer assembly and proper and regular maintenance should be practiced for a trouble free operation and efficient performance.

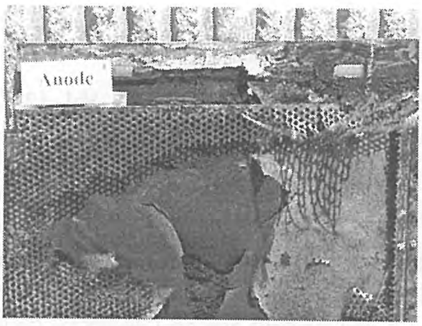
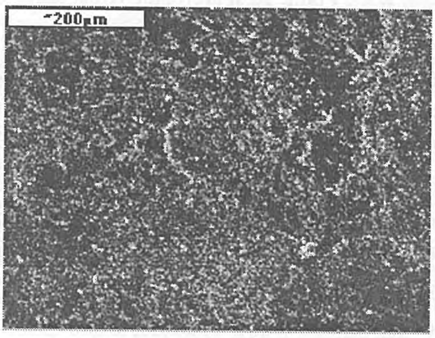


Figure 10: Photograph of the corroded anode plate



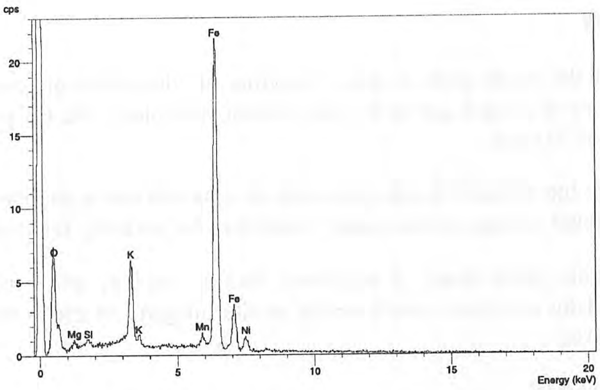


Figure 11: EDAX profile of asbestos diaphragm

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Treatment of Wastewater Primary Effluent Using the Microfiltration Technique

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TREATMENT OF WASTEWATER PRIMARY EFFLUENT USING THE MICROFILTRATION TECHNIQUE

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ABSTRACT

The implementation of the microfiltration (MF) technique for wastewater treatment is expected to produce a stable quality of treated wastewater. Product water could be used safely for greenery and irrigation or indirect human uses. Moreover, it will eliminate many conventional unit processes and, consequently, will reduce the cost of wastewater treatment drastically. This paper discusses the evaluation of MF as a treatment technique for treating wastewater primary effluent for reuse under prevalent conditions of Kuwait. This paper also outlines the experimental activities that were carried out during the period of this study.

Keywords: Conventional, primary, bacteria, membrane, pollutants

Introduction

Continued urbanization, population growth and the expensive process of producing potable water lead to the necessity of finding other water resources that can be utilized to handle this expansion. Treated wastewater effluent is the only increasing resource that is being generated, due to the high consumption of potable water desalinated by the multistage flash (MSF) technology, so it is regarded as perennial surface water in Kuwait that is an economic source available for extensive and appropriate use. However, this extremely important source of water is met with alarmist beliefs that have delayed its proper utilization. Its usage is also hindered by the incomplete elimination of contaminants and the lack of a distribution network. Therefore, the characteristics of its danger have constrained decision-makers in renovating the standard wastewater treatment with new technologies, such as semipermeable membranes, for further reclamation that can meet a variety of human needs.

Treated wastewater has been considered for use in almost every country in the Arabian Gulf region, but it has special importance in maintaining and increasing the water resources of Kuwait, as well as ameliorating the deterioration in the quality of the country's natural environment. It is a tremendous resource that can be used to compensate for the shortage of natural water resources, and will help in the development of Kuwait's future water resource plans. Its utilization varies between the present and the future, based on advances in treatment. Current uses include irrigation water, process cooling water, boiler feedwater, drinking water and ultrapure water with minimal contaminants.

Proper treatment of municipal wastewater alleviates surface water pollution problems, and not only conserves valuable water resources, but also takes advantage of the nutrients contained in sewage to grow crops. The availability of this additional water near population centers will increase the choices of crops that farmers can grow. The nitrogen and phosphorus contents of sewage might also reduce or eliminate the requirements for commercial fertilizers.

Many countries have included wastewater reuse as an important dimension of water resource planning. In the more arid areas of Australia and the United States of America (USA), wastewater is used in agriculture, releasing higher quality water supplies for potable use. Some countries, for example, the Kingdom of Jordan and the Kingdom of Saudi Arabia, have a national policy to reuse all treated wastewater effluents and have already made considerable progress towards this end. In China, sewage use in agriculture has developed rapidly since 1958, and, now, over 1.33 million hectares are irrigated with sewage effluent [1]. It is generally accepted that wastewater use in agriculture is justified on agronomic and economic grounds, but care must be taken to minimize adverse health and environmental impacts.

The principal objective of wastewater treatment is the reduction of pollutants (solids, and dissolved organic and inorganic compounds) and organisms (pathogenic agents such as bacteria and parasites). To fulfill this objective, a variety of processes and technologies are available. The collected wastewater is transported to a treatment plant, and the treated effluent is returned to a receiving water body or is used for irrigation or other purposes [2].

Different types of treatment processes are available in the field of wastewater treatment. They are usually classified into two major treatments, i.e., conventional treatment, which includes preliminary, primary, secondary and tertiary treatment, and nonconventional treatment, which includes membrane separation such as reverse osmosis (RO), ultrafiltration (UF) and microfiltration (MF).

Membrane technologies such as RO, UF and MF have developed extensively in recent years. The membrane separation process is applied in a wide variety of fields, varying from the chemical industries to the water industry.

Some of the main advantages of the membrane technology are as follows [3]:

- Energy saving: Energy consumption is low since no phase change is required for processing.
- Raw-material recovery:
- Valuable products can be recovered for re-use or sale, and
- Both the concentrate and the permeate streams may be usable.
- Low floor space requirements for system.
- Expansion: The modular character of membrane system designs makes it simple to plan a system to meet present needs, while providing for future expansion.
- Automation: Many systems can be instrumented to automatically start, stop, or begin a cleaning cycle, and can be installed to shut down automatically in the case of pH, pressure, or temperature problems.
- Environmental regulation: Membrane technologies can provide waste treatment that meet or exceed regulatory requirements.

Kuwait has three main wastewater treatment plants at Riqqa, Jahra and Sulibia. Riqqa and Jahra work on the principle of the activated sludge process at the tertiary level whereas, Sulibia works on the principle of advanced treatment using UF and RO.

The project was conducted at the Riqqa Wastewater Treatment Plant, which is the second largest sewage treatment facility in the State of Kuwait; its design capacity is 180,000 m^3/d, and it serves a population of 220,000. The plant was constructed in 1982, and upgraded in 1995.

Wastewater treatment at the Riqqa plant is accomplished through three stages: primary, secondary, and tertiary. In the primary treatment, i.e., physical treatment, wastewater is first received at the headwork's building for treatment, where floatables and debris are removed using bar screens and grit sedimentation. The secondary treatment i.e., biological treatment, is the main treatment stage and is divided into two stages:

- Aeration basin: Twelve aeration basins are connected to eight air compressors through a network of headers extending down to connect to fine-bubble diffusers.
- Clarifiers: In ten clarifiers, clear water flows through weirs leaving sludge to precipitate at the bottom and be pumped out either to aeration tanks as return activated sludge or to sludge handling and disposal facilities.

In principle, MF is attractive because it promises high flux at low pressure. However, MF is susceptible to fouling and the inability to retain submicron species. Methods to alleviate fouling are being developed, including backwashing, such as the Memtec air backwash and the back shock technique [4].

MF to less than 0.2 μm using modern membrane technology has the ability to reduce the biological oxygen demand (BOD), chemical oxygen demand (COD) and bacterial count in one pass. An MF unit acts as a barrier filter. At no time in its operation can particles less than 0.2 μm pass through to the product stream. Bacteria sizes range from 0.3 μm upward and, so, are removed. Viruses are generally smaller than 0.2 μm, but are associated with host bacteria. It is expected that 99.9% of virus removal will be achieved [5].

The study was aimed to evaluating the efficiency of microfiltration (MF) in treating primary wastewater effluent under the prevalent conditions in Kuwait. This paper also outlines the experimental activities that were carried out during the period of this study.

General Description of MF Unit

The MEMCOR 20 M10 unit is a microfiltration machine designed to remove impurities larger than 0.2 micrometers from feedwater.

Machine design is compact and consists of filtration modules, a circulation pump, associated valves, pipe work, instrumentation and a control system, all mounted in a stainless frame. Installation of required unit electricity, feed supply, compressed air, drains and filtrate pipe work to the termination points on the machine.

The filtration modules are bolted together with end blanking plates to which unit pipe work connections are made. Each module has a nominal membrane filtration area of 10 m^2, supplying a total nominal filtration area of 20 m^2. Acrylic end plates are fitted at the top and bottom of one module to allow visual monitoring of machine operation. A feed pump drives the liquid to be filtered into the filtration modules, either from the break tank or from an external tank.

The process valves are ball, butterfly or diaphragm valves, most fitted with pneumatic actuators. The actuators are operated by pressurized air distributed by electrically operated pilot solenoid valves mounted on the frame, near the control frame.

A programmable logic controller (PLC) mounted in the control cabinet controls the pilot solenoid valves and pump operation. The PLC also monitors various control switches and other inputs and illuminates the appropriate indicator lamps during machine operation.

Three pressure transmitters with LCD combined analogue and digital displays are used to monitor system operating pressures. The LCD indicators will flash if the operating pressure exceeds 600 kpa. Flow meters are used to indicate feed and filtrate flow rates.

Feed water for the unit was drawn from the unchlrionated primary treated wastewater canal at a depth of 1.2 m from the surface. An intake structure consisting of submersible feed pump supported by a steel clamp was connected to the system. The MF was operated in direct flow (dead-end) mode. After passing through a coarse strainer (size 150 μm), the treated primary effluent was fed to a break tank. From the break tank, the feed stream with a feed pressure of 1.8 bar passes from the outside of the membrane (from the module shell) into the center (lumen) and exits as filtrate. Suspended solids and microorganisms build up on the outside surface of the hollow fiber. This results in an increase of driving pressure with time. The membranes will be backwashed periodically to remove most of the contaminants from the outside of the fibers, therefore, improving productivity. In this backwash period, the contaminants will be first dislodged from the surface by an air pulse (6 bar) from the inside of the fibers. Then, the contaminants will be flushed out by a sweep of feed water to the recalculating line to the feed. The MF system uses an air backwash stream to clean the hollow fiber membranes. The backwash is automatically controlled by a programmable logic controller (PLC). Air at high pressure will be injected into the center of the hollow fibers and bursts through the membranes, removing the foulants that have accumulated on the membrane.

The MF was operated in direct flow mode during this test. Fig. 1 presents the MF process system. Table 1 presents the average operating parameters of the system for both tests.

Table 1: Average Operating Parameters of the System.

Parameter	Feed	Filtrate
Flow rate (m^3/h)	1.0	1.0
Pressure (kPa)	179	179
TMP (kPa)	90	90
Silt Density Index	>6.0	4.38

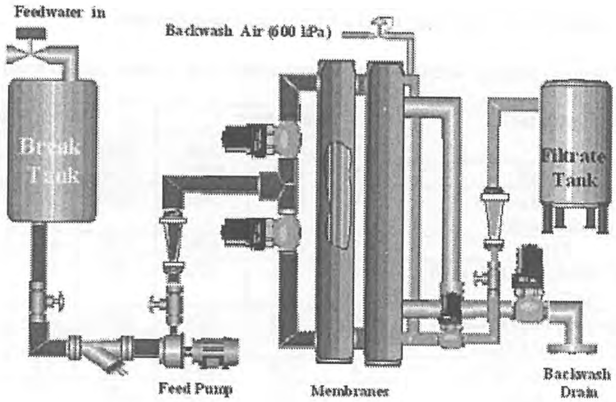

Fig. 1: MF process system

Rustles and Discussions

Primary Effluent as Feed to an MF System.

The results of the chemical and biological analyses of 36 samples of feed and filtrate water from MF units using primary treated wastewater effluents as feed are summarized in Tables 2 and 3.

The average removal efficiencies of the MF unit in improving, BOD, COD, total bacterial count and fecal coliform for primary treated wastewater effluent were 69.4, 26.55, 97.51% and 99.15% respectively.

It can be seen from Table 2 that MF is capable of reducing the average values of TSS and turbidity of the feedwater from 159.5 to 31 mg/l and from 161 to 11.5 FTU respectively. The removal efficiency of MF in removing TSS was 80.9%, and the removal efficiency for turbidity was 93.4%.

Figs 2, 3, 4 and 5 show graphical presentation of the MF performance in reducing the values of BOD, COD, turbidity and TSS respectively, when using Primary effluents.

Table 2: Biological analysis of effluent and filtrate water using primary wastewater as feed to MF unit.

Parameter	Feedwater			Filtrate water		
	Max	Min	Ave	Max	Min	Ave
BOD_5 (mg/l)	97.6	73.37	85.48	56	39.93	47.96
COD (mg/l)	198	122	160	159	58	108.5
Total Bacterial Count (Heterophic) (coloni/100ml)	9.0E+09	3.0E+09	6.0E+09	9.0E+09	2.9E+06	4.5E+09
E-coli (colony/ 100)	2.8E+08	3E+06	1.4E+08	5E+07	7E+06	2.8E+07
Salmonella. (coloni/ 100)	0	0	0	0	0	0
Fecal Coliform Bacteria (colony/ 100 ml)	5.0E+06	0	2.5E+06	1.16E+05	0	5.8E+04

BOD = Biological oxygen demand; COD = Chemical oxygen demand;

Table 3: Chemical analysis of effluent and filtrate water using primary treated wastewater as feed to MF unit.

Parameter	Feedwater			Filtrate water		
	Max	**Min**	**Ave**	**Max**	**Min**	**Ave**
TDS mg/l	770	657	713.5	750	650	700
Ec (electrical µs/cm conductivity)	1589	1337	1463	1595	1321	1458
PH	7.05	6.87	6.96	7.53	6.81	7.17
Total Alkalinity(T.ALK) mg/l as $CaCo_3$	196	144	170	194	170	182
Total Hardness mg/l as $CaCo_3$	228	220	224	220	206	213
Ca^{2+} mg/l as $CaCo_3$	82	76	79	78	72.8	75.4
Mg^{2+} mg/l as $CaCo_3$	11.4	7.29	9.34	10.5	5.8	8.15
So_4^- mg/l	350	250	300	235	190	212.5
Sio_4^- mg/l	12.7	8.4	10.55	8.4	6.3	7.35
Cl^- mg/l	238	230	234	238	230	234
Turbidity FTU	212	110	161	17	6	11.5
Free chlorine residual mg/l	0	0	0	0	0	0
TSS mg/l	254	65	159.5	48	5	31
NH_4-N mg/l	28	22	25.5	27	20	23.5
Barium mg/l	1	1	1	1	1	1
Fluoride mg/l	0.4	0.2	0.3	0.34	0.11	0.22
DO mg/l	1.0	0.7	0.85	2.5	1.9	2.2
PO_4^- mg/l	15	14	14.5	14	13.3	13.65

TSS = Total suspended solids; BOD = Biological oxygen demand

Operational parameters, such as temperature, feed pressure, filtrate pressure, flow rate and trans-membrane pressure, were monitored and recorded three times a day. In general all membrane processes the higher temperature will result in greater flux. At the Riqqa plant, the temperatures of the feed water for the system were in the range of 24 to 32°C, during the project's trial period. There was no significant change in temperature from day to day. Therefore, the effect of temperature on flux was almost negligible.

Performance Evaluation of the MF System

The MF unit was operated in constant permeate flux mode for both tests in which the transmembeane pressure increased over time as materials deposited on the membrane. Permeate flux was set at 1.0 m^3/h and readjusted as needed to maintain this flow. The unit was operated in direct flow mode. During direct flow filtration, all water entering the membrane module exists as permeate. The primary treated wastewater contained impurities, which resulted in an accumulation of impurities on the surface of the membranes during operation. Thus, the flux decreased with time, the filtrate pressure decreased and the transmembrane pressure increased. Fig. 6 shows the increases (i.e., from 25 kPa to 80 kPa) in transmembeane pressure over time. Fig. 6 shows the transmembeane pressure performance during the primary treated wastewater effluent test.

Fouling in MF manifests itself as a decline in flux with time of operation. Two major parameters affect rate of fouling in MF (i.e., quality of the feed, backwash time interval).

Quality of the Feedwater: One of the factors causing membrane fouling is the quality of the feedwater. In this study, the SDI and turbidity were used as indicators for feed water quality. The effluent produced at the Riqqa Wastewater Treatment Plant varies according to the plant's condition. The SDI of the feedwater measured was high (over 6%), but could not always be measured. The results showed that when the MF unit was fed with water with a high SDI, a high SDI was also measured on the filtrate side. Fig. 7 shows that the average SDI value during the study was 4.38%.

Backwash Interval: To improve the productivity of the MF unit, regular backwashing of the membranes must be carried out to dislodge and remove foulants from the membranes surfaces. To investigate the effect of the backwashing on the membrane flux and productivity, an experiment was performed. In this test, the backwash sequence has been set to 18 min and TMP was monitored and recorded on an hourly basis. Fig. 8 presents the performance of the TMP during this experiment. Fig. 8 shows the improvement of the TMP after each backwash.

Membrane Chemical Cleaning

Chemical cleaning of the MF unit becomes necessary if:

- The filtrate flow rate declines and cannot be restored by backwashing.
- There is an extended shutdown of the unit.
- The transembrane pressure increases to 80 kPa or above.

The recommended chemical used for cleaning MF membranes is a solution of 1% Memclean EX-A2 and 1% caustic soda (NaOH). This solution removes biofouling, and is then followed by another cleaning with 0.5% citric acid to remove metal fouling.

The unit was cleaned twice during the whole operation using the primary treated wastewater effluent. The cleaning was performed, as recommended by the MF manufacturer, when the transmembrane pressure reached 80 kPa to restore the performance and to avoid fouling of the membranes. After cleaning, the transmembrane pressure was improved to 25 kPa using chemical agents, as shown in Fig. 6. The chemical solution was very effective in removing some of the fouling such as iron, nitrite and chloride, as shown in Table 4.

Table 4: Chemical analysis of the chemical solution used for cleaning MF before and after.

Parameter	Chemical Solution Before Cleaning	Chemical Solution After Cleaning
Total Fe(mg/l)	0.05	1.36
So_4^- (mg/l)	0	4.0
No_3^- N (mg/l)	8.6	26.1
SiO_2^- (mg/l)	5.0	22.9
Cl^- (mg/l)	41.2	1242
NH_3^-N(mg/l)	0.22	9.0

Evaluating Water Quality

MF improved the color of the feedwater. The results revealed that MF can also significantly improve the turbidity of the primary treated wastewater effluent. The pH values of the primary effluent were approximately constant at 6.96 during the operation of the system. The chemical characteristics of the MF filtrate water confirm its suitability for irrigation purposes because all of the relevant values lie within the permissible limits set by Food and Agriculture Organization (FAO) as presented in Table 5.

Table 5: Characteristics of treated wastewater effluent used for irrigation in Kuwait

Parameter	Permissible level	MF Filtrate level
pH	6.5-8.5	6.5 -7.2
TDS (mg/l)	1500- 4000	690
TSS (mg/l)	20.0	10
BOD_5 (mg/l)	15.0	8.0
COD (mg/l)	100	14
DO (mg/l)	>2	>2
NH_3 (mg/l)	15	1.37
Barium (mg/l)	2	0.5
Fluorides (mg/l)	25	1.1
PO_4 (mg/l)	30	19.2

* After. Kuwait Alyoum, 2001

Conclusions

Based on the results of this study, the following conclusions have been made:

- The chemical analysis revealed that the MF system significantly improved the quality of the primary effluents, with an average SDI 4.38 %.
- There were consistent reductions in BOD, COD, TSS and total bacterial count.

- The overall results confirm that microfiltration can be operated efficiently on municipal scale to consistently and reliably produce a highly clarified water of a quality suitable for reuse. Therefore, water produced from a MF system is considered to be safe for use in agriculture, industry and for indirect human uses.

Acknowledgement

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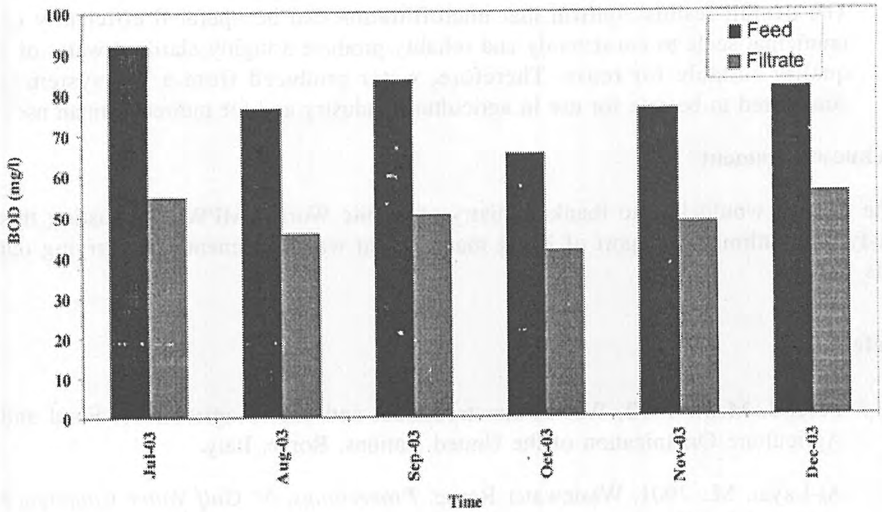

Fig. 2: Average monthly BOD of feed and filtrate water versus month.

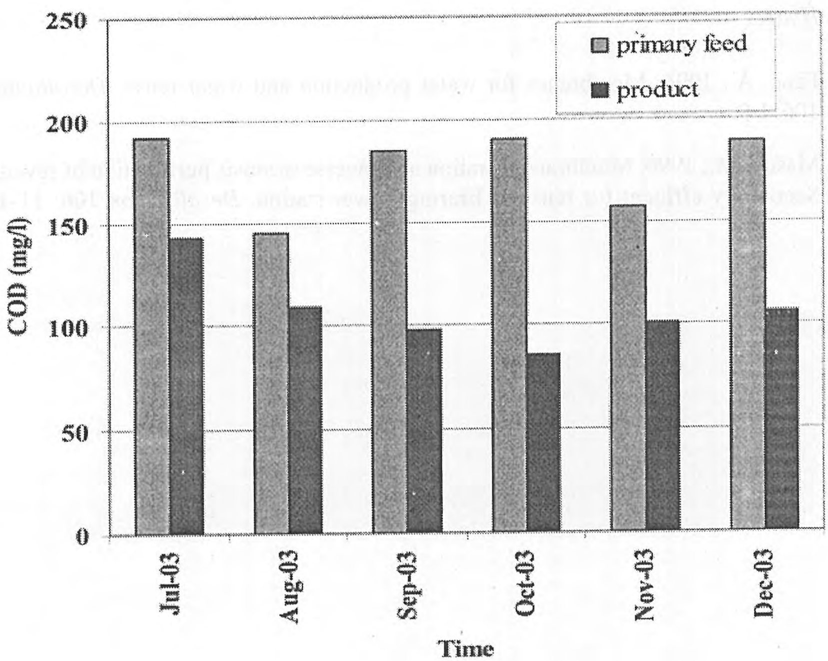

Fig. 3: Average monthly COD of feed and filtrate water versus month.

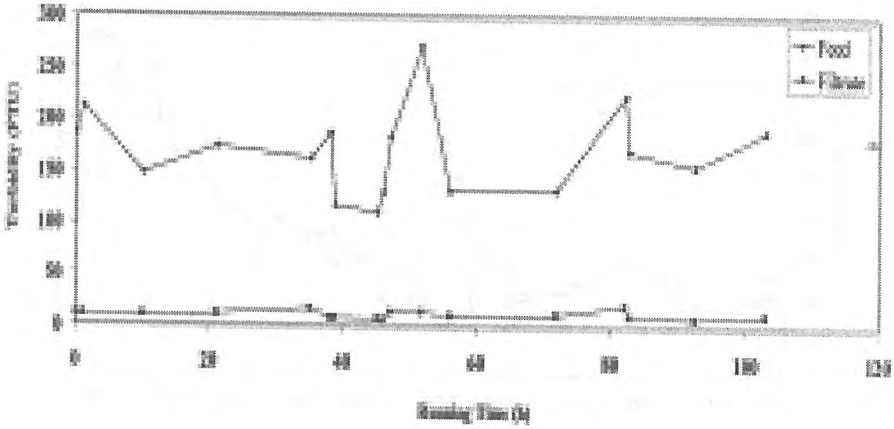

Fig. 4: Turbidity of feed and filtrate water versus running time.

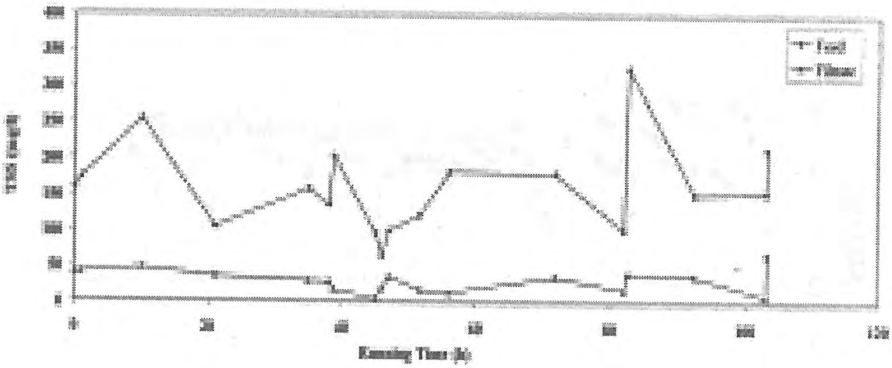

Fig. 5: TSS of feed and filtrate water versus running time.

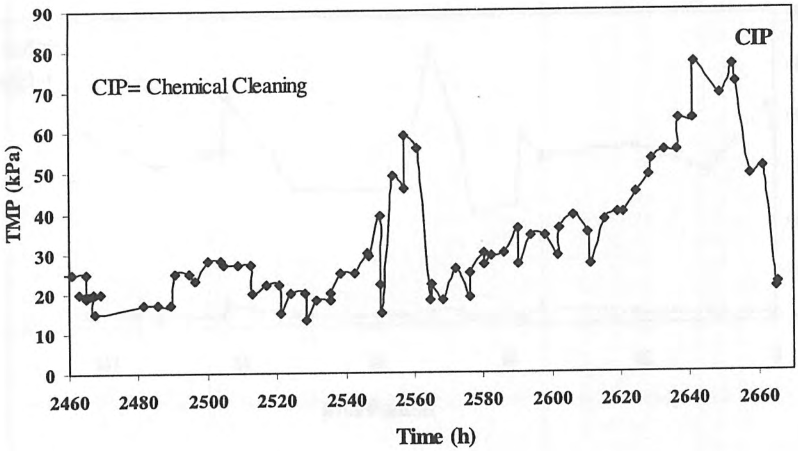

Fig. 6: TMP versus running time.

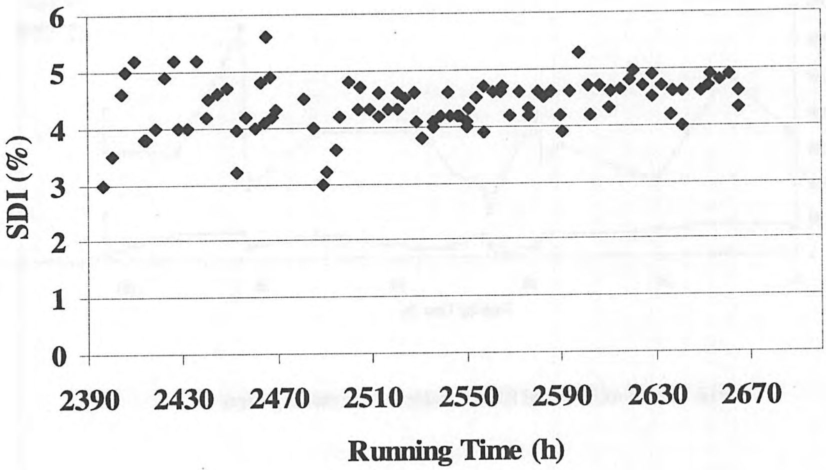

Fig. 7: SDI of filtrate water versus running time.

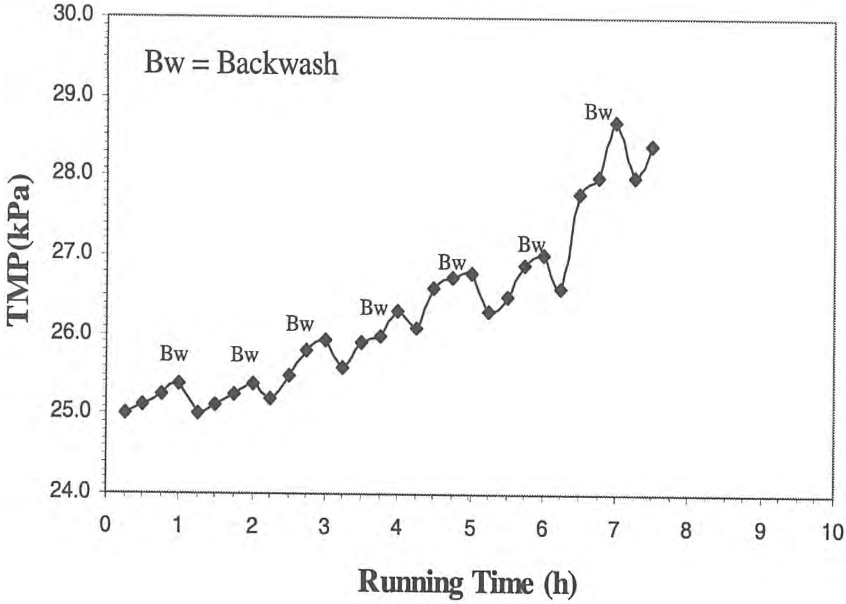

Fig. 8. TMP versus running time: (backwash intervel 18 min).

An online Cleaning System to reduce demister fouling in MSF Sidi Krir Desalination Plant, 2 x 5000 m3/Day

Hassan E. S. Fath and Mohamed A. Ismail

AN ONLINE CLEANING SYSTEM TO REDUCE DEMISTER FOULING IN MSF SIDI KRIR DESALINATION PLANT, 2 X 5000 M3/DAY.

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ABSTRACT

This paper discusses the demister scaling issue in (MSF) 2 x 5000 m^3/day Sidi Krir Desalination Units site, and how this issue affects directly the performance of this system and the production rate (which decreases gradually to 50%), in addition to the unscheduled outage costs for acid cleaning or replacement of these demister sheets by new ones. Fixing of an online cleaning system during operation was proposed using the flow of the condensate brine heater discharge pumps through two headers which were fixed at both lateral sides of those stages in which the scaled demisters were recorded. The proposed system will maintain the performance and saves the costs of chemical cleaning and the unscheduled outage of the system. The technical and economical advantages of this proposal are highlighted.

INTRODUCTION

The Sidi Krir power plant (2 x 325 MWe. Gas/oil Fired Steam Generators), located on the Mediterranean Sea (West coast of Alexandria, Egypt) has a 2 x 5000 m^3/day MSF distillation unit of the brine recirculation type. More plant specifications are given in Table (2). Scale formation occurrence on the demister sheets in the evaporator flashing chamber during operation is highlighted in this paper. Such phenomenon occur due to disturbances in some parameters (such as brine level, anti-foam injection rate and concentration factor, etc.), causing a drop in all target values of operation parameters and consequently in performance of the system as a whole see Table (1). The demister fouling was observed in stages (1-8), where the density scale gradually decreases toward stage (8). In stage (1), the inlet jet of the condensate brine heater return line has intensive flashing due to its high temperature causing a continuous flushing to the facing demister plates near this jet, maintaining those plates much cleaner than the ones far from the jet which are heavily scaled. Due to previous observation, a flushing system for demister sheets was proposed using the discharge flow of the condensate return brine heater through two headers fixed at the two lateral sides of the eight stages as show in Figure (8), (9). A periodical flushing term with a frequency three times per day with a duration time not exceeding 30 minutes is enough to maintain and save the performance of the demister plates. The expected costs and economical advantages are highlighted in Table (4).

The Cleaning System

The Demister is made of a metal mesh of thin wires (Stainless Steel) installed inside the evaporator flash chamber to reduce the salty saturated mist passing to distillate trays. According to its location, where it is usually facing the brine water level, it is usually exposed to the flashing activity. Also due to the disturbances in the brine water level inside the flashing chambers, these demisters plates receive high salinity water drops leading to formation of scale in the demister. This was detected in stages 1, 2,3,4,5,6,7,8 of the Sidi Krir Desalination Plant. Table (2) shows specifications of Sidi Krir Desal. Plant 2 x 5000 m^3/day.

The scale on the demister plates gradually builds up forming a considerable thick layer which becomes impermeable to vapor. In this case, the production efficiency at such stage decreases and the bottom brine temperature rises in the last stage because little heat was transferred from the brine into the recovery cycle in the stages with scaled demisters. Consequently, the efficiency of such cycle decreases as a coolant for steam flashing leading to deterioration in vacuum pressure and production rate (due to decreasing in flashing range).

Disturbances of brine seawater level inside flashing chambers and orifices disadjustment of their gates are not the only cause. Stage pressure also plays an important role. A good vacuum will reduce the impact of salty saturated brine mist droplets on demister tissues. Thus, due to the variation of vacuum along flashing chambers, the density of scale will be varied too. Stage temperature is another important factor in demister scaling.

By inspection, the scaled demisters in stages (1-8) were observed where the density of scale decreases as temperature decreases. Figure (1) shows the behavior of flashing steam in both cases (scaled and clean demister). Figures (3), (4), (5) show the scaled demisters in stage (5, 6, and 8) respectively.

Density Scaling has a direct effect in all parameters of the operation control system, consequently in the distillate production rate. Figure (2) shows a comparison of production rate between a system with a clean demister and that of a scaled one.

Chemical analysis tests of the collected samples of demister scale show that it was composed of 80% carbonates (soft scale) and 20% MgOH (hard scale).

Demister Cleaning Process

This cleaning process requires a plant shutdown of four days, where it is achieved by citric acid 5%. Sometimes in the case of large desalination plants, e.g. El Rowees 15000T/day - (Abu Dhabi,U.A.E), and in cases with severe demister scaling, plates are replaced by new ones. This will increase the maintenance cost.

As mentioned earlier, during demister inspection, it was noted that in stage (1), 60% of the demister plates were clean specially in the area which was near the inlet jet of the brine heater condensate. This is due to the intensity of the flashing action of this condensate due to its high temperature; demister plates were usually sprayed and flushed. This reduces the concentration of brine droplets in the demister and thus their scaling potential. Figure (7) shows the brine heater condensate system and its discharge.

This condensate system has two discharge paths. The first is transferred to the first stage flash chamber and the other one is into the main condensate circuit. This system is controlled by an online conductivity analyzer which directs the discharge according to the conductivity value of this condensate.

Proposal Description

It is proposed that the B/H condensate is discharged through two headers at both lateral sides of the eight stages to spray the demisters in order to reduce scaling. The proposed system is shown in Figures (8), (9).

The proposed online cleaning system could be applied three times per day with a duration time for each washing period not exceeding 30 minutes. Further, this schedule of washing should be maintained independent of the quality of the condensate.

The occurrence of demister scaling is matured after one year through the continuous operating system with a 100% load. This proposed modification to the original system is not expensive compared to the costs of production losses during shutdown for cleaning, restoration and chemicals, especially if the demisters are replaced. Table (4) shows a comparison between the costs of the present and modified system.

The proposed system will have the following advantages:

1 Maintains the performance of demisters during operation

2 Maintains the performance of the system and the target values of operation parameters

3 Saves the costs of outage for cleaning.

4 Saves the costs of chemical cleaning.

5 Saves the costs of demister replacement in the case of severe fouling.

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Table (1): Operation readings recorded in Sidi Krir Desalination system before and after the issue :-

Items Case	**Steam Flow T/h**	**Heating Steam Temp.**	**B/H inlet Temp.**	**B/H outlet Temp.**	**Bottom Brine Temp.**	**Distilled Flow T/h**	**Make up Flow T/h**	**Vent Condenser Pressure (Vacuum)**
Clean Demister	26	117	100.4	107.5	30.2	209	547	-1.0
Scaled Demister	29	114	100	108	55.0	133	338	-0.78

Table (2): Technical Specifications of Sidi Krir 2 x 5000 m^3/day MSF Desalination Plant

Parameter	Value (Remark)
No. of Units	2
Unit Capacity	5000 m^3/day
No. of Stages	20 (17 + 3)
Designed PR	8 kg (PW) / kg (steam)
TBT	110 C
Seawater Temp.	27 C
Heating Steam Temp.	117 C
Cooling Water Flow Rate	1570 m^3/hr
Brine Recirculation Flow Rate	1850 m^3/hr
Seawater Concentration	43900 ppm
Brine Concentration	63000 ppm
PW Quality	25 ppm
Method of Scale Control	High Temp. Additives (Belgard EV 2000)
Tube Sheet Material (BH + Condensers)	90 / 10 Cooper Nickel
Condensers Tubes	90 / 10 Cooper Nickel
Brine Heater Tubes	70 / 30 Cooper Nickel
Water Box (BH + Condensers)	90 / 10 Cooper Nickel

Table (3): Demonstrates the specification of B/H condensate system.

Item	Remark
Transfer Pump	2
Pump Type	Centrifugal
Pressure	6.0 Bar
Flow	26 Tons/h
Condensate Temperature	110 C`

Table (4): Estimating and comparison between present and modified system.

Item	Present	Modified
The modification which is required to the original system will cost Outage Time (Days)	4	----------
Costs of PW loss with scaled demisters (USD)	818,181.80	----------
Costs of PW loss during cleaning (USD)	36,363	----------
Chemical Costs (USD)	2000	----------
Modification Costs (USD)	-----------	2000
Total Costs (USD)	**856,544.80**	**2000**

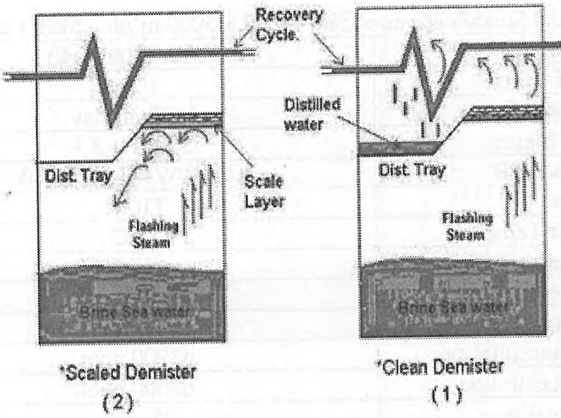

Figure (1): Behavior of Flashing Steam.

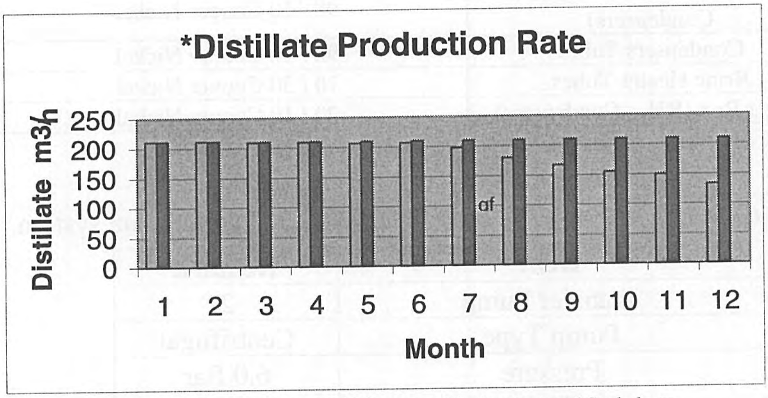

Figure (2) : Comparaison of PW. Between Ideal system and Scaled one.

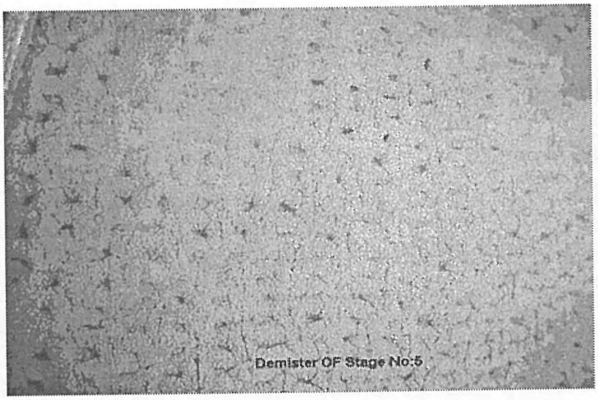

Figure (3): Demister of stage No: 5

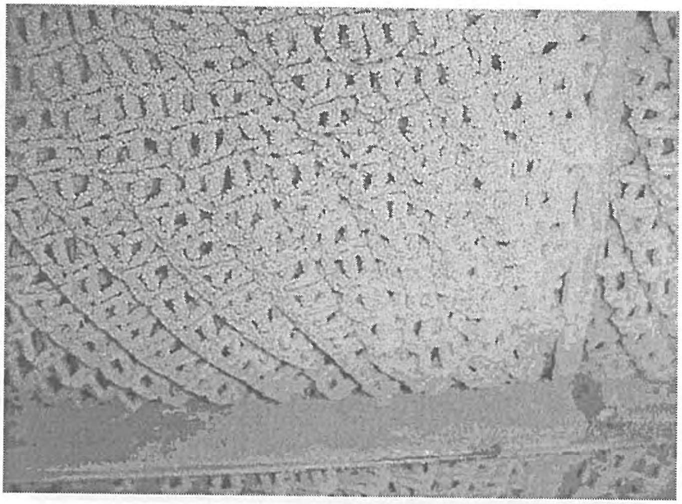

Figure (4): Demister of stage No: 6

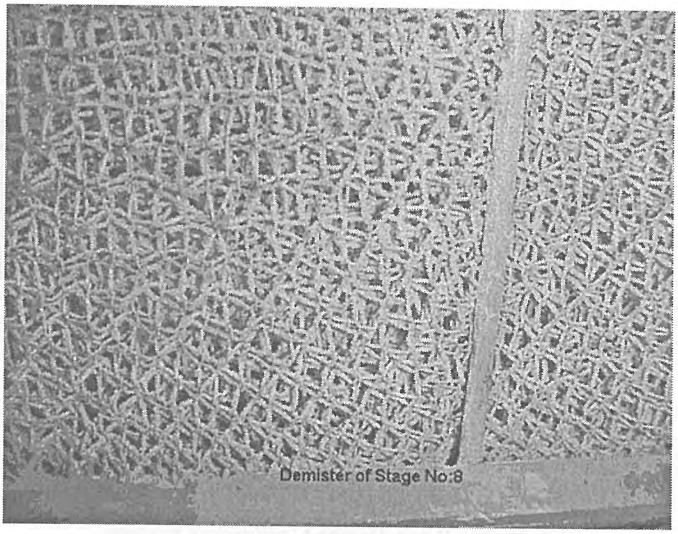

Figure (5): Demister of stage No: 8

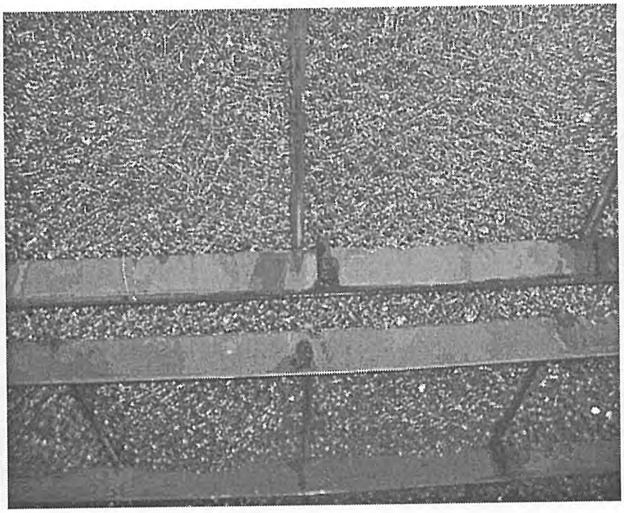

Figure (6): Clean demister. (After Acid Clean).

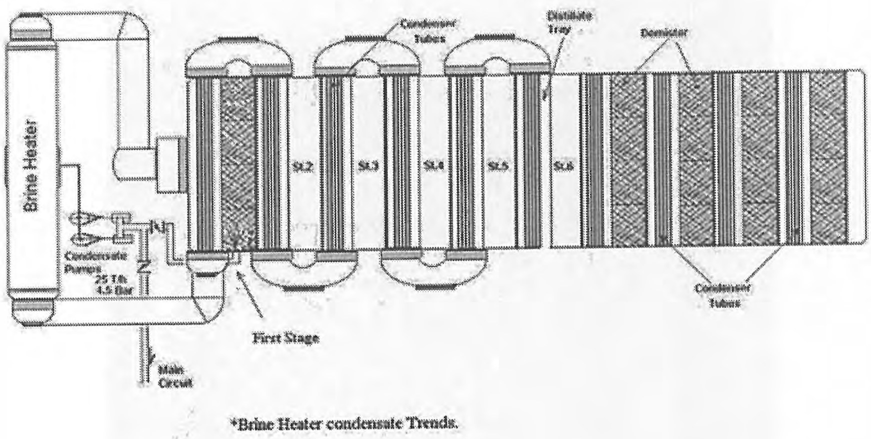

Figure (7): Brine Heater Condensate System.

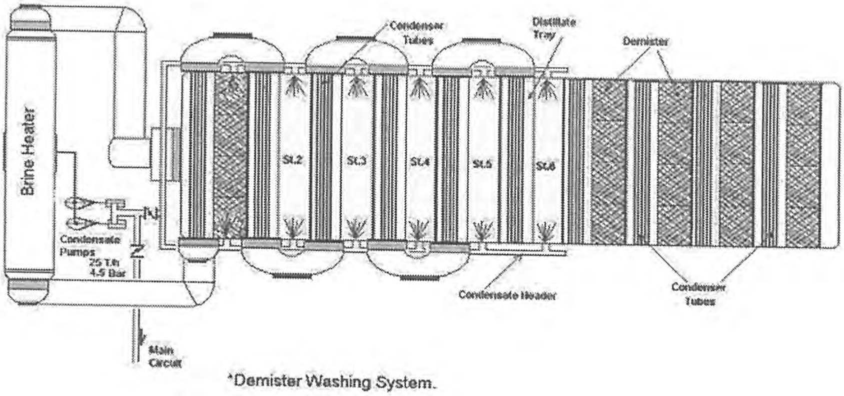

Figure (8): Demister Washing System.

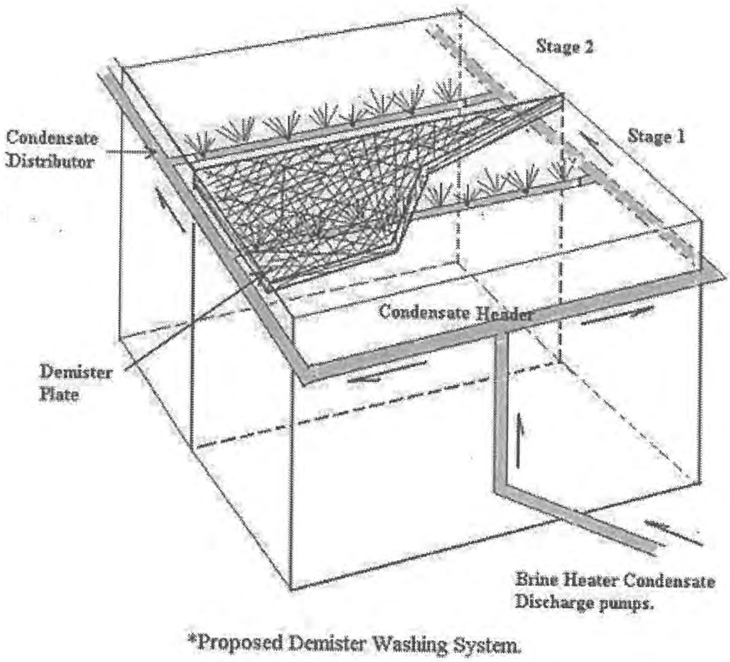

Figure (9): Demister Washing System.

Theoretical and Experimental Analysis of Radial Thickeners

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THEORETICAL AND EXPERIMENTAL ANALYSIS OF RADIAL THICKENERS

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ABSTRACT

Thickeners are a major unit in waste treatment plants used in secondary treatment to separate solids and sludge from the processed wastewater. Often they are used to concentrate dispersion from very thin suspension before filtration in order to save costs. They are also employed to produce viscous solutions or dispersions and are utilized to improve consistency and stability of emulsions. This work studies the thickener design methods, specifically finding the area of thickener for cylindrical geometry. The surest methods are based on experimental measurements of the settling curve. When such a curve is unavailable for the designer, one should use some correlation to estimate the settling velocity. The problem is complicated due to the involvement of several mechanisms such as free or hindered settling, laminar, transitional or turbulent settling. However, it is subtle that particle diameter plays an important role in the determination of thickener area. In the case of polydispersed particle size distribution, the determination of area is a bit involved. In this work, we attempt to analyze theoretically and experimentally the hindered settling of polydispersed particles in the laminar region. The analysis is then compared with different correlations from the literature. Experiments to determine the settling curve for kaolin, dicalite and calcium carbonate are performed. The thickener diameter is then calculated based on the settling curve using different correlations. The calculated diameter is close to the diameter derived from the force balance of the settling particles using the Carman-Kozeny formula. All theoretical models are checked on the basis of the industrial, pilot and laboratory data for continuous thickening of $Ca(OH)_2$, $CaCO_3$, and SiO_2.

Introduction

Although sedimentation is well known for ages and scientific interest in it comes from the beginning of the XX century [1], there is still a lot to be done in the area. So even now there are publications concerning yet unsolved problems [2].

In the accessible literature there are a lot of citations concerning the design methods of thickener cross-sectional area. However, every one of them requires the settling velocity of particles. *A priori* there are no exact calculation methods for determining this parameter. Results obtained from experiments from the settling tube, which are commonly considered as the most accurate, are only more or less an exact approximation of the real value of this parameter. This thesis is supported by the Kynch [3] theory which stated that the velocity of sedimentation for a given suspension depends only on solids concentration.

Velocity of sedimentation

Sometimes, when there is no possibility to carry out experiments, a quick estimate of the settling velocity is needed. In such a case the designer uses experimental formulae. Very often within such formulae exist the parameter *w0* free settling velocities of spherical particles, which for laminar settling (Ar<3.6) is described by Stockes' law:

Table 1: The formulae for the hindered settling velocity for monodispersed suspensions.

No.	The Formula	References
1.1	$w = w_0(1-\alpha C_v)$ $\quad C_v < 0.2, \quad \alpha = 2.5$ for spheres	Einstein [3]
1.2	$w = w_0\left(1-2.6C_v^{0.5}\right)$	Einstein [4]
1.3	$w = w_0\varepsilon^2 10^{-1.82C_v} = w_0\varepsilon^2 \exp(-4.19C_v)$	Steinour [5]
1.4	$w = w_0 \frac{(\varepsilon-b)^2}{1-b} 10^{\frac{-1.82(1-\varepsilon)}{1-b}}$	Steinour[6]
1.5	$w = \frac{kd^2(\rho_s-\rho)g}{18\eta}$	Robinson [5]
1.6	$w = \frac{\varepsilon d^2(\rho_s-\rho)g}{18\eta}$	Hawskley [5]
1.7	$w = K\frac{(1-ZC_v)^3}{C_v} + k\frac{(1-zC_v)^3}{C_v}$	Scott [7]
1.8	$w = K'(1-Z'C_v)^2 + k'(1-z'C_v)^2$	Gaudin [7]
1.9	$w = w_0(1-C_v)^n \qquad n = 4.65+19{,}5\frac{d}{D}$	Richardson, Zaki [8]
1.10	$w = w_0\varepsilon^2 \exp\left(\frac{2.5C_v}{1-\frac{39}{64}C_v}\right)$	Vand [9]
1.11	$w = w_0(1-kC_v)\left(1-0.75C_v^{1/3}\right)$	Oliver [9]
1.12	$w = w_0\left[1+0.75C_v\left(1-\sqrt{\frac{8}{C_v}}-3\right)\right]$	Brinkmann [9]
1.13	$w = w_0\frac{(1-C_v)^3}{2K^0C_v}$	Kozeny, Carman [7, 9]
1.14	$w = w_0\frac{3-4.5C_v^{1/3}+4.5C_v^{5/3}-3C_v^2}{3+2C_v^{5/3}}$	Happel [9]
1.15	$w = w_0\left(a_0+a_1C_v+a_2C_v^2+a_3C_v^3+a_4C_v^4\right)$	Shannon [9]

1.16	$w = w_0 \dfrac{1-C_v}{\sqrt{1+111\dfrac{C_v^2}{1-C_v}}}$	School [10]
1.17	$w = w_0 \dfrac{1-C_v}{1+\dfrac{1.2}{\sqrt{0.5+\left(\dfrac{\pi}{12C_v}\right)^2}-0.5}}$	Brauer [11]
1.18	$w = w_0\left[1-\left(\dfrac{C_v}{C_{v\max}}\right)^{1/3}\right]$	Iordache [12]
1.19	$w = w_0 \dfrac{(1-C_v)^2}{\left(1+C_v^{1/3}\right)\exp\left[\dfrac{5C_v}{3(1-C_v)}\right]}$	Barnea Mizrahi [13]

When the suspension is polydisperse, there are several different approaches. The simplest is to use for the calculation of the settling velocity the dimension of the smallest particles. However, this method is denied experimentally. Other methods for calculation of the velocity of sedimentation became even more complicated, e.g. [13, 14] than the formulae in the Table 1 because of using the Particle Size Distribution (PSD) discretized parameters. These methods do not consider the velocity of falling interphase as a velocity of sedimentation but discuss ideal classifying of particles.

A simple method is presented here, which was first used and checked experimentally [15-17]. An equivalent diameter for the PSD during settling is developed on a basis of the force balance acting on particles.

Based on the characteristic diameter of particle l the volume V and cross-section area f could be calculated from:

$$V = \psi_v l^3 \qquad f = \psi_f l^2 \tag{2}$$

After discretization the estimated means for the PSD class are:

$$V_i = \frac{\psi_v}{2}\left(l_{i-1}^3 + l_i^3\right)$$
$$f_i = \frac{\psi_f}{2}\left(l_{i-1}^2 + l_i^2\right) \tag{3}$$
$$l_i = \frac{1}{2}\left(l_{i-1} + l_i\right)$$

Using the probability definition for the PSD class one obtains:

$$p_i = \frac{n_i}{\sum_{i=1}^{k} n_i} = \frac{\dfrac{m_i}{V_i\rho_s}}{\sum_{i=1}^{k}\dfrac{m_i}{V_i\rho_s}} = \frac{\dfrac{m_i}{V_i}}{\sum_{i=1}^{k}\dfrac{m_i}{V_i}} \tag{4}$$

The average parameters of the whole PSD are:

$$\bar{V}=\sum_{i=1}^{k} p_i V_i \quad \bar{f}=\sum_{i=1}^{k} p_i f_i \quad \bar{l}=\sum_{i=1}^{k} p_i l_i \tag{5}$$

Considering steady state free falling of spherical particles, there is equilibrium of gravity, buoyancy and friction forces and the particle is moving with the monotonous motion, so with constant velocity:

$$V\rho_s g = V\rho g + \frac{w_0^2}{2}\lambda\rho f \quad \lambda=\frac{24}{\mathrm{Re}}=\frac{24\eta}{w_0\rho d} \tag{6}$$

Within the above equation appears the volume V, the cross-sectional area f and the diameter of the spherical particle d which gives after rearranging the equation (1). Instead of V, f and d for the whole PSD, the average parameters from equations (3-5) are substituted. Then instead of (1) the next equation is obtained:

$$w_0=\frac{\bar{V}\bar{l}}{\bar{f}}\frac{(\rho_s-\rho)g}{12\eta} \tag{7}$$

Table 2: Selected properties of investigated suspensions.

Substance	ρ_s kg/m3	ρ kg/m3	η Pas	mass fraction/dimeter of PSD class /µm 0-2	2-5	5-10	10-20	>20	l_{max} µm
Koaline	2490	998	0.0010	0.5000	0.1930	0.1375	0.1468	0.0227	60
Dicalite	2270	999	0.0010	0.2046	0.0522	0.1892	0.4631	0.0909	60
Calcite	2310	999	0.0010	0.5341	0.1364	0.1306	0.1364	0.0625	60
Dicalite	2270	998	0.0010	0.0676	0.1627	0.3248	0.3059	0.1391	60
Dicalite	2270	999	0.0010	0.0387	0.1291	0.4200	0.2990	0.1236	60
Dicalite	2270	999	0.0010	0.0562	0.0958	0.2508	0.3935	0.2037	60
Dicalite	2270	999	0.0010	0.0880	0.1030	0.3420	0.3050	0.1620	60
Dicalite	2270	999	0.0010	0.0747	0.0995	0.4978	0.2796	0.0484	40
Dicalite	2270	1089,5	0.0035	0.0747	0.0995	0.4978	0.2796	0.0484	40

It is known [5] that the Carman constant K^0 depends on the structure of suspension (shape and diameter of particles as well as concentration of dispersion). Using the Least Square Method (LSM) for experimental data of [16] the subsequent equation is obtained:

$$K^0=29.4\left(\frac{\bar{V}}{\bar{l}\bar{f}}\right)^{0.64} C_V^{0.88} \tag{8}$$

Finally combining equation (7), (8) and (1.13) the next formula is derived:

$$w=\frac{\bar{V}}{\bar{l}\bar{f}}\frac{(\rho_s-\rho)g(1-C_V)^3}{705.6\eta\left(\frac{\bar{V}}{\bar{l}\bar{f}}\right)^{0.64} C_V^{0.88}} \tag{9}$$

Which is valid for Ar < 3.6 (Re < 0.2).

Area of radial thickener

The main idea for calculation of a thickener area is based on the statement that the velocity of a falling particle within the settling zone should be larger or equal to the upward liquid velocity. There are a lot of graphical and analytical methods used for determining the area of a thickener. Information about the graphical methods is available in the literature [1, 10, 13, 18-27]. From the citations [1, 5, 10, 13, 18, 28-38] seven different types of analytical methods have been developed which are presented in Table 3.

Table 3: Types of analytical methods for calculation of radial thickener area.

No	Formula	Remarks
3.1	$A = k\frac{\dot{V}}{w}\frac{C_{Vu}-C_{Vi}}{C_{Vu}-C_{Vo}}$	$w = w(C_{Vi})$ k=1.5÷2 in heated buildings k=1.7÷3 outside buildings
3.2	$A = k\frac{\dot{V}}{w}\frac{C_{Vi}}{C_{Vu}}$	$C_{Vc}=0$; $w=w(C_{Vi})$ or $w=w(C_{Vo})$ $k=1÷4$
3.3	$A = k\frac{\dot{V}(1-C_{Vi})}{w}$	$w=w(C_{Vi})$ $k=1÷4$
3.4	$A = \frac{\dot{V}}{w}$	$w=w(C_{Vi})$
3.5	$A = \frac{\dot{V}}{w-v}\frac{C_{Vu}-C_{Vi}}{C_{Vu}-C_{Vo}}$	$w=w(C_{Vi})$; $v=0.152w$ or $v = 0.0282\frac{w}{h^{0.2}}$
3.6	$A = k\dot{V}\frac{C_{Vi}-C_{Vo}}{C_{Vu}-C_{Vo}}C_{Vu}\left(\frac{\frac{1}{C_V}-\frac{1}{C_{Vu}}}{w}\right)_{max}$	$C_V \in [C_{Vi}, C_{Vu}]$ $w=w(C_V)$ k=1.5÷2 in heated buildings k=1.7÷3 outside buildings
3.7	$A = k\dot{V}\frac{C_{Vi}-C_{Vo}}{C_{Vu}-C_{Vo}}C_{Vu}\left(\frac{\frac{1}{C_V}-\frac{1}{C_{Vu}}}{w-w(C_{Vu})}\right)_{max}$	$C_V \in [C_{Vi}, C_{Vu}]$ $w=w(C_V)$ k=1.5÷2 in heated buildings k=1.7÷3 outside buildings

Verification of the above formulae is performed using experimental data from industrial plants [28] (A=285 m^2), laboratory rigs [34-36] (A=0.09348 m^2), [37, 38] (A=0.07069 m^2) and pilot plants [39] (A=2.6267 m^2). Table 4 presents chosen characteristic measurement conditions for these experiments.

Table 4: Measurement conditions of continuous thickeners.

No	Reference	Sedimentation Substance	$\dot{V}_{i\max}$ dm3/min	C_{Vi}	C_{Vu}	C_{Vo}
4.1	[28]	$Ca(OH)_2$	666.7	0.00978	0.01997	0,00798
4.2	[34]	$CaCO_3$	2.820	0.035-0.10	0.070-0.216	0
4.3	[35]	$CaCO_3$	5.000	0.035-0.065	0.043-0.198	0
4.4	[36]	$CaCO_3$	6.000	0.025-0.084	0.030-0.210	0
4.5	[36]	SiO_2	5.244	0.037	0.0375-0.041	0
4.6	[37-38]	$CaCO_3$	1.852	0.0088-0.0202	0.0354-0.2213	0.0014-0.0078
4.7	[39]	SiO_2	105.03	0.0121-0.0836	0.0885-0.1696	0

Checking calculations are performed in a next specified way:

- For a given formula (Table 3), computations of the area of the thickener for every experimental point are done with $k=1$.
- A quotient of the calculated value of area to the actual value is computed for comparison of results obtained for different thickeners.

$$A_{qi} = \frac{A_i}{A_a}$$

- for statistical purposes the next estimators are calculated:

$$\overline{A}_q = \frac{\sum_{i=1}^{n} A_{qi}}{n} \quad ; \quad s = \sqrt{\frac{\sum_{i=1}^{n} (A_{qi} - 1)^2}{n-1}}$$

Obtained results are presented in Table 5. Generally there is a large spread observed (difference between $A_{q\,max}$ and $A_{q\,min}$). The best agreement give formulae (3.5, 3.1, 3.6) because the $\overline{A}_q$ values are the closest to *1* and *s* assesses are the smallest. Equation (3.5) requires *a priori* knowledge of the thickener height, so it can be used only in a very specific situation. Formula (3.6) is a simplification of (3.7) and it gives more accurate results. Using of (3.6) method requires also great caution and accuracy by determining experimental values of $w=w\,(C_V)$. Even small errors there could be multiplied by calculation of an expression

$$\frac{\frac{1}{C_V} - \frac{1}{C_{Vu}}}{w(C_V)}.$$

Based on obtained results, the safest method seems to be use of the classical formula (3.1) for calculation the thickener area.

Table 5: Experimental verification of formulae from Table 3.

No	$\overline{A}_q$	s	$A_{q\,min}$	$A_{q\,max}$	n
3.1	0.9512	0.7374	0.1963	4.1632	97
3.2	3.4206	6.7556	0.0367	32.1567	97
3.3	4.1368	6.8433	0.4670	31.8038	97
3.4	4.3438	7.1667	0.4727	33.0258	97
3.5	0.9662	0.7495	0.1996	4.2162	97
3.6	0.8313	0.4334	0.1591	2.5437	97
3.7	2.6468	2.9998	0.3069	13.0617	97

Conclusions

The settling curve for kaolin, dicalite and calcium carbonate in a cylindrical crystallizer is determined experimentally in order to calculate the unit diameter using different correlations. The most predictive model is the one based on the force balance of the settling particles using the Carman-Kozeny formula. This formula gives the closest diameter evaluation compared to all other theoretical models examined.

Symbols

$Ar = \dfrac{l^3(\rho_s - \rho)\rho g}{\eta}$	Archimedes' number	
$Re = \dfrac{wl\rho}{\eta}$	Reynolds' number	
A	the thickener area	/ m^2
C_v	volume fraction	
d	particle diameter	/m
f	particle cross-section area	/m^2
g	gravity acceleration	
h	height of the thickener	/m
k		
K		
K^0	Carman constant	
l	particle linear diameter	/m
m	mass	/kg
n	number	
p	probability	
v	velocity of deceleration	/m/s
V	volume	/m^3
$\dot{V}$	volumetric flow	/m^3/s
w	velocity	/m/s
ε	porosity	
η	dynamic viscosity	/Pas
λ	flow resistance coefficient	
ρ	density	/kg/m^3
ψ	shape factor	

Subscripts

f - area
i - inflow

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Microbial genetic control of heavy metal pollution from wastewater

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MICROBIAL GENETIC CONTROL OF HEAVY METAL POLLUTION FROM WASTEWATER

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ABSTRACT

Standards have been set and guidelines proposed by many countries and several intergovernmental organizations to determine the acceptable human exposure to certain environmental pollutants in drinking water. Factory-effluent polluted river water was tested for the removal of their heavy metal contents using microbial biomass of genetic engineered bacteria and yeast strains. Microbial biomass of most bacterial strains and transconjugants can successfully remove higher concentration of cadmium; cobalt and arsenic in the presence or absence of sugarcane refuse in solutions. Some of them could oxidise arsenic to arsenate, which is more easily precipitated from wastewater by Fe^{3+} than is arsenite with a decrease in toxicity and relevance wastewater treatment. Microbial cells of bacterial strains and transconjugants are shown to be more efficient in the removal and recovery of total heavy metals from industrial effluents. There is no clear relationship between the ability of bacterial strains or transconjugant to absorb cadmium from the effluents and its capacity for selective absorption of cobalt and arsenic. Furthermore, the transfer of DNA^- by conjugation certainly makes many of the transconjugants much more capable of uptaking heavy metals than its parental strains. Extremely high ability of total heavy metals uptake was newly found in most yeast hybrids. In addition, hybrid cells of yeast differed in their removal efficiencies of total heavy metals uptake. Many of yeast hybrids, in the presence of sugarcane refuse, showed greater removal of total heavy metals from industrial effluents. The recognition that genetic engineering could adapt the microbial strains to the effluents resulted from researchers in the chemical industry, especially geneticists, who developed biotechnology for use in pollution control of hazardous wastes.

Keywords: Arsenic, cadmium, cobalt, hybrid cells, industrial effluents, *Saccharomyces cerevisiae,* transconjugant.

INTRODUCTION

Man today is concerned very much with pollution of the environment, the slow poisoning of his surroundings by his own activities. This is the major global health hazard. Many metals are essential for microbial growth in low concentrations, but they become toxic in high concentrations. Some microorganisms, exposed to high levels of heavy metals, have involved resistance mechanism(s), such as active exclusion (42), forming cysteine-rich proteins (23), increasing w - cyclohexyl fatty acids level in the membrane (39), detoxification by redox conversion (10), and producing hydrogen sulfide (13) and extracellular polysaccharide (4). This enables them to maintain their functions in the presence of high concentrations of heavy metals. The cysteine-rich, metal-binding protein (30) has putative functions in the regulation of essential elements, metal detoxification, and protection against oxidative stress (9). The most compelling evidence for a role of metallothionein (MT) in protection against metal toxicity is provided by studies on yeast (22) and mice (34), which demonstrated disruption or deletion of the MT gene resulting in the inability to express MT and resist metal toxicity (Cu toxicity in the case of yeast and Cd toxicity in the case of mice).

The removal of radionuclides, metal or metalloid species, compounds and particulates from solution by biological material, particularly by non-directed physico-chemical interactions, is now frequently termed "biosorption" (44). Although virtually all biological material has biosorptive properties (33), most work up-to-date has been directed towards microbial systems. Biosorption, and related phenomena, are of importance because the removal of potentially toxic and/or valuable metals and radionuclides from aqueous effluents can result in detoxification and, therefore, safe environmental discharge (32). Furthermore, appropriate treatment of loaded biomass can enable recovery of valuable elements for recycling or further containment (5). This paper reports on the application of biotechnology for maximal accumulation of heavy metals from factory effluents by various genetically constructed microorganisms.

MATERIAL AND METHODS

Genetic material: Yeast and bacterial strains used in this study are listed in a previous work by Kosba *et al.* (28). Media for growing microorganisms and other culture conditions have been described previously by Horikoshi *et al.* (25). Precultured cells were used for the following uptake experiments.

Factory effluents: The present study was undertaken with the finishing industry of wastewater resulting from an ammonia unit at Talkha Fertilizer Factory (TFF) (Dakhlia Governorate). Polluted water was collected from the main pipe of the factory before being mixed with water in the river. This collection was done through January and June from every year (1995 and 1996). Sugarcane refuse was used for its residual sucrose to use as a sole carbon source in some biosorption experiments.

Uptake experiments: In the heavy metals uptake test, precultured cells were suspended in 250 ml conical flasks containing 100 ml minimal medium of yeast (12) and bacteria (43) supplemented with factory effluents instead of distilled water and incubated under static conditions at 30°C for six days. Thereafter, the cells were collected by filtration on membrane filter (pore size 0.45 mm) and others by centrifugation. Amounts of metals taken up by the cells were determined by measuring metal content in the filtrate, using the Atomic Absorption Spectrometer at Chemistry Dept., Faculty of Science, Mansoura University, according to Nakajima and Sakaguchi (37).

RESULTS AND DISCUSSION

Heavy metals uptake by parental strains of bacteria: Microorganisms can accumulate heavy metals and radionuclides from their external environment (2). Amounts accumulated can be large and a variety of physical, chemical and biological mechanisms may be involved, including adsorption, precipitation, complexion and transport. The cells of seven parental strains of bacteria were suspended in a solution containing factory effluents from the Talkha Chemical Fertilizer Factory. The results obtained with representative strains are shown in Tables (1, 2 and 3). From Table (1) it can be seen that the amounts of heavy metals (cadmium, cobalt and arsenic) absorbed by the bacterial cells differed markedly in different strains of bacteria. Of the seven bacterial strains tested, extremely high cadmium - absorbing ability was observed in *Micrococcus halobius* A and B, *Micrococcus luteus* and *Bacillus subtilis*. These strains resulted in a greater decrease in cadmium concentration, reaching up to 50%. *Micrococcus halobius* A, *Bacillus subtilis* and Bacillus *licheniformis* revealed a greater decrease in cobalt concentration up to 50%. This depends on the ability of microorganisms to effect chemical transformations of heavy metals and their compounds by oxidation, reduction, methylation and demethylation (31). Once inside cells, metal ions may be compartmentalized and/or converted to less toxic forms by precipitation or sequestration by metal-binding proteins (18). Arsenic removal efficiencies up to 91.67% have been recorded by *Bacillus licheniformis*. Some bacterial strains could oxidise arsenite As^{3+} to As^{5+}, which is more easily precipitated from wastewater by Fe^{3+} than is arsenite, As^{3+}. These strains included *Micrococcus halobius* A, *Bacillus cereus, Micrococcus luteus* and *Bacillus subtilis,* which increase the ionic concentration of arsenite. These results are in accordance with those reported by Williams and Silver (45), who reported that treatment of arsenic loaded sewage with arsenite oxidase-producing bacteria (which oxidise As^{3+} to As^{5+}) can improve certain arsenic removal methods since arsenate, As^{5+} is more easily precipitated from wastewater by Fe^{3+} than is arsenite, As^{3+}. Microbial transformation of arsenic also, is associated with a decrease in toxicity and may have relevance to wastewater treatment.

A similar range in absorption ability was observed by the same strains grown in minimal medium containing 1% sugarcane refuse (Table 2). It can be seen that extremely high cadmium-absorbing ability was observed in *Micrococcus lylae, Bacillus subtilis* and *Bacillus licheniformis*, which decreased cadmium concentration up to 40%. Extremely high cobalt-absorbing ability was found in *Bacillus cereus*, cobalt uptake by this strain reached up to 100%. The mechanisms and significance of metal-binding proteins to microbial cells have been recorded in all microbial groups examined, e.g. cyanobacteria, bacteria, microalgae and filamentous fungi (40). Metallothioneins are small cysteine-rich polypeptides that can play an essential role in the ability of strains for heavy metals uptake. They can bind essential metals, e.g. Cu, Zn and Co, as well as non-essential metals like Cd (21). In the presence of sugarcane refuse, most strains have arsenic removal greater than 50%. In that case, none of the strains oxidise As^{3+} (arsenic) to As^{5+} (arsenate), as seen by the same strains when grown without the addition of sugarcane refuse. Oxidation is one mechanism of the chemical transformation of heavy metals and their compounds (27).

Table 1: Heavy metals uptake by parental strains of bacteria from minimal medium containing industrial effluents.

Strain No.	Cadmium (Cd)		Cobalt (Co)		Arsenic (As)	
	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)
Control	0.08	0.00	0.12	0.00	0.012	0.00
1	0.04	50.0	0.06	50.0	0.022	+83.33
2	0.04	50.0	0.12	0.00	0.004	66.67
3	0.08	0.00	0.12	0.00	0.016	+33.33
4	0.04	50.0	0.12	0.00	0.023	+91.67
5	0.06	25.0	0.12	0.00	0.002	83.33
6	0.04	50.0	0.06	50.0	0.021	+75.00
7	0.06	25.0	0.06	50.0	0.001	91.67

Concen. = Concentration. + = Increase in concentration.

Amongst the different bacterial strains, there are many of them with a high ability for one or more of the heavy metals uptaken (Table 3), and that oxidise arsenic to arsenate in the medium, not containing sugarcane refuse. These results are in accordance with those reported by Nakajima and Sakaguchi (37), who found that uranyl, mercury, lead and copper ions were more readily accumulated by cells of bacteria, yeasts, fungi and actinomycetes than the other ions presented in the medium. Indeed, the quantities of zinc, manganese, cobalt, nickel and cadmium absorbed by almost all species of microorganisms, were found to be extremely low. The results suggest that the selective accumulation of heavy metal ions by microorganisms could be determined by interionic competition. On the other hand, the relationship between the uptake of cadmium, cobalt and arsenic was not the same in all strains.

The data tabulated in Table (4) showed that the total quantity of metal ions absorbed by bacterial cells differed greatly from one strain to another. Extremely high absorption of total heavy metals was observed in *Micrococcus halobius* A, *Bacillus subtilis* and *Bacillus licheniformis*. These strains show greater uptake of total heavy metals, more than 40%. This is in accordance with the results obtained by Nakajima and Sakaguchi (37), who found that the high absorption of heavy metals was observed in *Bacillus subtilis, Actinomycetes flavoviridis, Streptomyces obiraceus, Streptomyces albus, Streptomyces diastaticus, Streptomyces viridochromogenes* and *Mucor javanicus*.

Table 2: Heavy metals uptake by parental strains of bacteria from minimal medium containing factory effluents and 1% sugarcane refuse.

Strain No.	Cadmium (Cd)		Cobalt (Co)		Arsenic (As)	
	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)
Control	0.10	0.00	0.18	0.00	0.019	0.00
1	0.10	0.00	0.18	0.00	0.003	84.21
2	0.10	0.00	0.18	0.00	0.000	100.0
3	0.08	20.0	0.00	100.0	0.004	78.95
4	0.08	20.0	0.12	33.33	0.006	68.42
5	0.06	40.0	0.12	33.33	0.008	57.89
6	0.06	40.0	0.18	0.00	0.016	15.79
7	0.06	40.0	0.18	0.00	0.004	78.95

Concen. = Concentration.

Table 3: Concentration (mg/L) of heavy metals uptake by parental strains of bacteria.

Strains	Minimal medium of wastewater from TFF					
	Without sugarcane refuse			With 1% sugarcane refuse		
	Cd	Co	As	Cd	Co	As
1	0.04	0.06	+0.010	0.00	0.00	0.016
2	0.04	0.00	0.008	0.00	0.00	0.019
3	0.00	0.00	+0.004	0.02	0.18	0.015
4	0.04	0.00	+0.011	0.02	0.06	0.013
5	0.02	0.00	0.010	0.04	0.06	0.011
6	0.04	0.06	+0.009	0.04	0.00	0.003
7	0.02	0.06	0.011	0.04	0.00	0.015

+ = Increase in arsenic concentration of tested sample., TFF = Talkha Fertilizer Factory.

In the presence of sugarcane refuse, *Bacillus cereus* recovered total heavy metals at a high adsorption rate greater than 70%. In addition, the total quantity of metal ions absorbed by microbial cells greatly differed from one strain to another. Extremely lower absorption of total heavy metals uptake was observed in *Micrococcus halobius* A and B rather than the other ones. These results are in agreement with those obtained previously on uranium uptake by immobilized *Chlorella regularis* and *Streptomyces viridochromogenes* (38). These results show that the microbial cells are more efficient in the removal and recovery of heavy metals from factory effluents. Microbial biomass showed higher percent in the removal of total heavy metals in the presence of sugarcane refuse. It appears that the metals are largely bound by the extracellular polymers produced by the microorganisms present, particularly bacteria (19).

Table 4: Efficiency of biological control of pollutants by parental strains of bacteria on total heavy meals uptake from minimal medium containing factory effluents.

Strains	Concentration (ppm) of Cd, Co and As		Total concentration Tion absorbed (ppm)		Removal (%)	
	I	II	I	II	I	II
Control	0.212	0.299	0.00	0.00	0.00	0.00
1	0.122	0.283	0.09	0.016	42.45	5.35
2	0.164	0.280	0.048	0.019	22.64	6.35
3	0.216	0.084	+0.004	0.215	+1.89	71.91
4	0.183	0.206	0.029	0.093	13.68	31.10
5	0.182	0.188	0.03	0.111	14.15	37.12
6	0.121	0.256	0.091	0.043	42.92	14.38
7	0.121	0.244	0.091	0.055	42.92	18.39

+ = Increase in total concentration. I = Minimal medium without sugarcane refuse.
II = Minimal medium with 1% sugarcane refuse.

Removal of heavy metals by bacterial transconjugants: Bacterial transconjugants (Table 5) provide an additional capacity for the removal of cadmium, cobalt and arsenic. Heavy metal ions absorbed by transconjugants differed greatly from one to another. Extremely high absorption, greater than 40% was observed in many of the transconjugants for cadmium, cobalt and arsenic. The results, however, do not show a clear relationship between the ability of transconjugant to absorb cadmium from these industrial effluents and its capacity for selective absorption of cobalt and arsenic. These are in accordance with the results obtained by Nakajima and Sakaguchi (37), who found that the amounts of uranium absorbed by the bacterial cells markedly differed in different species of bacteria. The present results, also, are in agreement with those obtained by Gadd (17), who found that living cell biofilms might provide an additional capacity for the removal of pollutants, including hydrocarbons, pesticides and nitrates. A similar range in absorption ability was observed by transconjugants in the presence of sugarcane refuse (Table 6). It has been shown that the amounts of cadmium, cobalt and arsenic markedly differed in different transconjugants of bacteria. Many of transconjugants recovered cadmium and cobalt with removal efficiencies up to 71.43%. However, many of them could oxidise arsenic to arsenate. This indicated that these strains produce arsenite oxidase, which oxidise As^{3+} to As^{5+}, and could improve certain arsenic removal methods, as arsenate, As^{5+}, is more easily precipitated from wastewater by Fe^{3+} than is arsenite, As^{3+} (45). These results are in accordance with the results reported by Mergeay *et al.* (36), who demonstrated that correlation between selection pressure exerted by pollutants and plasmid emergency was, in fact, suggested in many cases: plasmids seem to play a major role in the adaptation of bacteria to xenobiotics and in the acquisition of new genetic traits due to pollution. Plasmid-encoded pathways are ecologically advantageous because they provide genetically flexible systems and can be transferred between bacterial species (41). A particularly important aspect is the occurrence of some broad host range plasmids specialized in the degradation of synthetic chemicals (7). Due to their broad transfer and replication range, their introduction into a microbial community could provide the latter with enhanced degradative capacities.

As shown in Table (7) cobalt and arsenic ions were more readily accumulated by cells of bacterial transconjugants in minimal medium without sugarcane refuse than the other ions of cadmium in the medium. However, all transconjugants tested accumulated larger amounts of the cobalt ion than cadmium in the presence of sugarcane refuse. This suggests that the selective accumulation of heavy metal ions by transconjugants is determined by interionic competition (37).

Table 5: Heavy metals uptake by bacterial transconjugants from minimal medium containing factory effluents.

Strain No.	**Cadmium (Cd)**		**Cobalt (Co)**		**Arsenic (As)**	
	Concen. (ppm)	**Removal (%)**	**Concen. (ppm)**	**Removal (%)**	**Concen. (ppm)**	**Removal (%)**
Control	0.06	0.00	0.36	0.00	1.04	0.00
10	0.04	33.33	0.28	22.22	0.98	5.77
11	0.03	50.00	0.21	41.67	0.47	54.81
12	0.04	33.33	0.21	41.67	0.48	53.85
13	0.03	50.00	0.28	22.22	0.10	9.38
14	0.05	16.67	0.28	22.22	0.97	6.73
15	0.03	50.00	0.21	41.67	0.51	50.96
16	0.02	66.67	0.21	41.67	0.95	8.65
17	0.03	50.00	0.21	41.67	0.85	18.27
18	0.02	66.67	0.28	22.22	1.01	2.88
19	0.04	33.33	0.28	22.22	0.53	49.04
20	0.02	66.67	0.21	41.67	0.97	6.73
21	0.02	66.67	0.28	22.22	0.97	6.73

Concen. = Concentration.

Table 6: Heavy metals uptake by bacterial transconjugants from minimal medium containing factory effluents and 1% sugarcane refuse.

Strain No.	Cadmium (Cd)		Cobalt (Co)		Arsenic (As)	
	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)	Concen. (ppm)	Removal (%)
Control	0.07	0.00	0.28	0.00	18.03	0.00
10	0.02	71.43	0.14	50.0	18.03	0.00
11	0.03	57.14	0.21	25.0	17.70	1.83
12	0.02	71.43	0.14	50.0	18.90	0.00
13	0.05	28.57	0.21	25.0	16.06	10.93
14	0.04	42.86	0.14	50.0	18.20	+0.94
15	0.04	42.86	0.14	50.0	17.60	2.38
16	0.03	57.14	0.14	50.0	18.70	+3.72
17	0.02	71.43	0.14	50.0	17.80	1.28
18	0.05	28.57	0.21	25.0	16.40	9.04
19	0.05	28.57	0.21	25.0	18.90	4.83
20	0.06	14.29	0.21	25.0	19.10	+5.93
21	0.06	14.29	0.21	25.0	17.10	5.16

Concen. = Concentration.

Table 7: Concentration (mg/L) of heavy metals absorbed by transconjugants.

Transconj-Ugants	Minimal medium of wastewater from TFF					
	Without sugarcane refuse			With 1% sugarcane refuse		
	Cd	Co	As	Cd	Co	As
10	0.02	0.08	0.06	0.05	0.14	0.00
11	0.03	0.15	0.57	0.04	0.07	0.33
12	0.02	0.15	0.56	0.05	0.14	+0.87
13	0.03	0.08	0.94	0.02	0.07	1.97
14	0.01	0.08	0.07	0.03	0.14	+0.17
15	0.03	0.15	0.53	0.03	0.14	0.43
16	0.04	0.15	0.09	0.04	0.14	+0.67
17	0.03	0.15	0.19	0.05	0.14	0.23
18	0.04	0.08	0.03	0.02	0.07	1.63
19	0.02	0.08	0.51	0.02	0.07	+0.87
20	0.04	0.15	0.07	0.01	0.07	+1.07
21	0.04	0.08	0.07	0.01	0.07	0.93

The removal and recovery of total heavy metal pollutants without any addition of sugarcane refuse (Table 8) indicated that many of the transconjugants showed removal efficiencies greater than 40%. The removal efficiencies of total heavy metals measured in this work was up to 71.92%. Recovery efficiencies, obtained in transconjugants, were greater than those observed by parental strains (Table 4). Furthermore, the transfer of DNA by conjugation certainly makes many of transconjugants much more able to uptake heavy metals than the capacity of its parental strains. This is in agreement with those reported by Day (11), who demonstrated that the horizontal gene transfer was of special interest in the communities of polluted soils because specific adaptations to pollutants were often plasmid-bound. In the presence of sugarcane refuse, total heavy metals uptake by transconjugants were weaker than without it. Removal efficiencies of total heavy metal ions up to 11.21% have been recorded. However, parental strains under the same conditions were more efficient for removal other than their transconjugants. These are in agreement with those reported by McClure *et al.* (35), who noticed that a strain with plasmid-borne catabolic genes, introduced into an activated sludge unit, did not enhance the degradation of 3-chlorobenzoate (3CB), while a total breakdown could be achieved in batch cultures. On the other hand, a constructed *Pseudomonas aeruginosa* strain, carrying a degradative plasmid, has been shown to be useful for cleaning up soil contaminated with kelthane residues (20).

The results obtained with representative transconjugants for their ability to uptake cadmium and cobalt are shown in Table (9). It can be seen that all transconjugants revealed a positive uptake of heavy metals, related to the mid parents, or to the better parent, or to both of them. The positive uptake of heavy metals appeared in the presence of sugarcane refuse in minimal medium or without the addition of any carbon source to the medium. It has been shown that, in many cases, transconjugants greatly facilitated the efficiency of heavy metals removal and the use of cells as biological catalysts (16). This is in accordance with the results reported by Apajalahti and Salkinoja (1), which isolated a *Rhodococcus chlorophenolicus* strain capable of degrading polychlorinated phenols. This strain continued to degrade polychlorinated phenols in natural soil when immobilized on biodegradation foam (6). These results suggest that it may be possible to develop practical systems, based on the use of chemical treatment, to detoxify chemicals in factory wastes with biological treatment. There is an urgent need for the removal of heavy metals from industrial effluents rather than with chemical therapy. In this respect, the transfer of DNA by conjugation certainly plays a greater role in the capacity of removal of heavy metals from these effluents (11). The increased knowledge on DNA transfer through conjugation opens exciting new areas in environmental biotechnology. At present, industry complains about high costs of chemical therapy, although the recovery of the effluents and, also, the environment from the pollutants is often extremely difficult with this therapy of chemical methodology.

Recovery of heavy metals by yeast hybrids: As shown in Table (10) many of the heterozygous diploids of yeast were found to be efficient in uptaking and removing cadmium and cobalt (> 33.33%). Their removal efficiency reached up to 60% and 66.67%, respectively, although the removal of arsenic reached 45.36%. On the other hand, the relationship between the uptake of cadmium and absorption of cobalt or arsenic was not the same in all hybrids. In almost all hybrids of yeast, the removal of arsenic was less than that of cadmium and cobalt. This is in accordance with the results obtained by Nakajima and Sakaguchi (37), who found that, in bacteria and yeasts many species were found to accumulate mercury more abundantly than uranium. The present results indicated that cells of yeast hybrids have been newly found to have extremely high ability for heavy metals uptake. There is now great awareness of the potential efficiencies in removal and recovery of environmental pollution by heavy metals through biological control of pollutants by yeast rather than chemical methods. The biosorbent costs have been analysed by Kuyucak (29). Specifically, cultured biomass, e.g. fungi, yeasts, may cost ~ \$ 1-5 Kg dry weight^{-1} with specifically-cultured algae being ~ \$ 15-18 Kg dry weight^{-1}. The production cost of yeast (*S. cerevisiae*) by a large company was estimated as ~ \$ 1.3 Kg^{-1} with the cost of supply being ~ \$ 2-2.6 Kg^{-1} depending on demand, in comparison, with the higher expense of chemical treatment.

Table 8: Efficiency of biological control of pollutants by transconjugants on total heavy metals uptake from minimal medium containing factory effluents.

Transconjugants	Concentration (ppm) of Cd, Co and As		Removal (%)	
	I	II	I	II
Control	1.46	18.38	0.00	0.00
10	1.30	18.19	10.96	1.03
11	0.71	17.94	51.37	2.39
12	0.73	19.06	50.0	+3.70
13	0.41	16.32	71.92	11.21
14	1.30	18.38	10.96	0.00
15	0.75	17.78	48.63	3.26
16	1.18	18.87	19.18	+2.67
17	1.09	17.96	25.34	2.29
18	1.31	16.66	10.27	9.36
19	0.85	19.16	41.78	+4.24
20	1.20	19.37	17.81	+5.39
21	1.27	17.37	13.01	5.50

+ = Increase in concentration. , I = without sugercane refuse.
II = In the presence of sugarcane refuse.

In the presence of sugarcane refuse, it has been shown that many hybrids showed higher removal efficiencies up to 50, 66.67 and 62.30% for cadmium, cobalt and arsenic, respectively. The investigation of these phenomena for heavy metals uptake capacity by hybrid cells of yeast should, also, be applied to maximize the efficiency of industrial effluents therapy. This is in agreement with the results reported by Fogel *et al.* (14), who demonstrated that the efficiencies of metal removal were due to metallothioneins (MT). They were very small cysteine-rich polypeptides that could bind essential metals, e.g. Cu and Zn, as well as non-essential metals like Cd. Copper resistance, for example, in *Saccharomyces cerevisiae* is mediated by the induction of a 6573 - dalton cysteine-rich protein, copper metallothionein (Cu - MT). It has been suggested that Cu – MT (and analogous proteins) might be of potential in metal recovery since it could bind other metals besides Cu, e.g. Cd, Zn, Ag, Co and Au, although these metals did not generally induce MT synthesis (8). One report has described an inducible Cd-binding protein (9 KDa) in *S. cerevisiae* which was cysteine-rich (18 mol %) and high in Cd content (63 mg Cd (mg protein)$^{-1}$) and showed a high similarity in amino acid composition with Cu - MT (26). The *Cup 1* gene has been cloned into *E. coli* with a resulting expression of a functional Cu, Cd and Zn binding protein product and an increased ability for metal accumulation by the bacterium (3). An ultimate aim may be the production of different MT specific for different metals by engineering yeast strains with constitutive expression of MT genes which may, then, accumulate elevated amounts of metals (8).

The total quantity of metal ions absorbed by hybrid yeast cells (Table 11) differed greatly from one hybrid to another. It can be seen in the hybrid cells, derived from the same cross that they differed in their removal efficiencies of total heavy metals uptake. The results are in agreement with those obtained previously on uranium uptake by immobilized *Chlorella regularis* and *Streptomyces viridochromogenes* (38). On the basis of these results, further studies will be undertaken to devise a practical approach to the recovery of heavy metals from industrial effluents by genetically engineering strains. Total heavy metals removal efficiencies by yeast rose up to 43.90%. Many of the hybrids show in the presence of sugarcane refuse, greater removal of total heavy metals from industrial effluents. Total removal efficiencies up to 57.05 have been recorded. Most hybrid yeast cells uptake more than 20% of the total heavy metals presented in these effluents. The results indicated that the use of sucrose from sugarcane refuses, as a sole carbon source in pollution control technology by yeast show high metal uptake capacities from industrial effluents. It is evident that geneticists should be involved in the development of these strain protocols, especially in relation to the aspects of biotherapy of pollution. Therefore, it is essential that environmental technology in the near future becomes an established business opportunity (24).

Table 9: Transconjugation vigour for the concentration of heavy metals absorbed by bacterial cells.

Transconj-ugants	**Parameters**	**MM without sugarcane refuse**		**MM with 1% sugarcane refuse**	
		Cd	**Co**	**Cd**	**Co**
10	MP	0.04	0.06	0.02	0.00
	BP	0.04	0.06	0.04	0.00
	TC	0.02	0.08	0.05	0.14
	TCV (MP)	-50	+33	+150	
	TCV (BP)	-50	+33	+25	
11	MP	0.02	0.00	0.01	0.09
	BP	0.04	0.00	0.02	0.18
	TC	0.03	0.15	0.04	0.07
	TCV (MP)	+50		+300	-22.22
	TCV (BP)	-25		+100	-61.11
12	MP	0.04	0.00	0.01	0.03
	BP	0.04	0.00	0.02	0.06
	TC	0.02	0.15	0.05	0.14
	TCV (MP)	-50		+400	+366.67
	TCV (BP)	-50		+150	+133.33
13	MP	0.03	0.06	0.02	0.00
	BP	0.04	0.06	0.04	0.00
	TC	0.03	0.08	0.02	0.07
	TCV (MP)	0.00	+33.33	0.00	
	TCV (BP)	-25.0	+33.33	-50.0	
14	MP	0.02	0.03	0.01	0.09
	BP	0.04	0.06	0.02	0.18
	TC	0.01	0.08	0.03	0.14
	TCV (MP)	-50	+166.7	+200	+55.55
	TCV (BP)	-75	+33.33	+50	-22.22
15	MP	0.02	0.03	0.04	0.03
	BP	0.02	0.06	0.04	0.06
	TC	0.03	0.15	0.03	0.14
	TCV (MP)	+50	+400	-25.0	+366.67
	TCV (BP)	+50	+150	-25.0	+133.33

16	MP	0.03	0.03	0.02	0.00
	BP	0.04	0.06	0.04	0.00
	TC	0.04	0.15	0.04	0.14
	TCV (MP)	+33.33	+400	+100	
	TCV (BP)	0.00	+150	0.00	
17	MP	0.04	0.03	0.03	0.03
	BP	0.04	0.06	0.06	0.06
	TC	0.03	0.15	0.05	0.14
	TCV (MP)	-25	+400	+66.67	+366.67
	TCV (BP)	-25	+150	-16.67	+133.33
18	MP	0.03	0.00	0.03	0.06
	BP	0.04	0.00	0.04	0.06
	TC	0.04	0.08	0.02	0.07
	TCV (BP)	0.00		-50.0	+16.67
19	MP	0.01	0.00	0.03	0.12
	BP	0.02	0.00	0.04	0.18
	TC	0.02	0.08	0.02	0.07
	TCV (MP)	+100		-33.33	-41.67
	TCV (BP)	0.00		-50.00	-61.11
20	MP	0.04	0.03	0.02	0.00
	BP	0.04	0.06	0.04	0.00
	TC	0.04	0.15	0.01	0.07
	TCV (MP)	0.00	+400	-50	
	TCV (BP)	0.00	+150	-75	
21	MP	0.03	0.03	0.04	0.03
	BP	0.04	0.06	0.04	0.06
	TC	0.04	0.08	0.01	0.07
	TCV (MP)	+33.33	+166.67	-75.0	+133.33
	TCV (BP)	0.00	+33.33	-75.0	+16.67

MP = Mid parent BP = Better parent
TC = Transconjugant
TCV (MP) = Transconjugation vigour related to mid parent.
TCV (BP) = Transconjugation vigour related to better parent.

Table 10: Heavy metals uptake by heterozygous diploids of yeast from minimal medium of factory effluents.

Hybrids	Cadmium (Cd) Concen. (ppm)		Cadmium (Cd) Removal (%)		Cobalt (Co) Concen. (ppm)		Cobalt (Co) Removal (%)		Arsenic (As) Concen. (ppm)		Arsenic (As) Removal (%)	
	I	II	I	II	I	II	I	II	I	II	I	II
Control	0.05	0.06	0.00	0.00	0.21	0.21	0.00	0.00	0.97	1.22	0.00	0.00
H_5	0.02	0.04	60.0	33.33	0.14	0.14	33.33	33.33	0.53	1.01	45.36	17.21
H_6	0.03	0.04	40.0	33.33	0.14	0.07	33.33	66.67	0.89	0.77	8.25	36.88
H_7	0.04	0.03	20.0	50.00	0.07	0.14	66.67	33.33	0.88	0.98	9.28	19.67
H_8	0.03	0.04	40.0	33.33	0.07	0.07	66.67	66.67	1.01	1.15	+4.12	5.74
H_9	0.03	0.05	40.0	16.67	0.14	0.14	33.33	33.33	0.80	0.53	17.53	56.56
H_{10}	0.03	0.05	40.0	16.67	0.14	0.14	33.33	33.33	0.95	0.75	2.06	38.52
H_{17}	0.02	0.04	60.0	33.33	0.07	0.14	66.67	33.33	0.69	0.46	28.87	62.30
H_{18}	0.04	0.05	20.0	16.67	0.14	0.14	33.33	33.33	1.04	1.10	+7.22	9.84

+ = Increase in concentration. Concen. = Concentration

Table 11: Efficiency of biological control of pollutants by heterozygous diploid of yeast on total heavy metals uptake from minimal medium containing factory effluents.

Hybrids	Concentration (ppm) of Cadmium, cobalt and arsenic		Removal (%)	
	I	II	I	II
Control	1.23	1.49	0.00	0.00
H_5	0.69	1.19	43.90	20.13
H_6	1.06	0.88	13.82	40.94
H_7	0.99	1.15	19.51	22.82
H_8	1.11	1.26	9.76	15.44
H_9	0.97	0.72	21.14	51.68
H_{10}	1.12	0.94	8.94	36.91
H_{17}	0.78	0.64	36.59	57.05
H_{18}	1.22	1.29	0.81	13.42

CONCLUSION

This is a standard water-treatment test for the presence of heavy metal pollutants. There is now great awareness of the potential dangers of environmental pollution by heavy metal compounds which arise in waste waters from fertilizer industries (15). The removal of these pollutants from contaminated solutions by living or dead microbial biomass, and derived or excreted products, can provide an economically feasible and technically efficient means for element recovery and environmental protection (29).

The removal and recovery of heavy metals from industrial effluents by genetically engineered microorganisms have several advantages related to their greater absorbing ability, in their rapid accumulation of metals and the fact that they are inexpensive. Further development of this area of biotechnology is essential on both environmental and economic grounds, but is dependent upon adequate support from the government and industry.

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Evaluation of the potential of using an omani clay mineral as a sorbent in treatment of cationic organics

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EVALUATION OF THE POTENTIAL OF USING AN OMANI CLAY MINERAL AS A SORBENT IN TREATMENT OF CATIONIC ORGANICS

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ABSTRACT

The goal of this study is to evaluate the potential of using an Omani clay mineral called Atta-pulgite for resolving and controlling some of the environmental challenges in the Gulf region. In particular, a cost-effective procedure was investigated and developed for the treatment of contaminated water by sorption technology using Attapulgite. Attapulgite (Palygorskite) is a clay mineral that exists in the Sultanate of Oman in huge reserves in two deposits in the southern part of Oman. Through detailed geological investigations, it is approximated that about 300 million to 400 million tons exist in the al-Shuwaymiah deposit only. Presently Attapulgite is not being mined or used in Oman. This study is the ûrst comprehensive and fundamental research on the clay that attempts to develop successful applications for the material beneûting for its promising high adsorption capacity. In this paper, the ûrst ûndings was presented about this important clay in the region stressing its great potential as adsorbent to certain contaminants in water. The XRD analysis indicates that the purity of some selected samples of Attapulgite clay is very high (about 70% of the clay minerals is palygroskite and 30% Kaolinite). The preliminary Scanning Electron Microscopy (SEM) and Transmission Electron Microscopy (TEM) images clearly support this conclusion. The values of cation exchange capacity and speciûc surface area are also very promising. Batch adsorption stud-ies were performed to evaluate the adsorption performance of Methlyene Blue (MB) and Crystal Violet (CV) on the local clay mineral. The results obtained from these laboratory-scale adsorption tests indicate the strong adsorption capability of the Omani Attapulgite in reducing the level of concentration of these two dyes in water.

1 Introduction

In a very arid country like Oman, a very precious element of the Omani environment is the groundwater. It is the most important source for drinking and other essential domestic, agricultural, industrial, and commercial applications. However, with the rapid development that the country witnessed in the last three decades, groundwater contamination has become one of the major problems facing the country and its people. The contaminated groundwater is not only reducing the available water resources for use but it also poses a threat to the human health. In Oman, there are a wide variety of materials that have been identi?ed as contaminants in groundwater such as synthetic organic chemicals, hydrocarbons, inorganic cations and anions, and pathogens [1]. These contaminants originate from under storage tanks (UST), oil pipelines, septic tanks, municipal land?lls, saltwater intrusion, oil injection wells and others. Many locations in different areas have been contaminated by enormous amounts of contamination [13]. With this widespread groundwater contamination over the country, new innovated treatment technologies should be investigated and implemented.

The application of clay minerals in various processes of environmental engineering is gaining an increasing interest worldwide [4]. For example, the clay minerals bentonite and attapulgite are used as sorbents for treatment of contaminated water [59], as a slurry trench in cut-off walls [10] for environmental pollution control, and as a liner [11] in properly-designed waste land?ll. The worldwide clay markets have changed signi?cantly over the past years resulting initially in consolidation and expanded production from traditional high-grade clay sources but also creating opportunities for development of new discoveries [12]. Unfortunately, the clay minerals industry (or market) in Oman is not yet very well developed. A clay mineral that exists in the Sultanate of Oman in huge reserves is attapulgite (or palygorskite). Attapulgite has been recorded at many locations in Dhofar, the southern part of Oman: in the Hasik Member of the Umm er Radhuma formation, in the Andur Member of the Dammam formation and in the Nakhlait member of the Ashawq formation. However, it has always been a problem to locate a place with a good thickness of attapulgite beds along with low overburden thicknesses. In 1995, two sites (Shuwaymiya and Tawi Attair) have been selected by the Department of Geological Survey and Exploration for reserve estimation, taking into consideration exposure, overburden, accessibility and vegetation. The exploration con?rmed that attapulgite exists in the Sultanate in huge reserves: the Shuwaymiyah deposit contains 300 million to 400 million tons, and Tawi Attair deposit contains 200, 00 to 1 million tons.

Presently Attapulgite is not being mined or used in Oman [13]. There is not even any local fundamental research on the applications of this material. The long-term objective of our research is to evaluate the potential of using the local clay mineral attapulgite as a sorbent in treatment of hydrocarbon/heavy metals polluted groundwater. This research is the ?rst comprehensive study in Oman trying to ?nd an application for attapulgite. If successful, the mining and export of crude attapulgite to the local market as well as the AGCC or other neighboring countries is possible. However, in this ?rst paper on the Omani attapulgite, we attempt to fully quantify the Shuwaymiyah attapulgite and thoroughly investigate its potential adsorption to two selected basic dyes: methylene blue (MB) and crystal violet (CV). The paper is organized as follows. First, we provide an overview of the geologic settings of the Shuwaymiyah deposit in Southern Oman. Second, we describe the basic physical and chemical characteristics of the Omani attapulgite. Third, we discuss the kinetics and adsorption capacity of Shuwaymiyah attapulgite to MB and CV dyes.

2 Clay Deposits and Geological Settings

Attapulgite has been recorded at many locations in the southern part of Oman, Dhofar: in the Hasik Member of the Umm er Radhuma formation, in the Andur Member of the Dammam formation and in the Nakhlait member of the Ashawq formation [13]. However, it has always been a problem to locate a place with a good thickness of attapulgite beds along with low overburden thicknesses. During the execution of an exploration program for the large amounts of gypsum discovered in a Shuwaymiyah area, it was easily recognized that huge reserves of attapulgite clay occur in this coastal area. The Shuwaymiyah attapulgite deposit is located in the Shuwaymiyah and Thadbut topographic sheets that occur about 3.5 km NW of Shuwaymiyah village on the sea shore. The Shuwaymiyah area constitutes a closed coastal plain about 30 km long and 4-5 km wide. The plain is bounded to the south by the sea, and the north by two sharp, successive scraps; the Dammam Formation and the Aydim Formation. To the east and west the scrap rises directly from the shore. The area is drained by numerous, south-?owing wadis, the biggest of which is Wadi Shuwaymiyah. These wadis dissect the plain into numerous very low lying hills.

The geological map of Shuwaymiyah area (Figure1a and b) shows the distribution of the Andhur Member in Shuwaymiyah area, which more or less represents the distribution of attapulgite at the foot of the Shuwaymiyah scarpment. The Andhur Member forms a gentle slope along which a substantial amount of attapulgite can be easily scraped without having to remove much of over-burden. In addition to the occurrence of attapulgite along the foot of the scrap, a good part of the Quaternary deposits on both sides of Wadi Shuwaymiyah cover low lying hills representing the lower part of the Andhur Member and containing a lot of easily quarryable attapulgite deposits.

The department of Geological Survey and Exploration in the Directorate General of Petroleum and Minerals has selected in 1995 two sites (site-1 and site-2) for reserve estimation, taking into consideration exposure, overburden, accessibility and vegetation. The lithologic section shown in Figure.1b was logged in site-2. The lithologic structure of site-1 is very similar to site-2. It is clear from the ûgure that the Andhur Member is composed of several clay layers (mainly attapulgite) varying in thickness with interbedded layers of chalky marly limestone. In site-1, the attapulgite horizons range in thickness from 0.4m to 3.5m with an average of 1.1m. The site is estimated to contain about 321 million tons of attapulgite and an interbedded limestone of 264.6 million tons over an area of 6.7 km^2. In site-2, the limestone beds range in thickness from 0.2m to 0.6m, with a total thickness of 3m. The attapulgite beds range in thickness between 0.2 m and 2 m. Their total thickness is 7.1 m. Site-2 contains about 33.6 million tons of attapulgite and an interbedded limestone of 18.7 million tons over an area of 2.4 km^2.

3 Mineralogical, Chemical and Physical Characteristics of Shuwaymiyah Attapulgite

In this section, we characterize the Omani attapulgite clay mineralogically (X-ray diffraction and electron microscopy), chemically (oxides composition), and physically (cation exchange capacity and speciûc surface area).

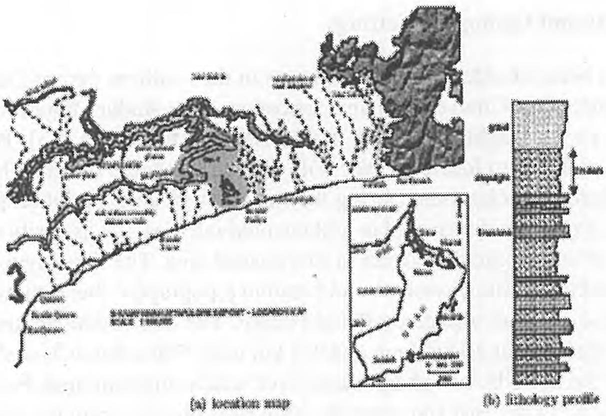

Figure 1: Al-Shuwaymiyah attapulgite deposit in Oman

3.1 Experimental Procedures

The crude clay has been dried and mechanically crusted and sieved to pass the 200 sieve (75m). Then, the sample was washed in order to remove carbonates, soluble salts and organic matter. The clay ?ne fraction was separated by centrifuge. These processes of clay washing and separation were implemented following the detailed procedure described in [14]. Oriented clay samples on glass slides have been prepared for phase identi?cation by X-ray diffraction (XRD). The X-ray diffractometer used in this analysis was Philips PW1710 automated powder diffractometer, with a generator settings of 40 kV, 40 mA and nickel-?ltered CuK?(?=1.54056A) radiation over 2O range of 2.5 - 70 at a scanning rate of 2 min-1 . Ethylene glycol and heat treatments were used to provide additional information essential for the identi?cation of clay minerals. Size and morphology of particles were determined using Jeol JSM-5600LV 20 kV scanning electron microscope (SEM) and Joel JEM-1230 120 kV transmission electron microscope (TEM). Oxides content in the attapulgite samples has been measured using wet chemical analysis. The cation-exchange capacity (CEC) was determined by ammonium acetate exchange. The speci?c surface area (SSA) of the clay mineral was determined using the N2 -BET with Quantachrome Autosorb Automated Gas Sorption Instrument. These two properties have been also determined by the Methylene Blue (MB) adsorption method developed by Hang and Brindley [15]. Details of the experiential procedure followed for MB adsorption is described in Section4.

3.2 XRD

Clay phase identi?cation has been examined in six forms: as an oriented clay sample (untreated), as an oriented clay sample treated with ethylene glycol, as an oriented clay sample heated to 350C, 600C, 700C, and 800C for two hours. The X-ray powder diffractogram of the Shuaimiah Attapulgite (Figure.2) shows the presence of two main sets of phases: palygorskite and kaolinite. The d-values of palygorskite are at 10.5, 6.3, 5.5, 4.5, 4.2, 3.7 and 3.2 Å, whereas the d-values of the kaolinite are at 7.1 and 3.6 Å. The ?gure also clearly shows that both minerals are unaffected by glycolation. When subjected to heat treatment at 350C, the intensity of the main re?ection decreases for both minerals. At 600C heat treatment, the d-value 10.5 Å for palygorskite and 7.1 A for kaolinite completely disappeared.

The constant standard minerals method [16] was used to quantify the percentages of palygorskite and kaolinite minerals in Shuwaymiyah attapulgite clay. Different percentages of clay were added to constant standard minerals of equal weights. All the prepared samples were treated by ethylene glycol. The peak area ratio was calculated from X-ray diffraction of the standard mixes with the clay. The mineral peak area ratio against the percentages of clay is linearly best ?tted and the minerals percentage was calculated from the slope of the ?tted line. The estimated percentage of palygorskite in Shuwaymiyah attapulgite is in the range of 67 to 72%.

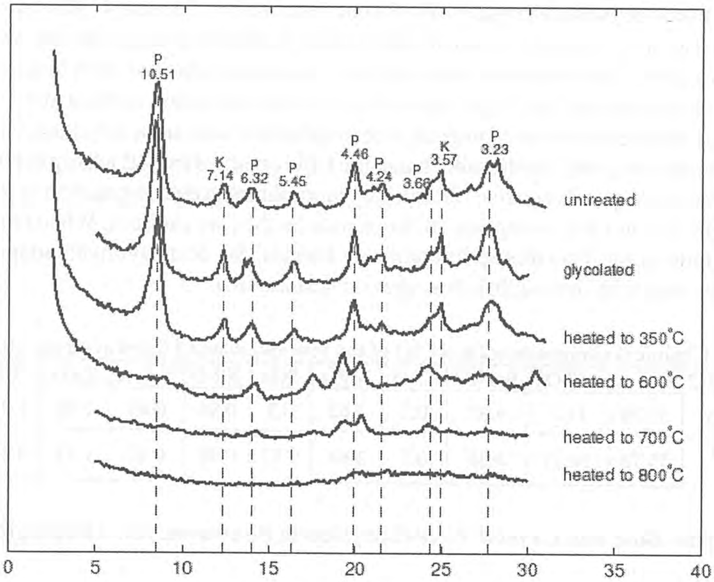

Figure 2: X-ray diffraction patterns showing the characteristic peaks of palygorskite (P) and kaolinite (K) from representative samples of Al-Shuwaymiyah attapulgite deposit in Oman

3.3 Electron Microscopy

Electron microscopy is an indispensable tool for observation of morphology, size and texture of the individual particles of clay minerals, particulary palygorskite [17, 18]. Selected SEM images of Al-Shuwaymiyah attapulgite are shown in Figure3 and TEM images in Figure.4 The micrographs shown in the right hand side are always obtained at a higher magni?cation than the corresponding micrographs in the left hand side. The micrographs clearly re?ect the abundance of palygorskite ?bers in Al-Shuwaymiyah attapulgite. Individual palygorskite ?bers are 0.2 to 0.5 μm long (Figure 4c and d). However these individual ?bers often form long palygorskite bundles that are 5 to 10 μm long (Figure.4a and b). As seen from the SEM images, the palygorskite ?bers may exist as individual well separated ?bers (Figure 3a and b) or as elongated bundles of many ?bers (Figure 3c and d) or sometimes circular aggregates of clay ?bers (Figure 3e and f). Due to such variations in the existence of palygorskite ?bers, the pore space in the formation consists of tiny pores between the individual ?bers and larger pores between the bundles of ?bers (Figure 3c and d).

3.4 Physio-Chemical Properties

The chemical composition of the raw and washed Shuwaymiyah attapulgite is reported in Table 1. The table shows that the main constituent of the clay is SiO2 (53-55%). This high content of silica in attapulgite gives its structure credibility as a good adsorbent. It is also observed that the difference in chemical composition between raw and washed Shuwaymiyah attapulgite is minimal. The measured surface area and the cation exchange capacity of Al-Shuwaymiyah attapulgite are reported in Table 2. The values are also compared with those estimated from the MB adsorption experiment described in Section 4. MB adsorption is largely used as an ef?cient procedure for determining cation exchange capacity and speci?c surface area [19]. The difference between the determined values of SSA obtained by the two methods is expected since it is known that the MB adsorption method allows measuring the total surface area of clay mineral, whereas the BET technique measures the external speci?c surface area only. On the other hand, the CEC obtained by MB adsorption is slightly less than the corresponding value obtained by ammonium acetate extraction re?ecting the fact that MB was not able to replace all the cations on the clay surfaces. When compared to the attapulgite in the Hawthorne formation in Florida, the Shuwaymiyah attapulgite has exactly the same CEC but slightly less speci?c surface area.

Table 1: Chemical composition (in wt.%) of the raw and washed Shuwaymiyah attapulgite

SiO2		Al2 O3	Fe2 O3	CaO	MgO	SO3	K2 O	Na2 O	TiO2	CI	LoI
Raw Clay	53.28	14.2	4.93	0.2	3.62	0.3	0.94	0.84	1.06	1.02	19.09
Washed Clay	55.76	14.13	4.78	0.1	3.64	0.23	0.96	0.32	1.12	0.23	17.69

4 Methylene Blue and Crystal Violet Sorption to Shuwaymiyah Attapulgite

In this section, the adsorption of natural Shuwaymiyah attapulgite clay powder to the organic cations methylene blue (MB) and crystal violet (CV) is studied. First, the adsorption experimental procedure is described. Then, the kinetic experiment is discussed. Finally the derivation of adsorption isotherms is presented.

Table2: Specific surface area and cation exchange capacity of the Shuwaymiyah attapulgite

	SSA (m2 /g)		CEC (meq/100g)	
	BET	MB	Ammonium Acetate	MB
Al-Shawaymiah attapulgite (Oman)	90	125	19	16
Hawthorne attapulgite (Florida, USA)	136.35		19.5	

4.1 Experimental Setup

Sorption experiments were performed using the standard batch technique. Initially, kinetic experiments were performed to study the effect of contact time on the adsorption processes. In the kinetic experiment, ûxed mass of attapulgite (30 mg) and ûxed concentration of dye (100 mg/l) were added in 40-ml borosilicate glass bottles with Teûon-lined caps. The mixture total volume was set to 20 ml. Several identical bottles were placed on a specially designed end-over-end shaker with ûxed speed motor. At speciûed times, bottles were removed from the shaker and centrifuged at 3000 rpm for 30 minutes. Then, the dye concentration in the supernatant was measured using UV-visible spectrophotometer (PerkinElmer) at wavelength of 665 nm for MB and 580 nm for CV.

For the adsorption isotherms, one set of several 40 ml glass bottles containing constant attapulgite clay mass (30 mg) and variable initial concentrations of dye (5 to 100 mg/l) was prepared. A second set of glass bottles containing the dye with constant initial concentration (100 mg/l) but variable attapulgite clay mass (10 to 90 mg) was simultaneously prepared. The volume of the mixture was also set to 20 ml. Both sets, along with blank bottles (bottles containing MB without attapulgite clay) were placed on the shaker overnight (18 hrs) at room temperature (25°C) in order to reach equilibrium. Then, the bottles were centrifuged at 3000 rpm for 30 minutes. The equilibrium concentration of MB was measured using the UV-visible spectrophotometer at the previously mentioned wavelengths. In all cases, dye solutions were prepared using distilled water. All experiments were performed in duplicates. The amount of dye adsorbed on the surface of attapulgite, q_e (mg/g), was obtained as follows:

$$q_e = \frac{(C_i - C_e)\,V}{M} \tag{1}$$

$$\frac{dq(t)}{dt} = k_1\left[q_e - q(t)\right] \tag{2}$$

Where k1 [1/min] is the rate constant of pseudo-first-order model. When integrating over time and linearizing the model, the following equation is obtained

$$\ln\left[q_e - q(t)\right] = \ln\left[q_e\right] - k_1 t \tag{3}$$

The pseudo-second-order equation based on adsorption equilibrium capacity is expressed in the form

$$\frac{dq(t)}{dt} = k_2\left[q_e - q(t)\right]^2 \tag{4}$$

Where k_2 [g/mg.min] is the rate constant of pseudo-second-order model. With a similar manner performed for the previous model, the linearized form of the variation of clay uptake with time is

$$\frac{t}{q(t)} = \frac{1}{k_2 q_e^2} + \frac{1}{q_e}t \tag{5}$$

Figure 5 summarizes the results obtained from the kinetic experiments of MB and CV. Figure 5a plots the attapulgite clay uptake of MB and CV at various contact times. One can immediately observe the fast decrease in residual dye concentration at a short time scale (just a few minutes) implying the strong electrostatic interaction between the negatively charged surface and the basic dye cations. The CV adsorption requires longer time to reach equilibrium than MB. The estimated equilibrium time for MB adsorption is 60 minutes, whereas the estimated equilibrium time for CV is about 200 minutes.

The validity of the two kinetic models was checked using linear plots of ln [q_e “ $q(t)$] *vs t* (Figure 5b) for the pseudo-ûrst-order and $t/q(t)$ vs t (Figure. 5c) for the pseudo-second-order model. Even though the pseudo-ûrst-order model provided a good ût (R^2 =0.90) in the early time stages, the experimental data has shown considerable deviation at later times. However, on the other hand, pseudo-second-order model has shown almost perfect ût (R^2 =0.9999) in the whole range of considered time. The rate constant of the pseudo-ûrst-order model is k_1 =0.035 [1/min], whereas the corresponding rate constant for the pseudo-second-order model is K_2=0.031 [g/mg.min]. This perfect ût of the pseudo-second-order kinetic model suggests that the model can be efûciently used to predict the kinetic of adsorption of MB or CV by attapulgite. Since this model is based on the adsorption capacity, it will help to predict adsorption behavior over the whole range of concentration and supporting chemisorption (chemical reaction) as the rate controlling mechanism. Chemisorption usually indicates that the adsorption process is irreversible. Views of a similar kind have been put forward by other workers [20–22] who had studied basic dyes adsorption.

4.3 Adsorption Isotherms

When the system is in the state of equilibrium, distribution of dye between the adsorbent and the dye solution, it is important to establish the absorbtion isotherms and the capacity of the adsor-bent to the dyestuff. Figure 6a shows the experiential curves of equilibrium clay uptake (*qe*) and equilibrium concentration (*Ce*) for adsorption of MB and CV. Both dyes show strong afûnity for attapulgite clay. In both cases, the clay uptake increased sharply with increasing the solution con-centration from 0 to about 5 mg/l to reach an adsorption capacity of 40 mg/g. Only beyond this equilibrium solution concentration, the adsorption behavior of the MB and CV starts to deviate. In the case of MB, this sharp increase in adsorption is followed by a gradual increase in the adsorption capacity in the regions from 5 to 20 mg/l beyond which the adsorption is leveled to a maximum value equal to 51 mg/g. In the case of CV, the gradual increase in the adsorption continues with increasing the concentration of the solution. The largest adsorbed amount of CV was 70 mg/g.

Table 3: Isotherm constants for the adsorption of MB and CV onto attapulgite clay

	MB	CV
Langmuir:	q_{max} = 50.76	57.8 mg/g
	b = 1.7	1.2 l/mg
	R^2 = 0.999	0.854
Freundlich:	K_f = 40.15	34 (mg/g)(l/mg)1/n
	1/n = 0.06	0.17
	R^2 = 0.722	0.975

The adsorption isotherms that are usually used for describing dye adsorption are the Langmuir and the Freundlich. The linear transformation of the Langmuir isotherm is

$$\frac{C_e}{q_e} = \frac{1}{q_{max}} C_e + \frac{1}{b\ q_{max}} \tag{6}$$

Where q_{max} and b are Langmuir constants related to maximum adsorption capacity and energy of adsorption, respectively. Using a least square linear fit of C_e/q_e versus C_e, the two unknown parameters can be determined from the slope of the line ($1/q_{max}$) and the intercept ($1/[bq_{max}]$). On the other hand, the Freundlich isotherm is linearly transformed to

$$\ln(q_e) = \frac{1}{n} \ln(C_e) + \ln(K_f) \tag{7}$$

Where K_f and $1/n$ are Freundlich constants related to adsorption capacity and intensity of adsorption, respectively. A linear fit of $\ln(q_e)$ versus C_e would give $\ln(K_f)$ (intercept) and $1/n$ (slope). The value of n is usually greater than 1 and must reach some limit when the surface is fully covered.

The goodness of fit of the Langmuir model to the MB and CV experimental data is shown using the liner plot in Figure 6c, whereas the goodness of fit of the Freundlich model is shown in Figure 6d. The parameters of the fitted models with their regression coefficient are tabulated in Table 3. Visual inspections and values of the regression coefficients yield the conclusion that MB ad-sorption is best fit by Langmuir isotherm, whereas CV adsorption is best fit by Freundlich isotherm as summarized in Figure 6b.

5 Conclusion

A very important clay mineral in the Sultanate of Oman has been characterized. The mineralogical investigation on Al-Shuwaymiyah attapulgite clearly indicates its high purity. Quantification anal-ysis of XRD revealed that the percentage of palygorskite in the clay is at least 70%. SEM and TEM images clearly indicate the existence of palygorskite as a dominant mineral in Al-Shuwaymiyah clay. Al-Shuwaymiyah attapulgite was used as an adsorbent for the removal of Methylene Blue (MB) and Crystal Violet (CV) basic dyes from water. The adsorption equilibrium revealed that Al-Shuwaymiyah attapulgite can uptake up to 51 mg of MB and 70 mg of CV per one gram mass of clay. MB adsorption is best fit by Langmuir isotherm, whereas CV adsorption is best ût by Freundlich isotherm. A pseudo-second-order kinetic model can be efficiently used to predict the kinetic adsorption of MB and CV by the attapulgite. This naturally occurring material could substitute in the Gulf region the use of activated carbon as adsorbent due to its availability and its low cost.

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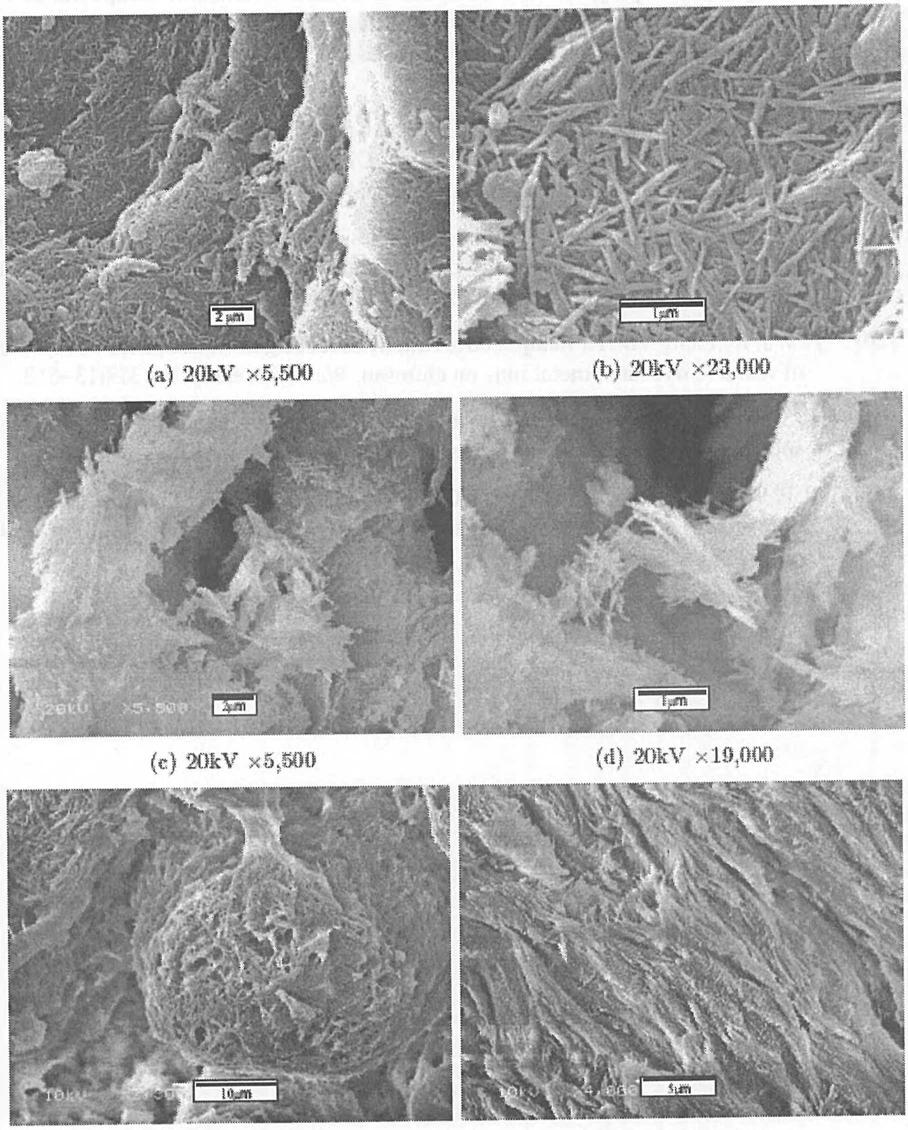
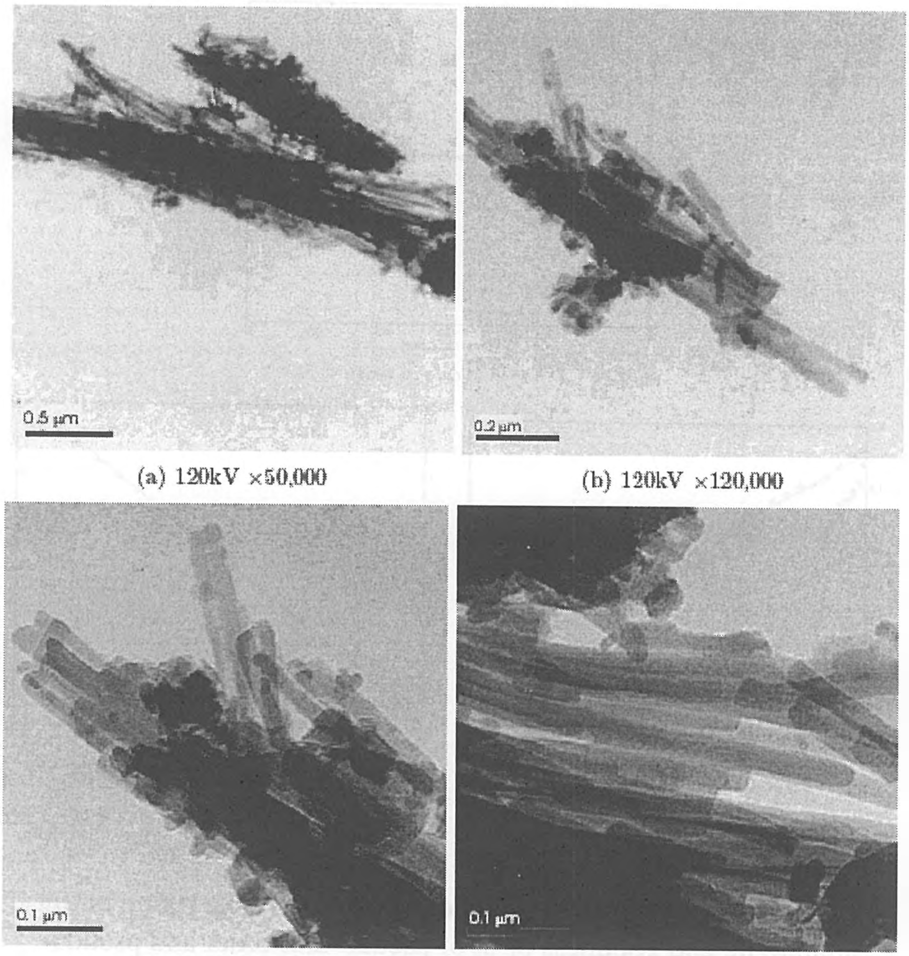


(a) 20kV ×5,500 (b) 20kV ×23,000

(c) 20kV ×5,500 (d) 20kV ×19,000

(e) 10kV ×2,300 (f) 10kV ×4,000

Figure 3: SEM images of palygorskite ûbers and aggregates in Al-Shuwaymiyah attapulgite



(a) 120kV ×50,000

(b) 120kV ×120,000

(c) 120kV ×250,000

(d) 120kV ×300,000

Figure 4: TEM images of palygorskite ûbers and aggregates in Shuwaymiyah attapulgite

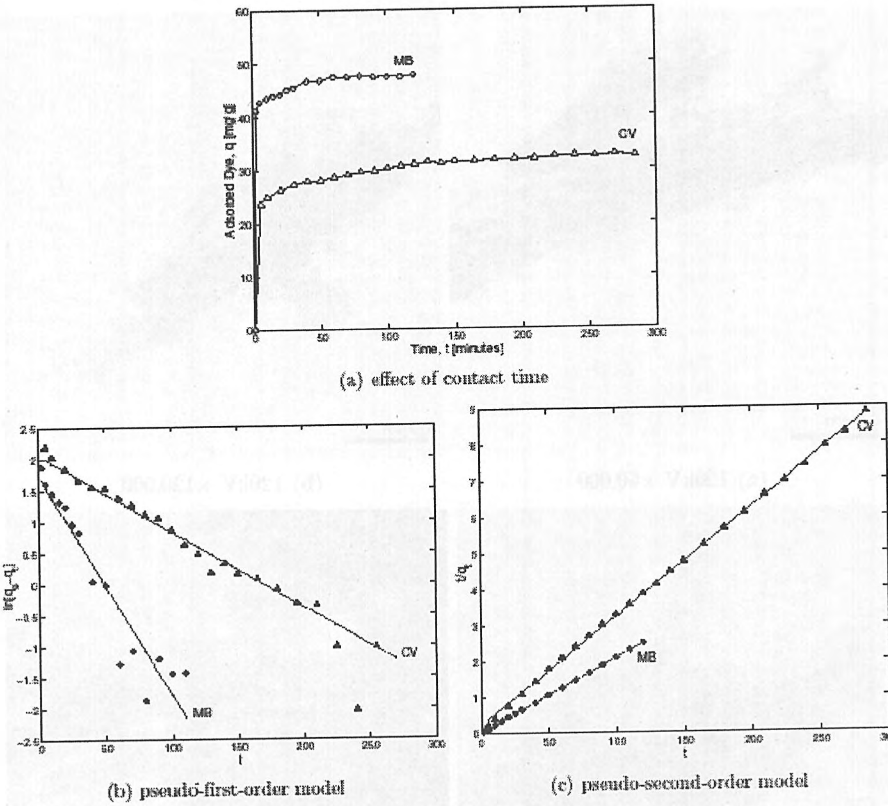
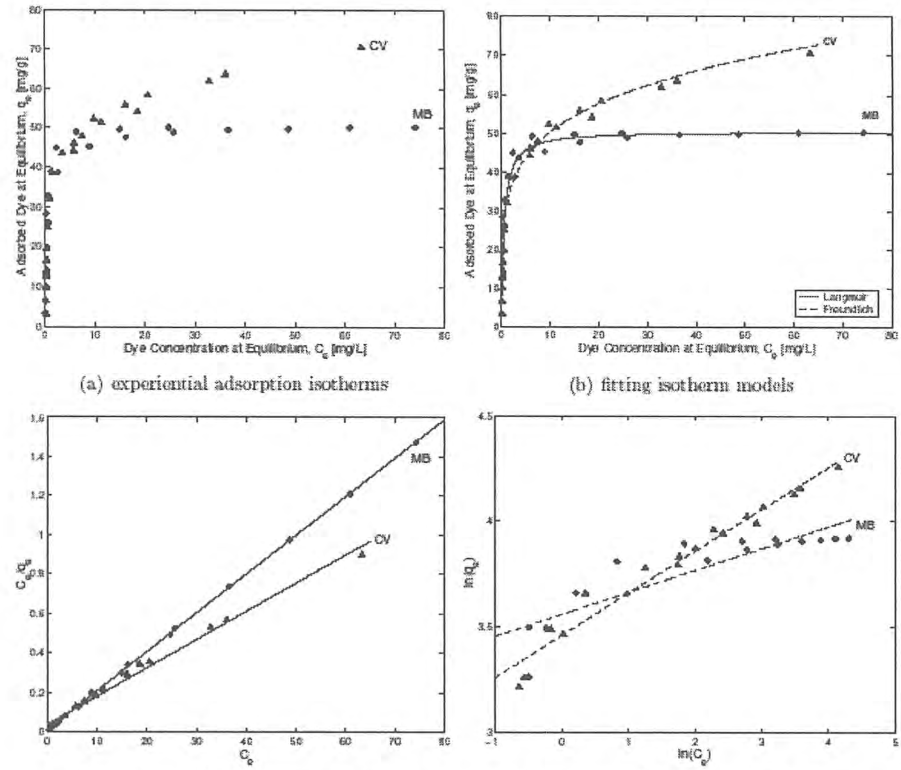


Figure 5: Kinetic experiments of MB and CV adsorption onto attapulgite and goodness of ût of pseudo-ûrst-order and pseudo-second-order models to the experimental data



(a) experiential adsorption isotherms

(b) fitting isotherm models

(c) goodness of fit of Langmuir isotherm

(d) goodness of fit of Freundlich isotherm

Figure 6: The variation of adsorbed amount of dye with the equilibrium concentration of dye solution and their goodness of ût to the theoretical Langmuir and Freundlich isotherms

Integrated catchments management project
Case study: Wadi al ma'awail catchment

Ali Mohammed Al Abri and Rashid Khalfan Al Subhi

INTEGRATED CATCHMENTS MANAGEMENT PROJECT CASE STUDY: WADI AL MA'AWIL CATCHMENT

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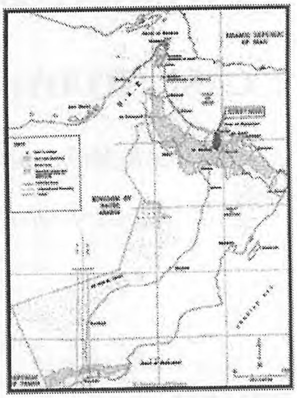
Ministry Of Regional Municipalities, Environment And Water Resources.
Sultanate of Oman

ABSTRACT

Wadi Ma'awil Integrated Catchment Management Plan (ICMP) is a leading example of how a genuine partnership between a community and the government can address current water management problems on a catchment scale. When implemented, the Wadi Ma'awil Integrated Catchment Management Plan will be the first such comprehensive work in the region. All future water management within the Ma'awil catchment should fall within the ICMP framework. Modern principles and concepts relating to formulation of an ICMP were identified during the review phase of the study to address water management issues such as Establishment of safe yield threshold, Establishment of water development limits, Reduction in current water use, particularly in agricultural sectors, Control of saline water intrusion, Improvement of on-farm water use through introduction of conservation measures and ?Protection of groundwater from contamination.

The objective of the Wadi Ma'awil ICMP is to achieve sustainable agricultural productivity while protecting and enhancing the catchment's environment. The focus has been on methods to reduce groundwater extraction and reverse saline water intrusion into the groundwater reservoir. The review of international practice on Integrated Catchment Management Planning (ICMP) shows that the key to a successful implementation is the peoples' participation in decision-making. In an ICMP approach, participation of the people or stakeholder is achieved through formulation of a Catchment Management Committee (CMC) representing the interests of all people in the catchment and working in partnership with Government agencies to implement a broad range of accepted land and water management measures. Ideally, the government is to provide significant technical and financial resources while the community is to provide local experience and knowledge

Introduction

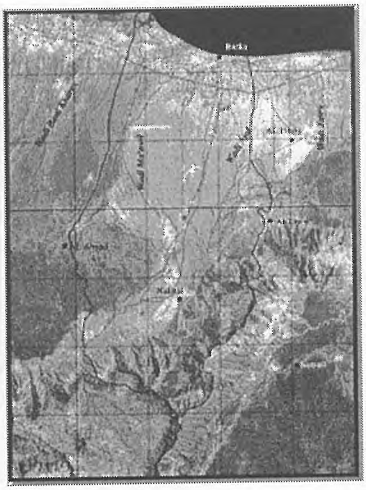


Wadi Al Ma'awil catchment, located in the eastern part of the Al-Batinah Region (Fig. -1) of the Sultanate of Oman, it is part of the Barka Water Assessment Area as classified by the Water Resources Department of Ministry of Regional Municipalities, Environment and Water Resources. Administratively, Wadi Ma'awil catchment includes all or parts of the Wilaya'a of Nakhl, Wadi Ma'awil and Barka. The study area extended from the grid coordinates 569200E to 597400E and 2574000N to 2630600N. The catchment lies to the north of the mountains and extends across to the costal plane to the Gulf of Oman. Wadi Ma'awil covers an area of about 1300 km2 and is known by different names along its course. The total population in 1993 was 10,102 in the upper wadi catchment and 40,283 in the lower catchment. The largest population centers are the towns of Nakhl (7,307) and Muslimat (2,054) and Barka Wilayat Centre (12,484).

The main reasons for selecting this Catchment are:

- *The catchment has within a wide range of terrains, water sources and uses.*
- *All the potential water supply groundwater and aflaj.*
- *Catchment display problems due to Sea water intrusion.*
- *The catchment has enough data for the study.*

Objectives



The principal objective of the study is the development of an Integrated Catchment Management Plan (ICMP) methodology for water management at the Catchment level. The management of catchments for sustainable multiple use requires a knowledge of both the biophysical resource base and societal values associated with its use. Therefore, broad objectives pursued under this study include:

- Review and analysis of hydrological and hydrogeological data available for Wadi Ma'awil catchment.
- Analysis and categorisation of water use pattern in the catchment.
- Estimation of catchment water balance.
- Estimation of the socio-economic and ecological value of groundwater in the catchment and nature and scale of the consequences of its unsustainable use.
- Establishment of supply and demand management measures.

Water Resources

Government agencies carried out exploratory drilling to assess the groundwater potential of the area and drilled 47 exploratory wells. The Ministry of Housing, Electricity, and Water (MHEW) also drilled several wells to supplement the local water supply. MRMEWR carried out a detailed national well inventory project (NWIP) between 1990 and 1993. Statistical data on agricultural and domestic water use in this study has been derived from the NWIP database.

Other than wells, aflaj and springs are major sources of water to meet agricultural and, to some extent, domestic water demands. Within the upper catchment, discharge from the Al Thowara hot spring is run through a series of aflaj to supply agricultural water demands in Nakhl and downstream villages. Several ghaily aflaj originate at the tail end of the upper falaj and re-circulate the same water (e.g. Falaj Hubra and Falaj Wasit). The aflaj provide steady flow throughout the year as they are fed by spring water. Wells existing in the upper catchment are occasionally used in the event of emergency and remain mostly on "standby" status. Hence, there is practically little or no pumping from these wells. (Table -2) shows the number of dug and bore wells .

Table-2: Well Statistics in Wadi Ma'awil Catchment

Type	Category	Total
Dug wells	Operational	3,522
	Backfilled	301
	Disused	1,017
	Total	4,840
Bore wells	Operational	1,055
	Exploratory	47
	Backfilled	2
	Disused	200
	Total	1,304

The dug wells and bore wells used for agriculture are shown in (Table-3). In total, 2,754 wells/boreholes are in use to irrigate agriculture land.

Table -3: Number of Dug and Bore Wells Used for Agriculture

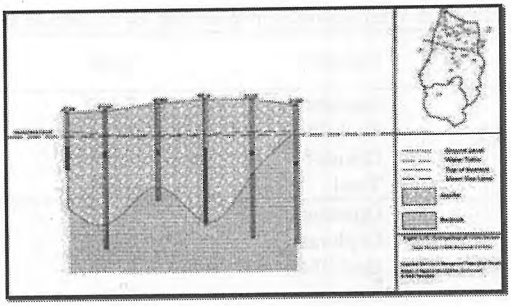
Type	Total
Dug well	1,833
Borehole	921
Total	2,754

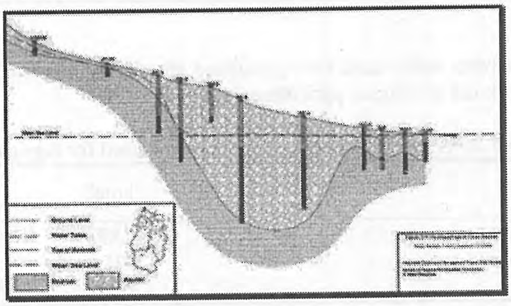
Total abstraction from the operational dug wells and boreholes is estimated at 94.93 MCM/yr in Wadi Ma'awil. This volume is 40.38 MCM/yr greater than the estimated 54.55 MCM/yr of replenishable resource. The over-development of groundwater has resulted in seawater encroachment over a wide area in the northern end of the catchment in response to the decline in water levels below sea level.

Characteristics Of The Main Aquifer In The Study Area

Wadi Ma'awil aquifer is formed by three units of gravel. The upper clean gravel and sand with boulders unit is fully saturated near the coast. The saturated thickness of the alluviam varies from 10 m in the southern part to 60 m near the coast. In the central part of the catchment, it generally varies from100 to 240 m but can reach a thickness exceeding 300 m.

Hydrogeological Cross Section





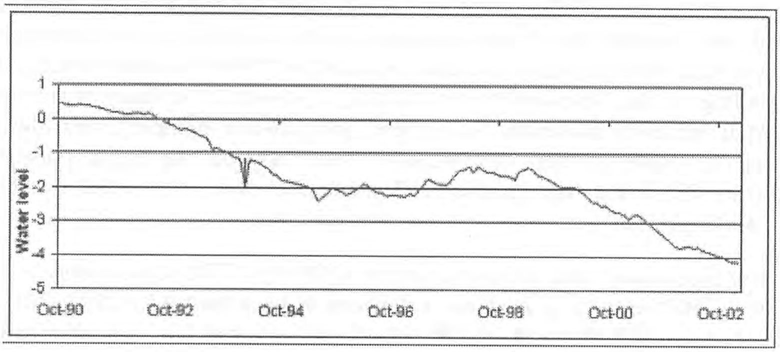
Existing Infrastructure

The existing data recorded at Wadi Ma'awil dam shows that no outflow from the dam reservoir occurred during the period. Since inception of the Wadi Ma'awil dam in Oct 1991, 32.98 MCM of wadi flow has been intercepted until September 2000, averaging at 3.64 MCM/year. It is assumed that except for the evaporative losses (20% in the short residence period in dam), all water has percolated and recharged the aquifer. Thus, 2.90 MCM of recharge is achieved by the dam.

Groundwater Levels

Monitoring well in Barka SSA1

The Ministry of Regional municipalities, Environment and Water Resources (MRMEWR) regularly monitors water levels in 37 boreholes located across the catchment. Depth to water measurements (DTW) for the year 2000 is shown in Table 2. The DTW values range between near surface at the coast to over 85 m below ground level (mbgl) in the centre of the mid-lower catchment. In the upper catchment, the depth to water level is generally shallow (less than 20 m) and does not show much fluctuation since groundwater abstraction is minimal. There are only few monitoring wells in the upper catchment. The hydrograph of the monitoring dug-well (SSA1) shows an average amplitude of 5 m. Rise in water level is in direct response to wadi flow losses as indicated by the sharp rise and equally steep decline within a short period. In the lower catchment, depth to water becomes shallower reaching close to ground surface near the Coast.



Water Quality

Historically, groundwater quality was generally very good in the Wadi Ma'awil catchment including the coastal plain. This could be attributed to the small volumes of water withdrawn for consumptive use prior to 1970. However, with the installation of mechanical pumps on existing and new wells since that time, a rapid expansion of agriculture has occurred. This has resulted in a sharp increase in groundwater withdrawal. As long as groundwater inflow from upper reaches met the pumping requirements, balance was maintained. Once groundwater abstraction started exceeding groundwater inflow, water levels began to decline and landward movement of seawater started.

The expansion of agriculture took place in the coastal plain initially. Over time, an increasing number of larger-scale, profit-oriented farms were developed south of the main highway. These farms required deeper wells and more powerful pumps to generate income for the investors in the farming operations. Increased pumping has lead to the development of a reverse groundwater gradient and further seawater intrusion and salinity in the costal area increased and destroyed the farms. After completion of the National Well Inventory Project (NWIP) in 1993, the expansion of agriculture was stopped and a ban on construction of new wells was strictly enforced.

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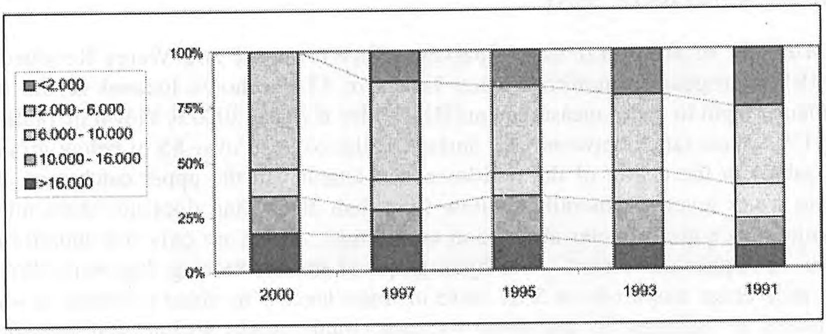


Groundwater Abstraction

MWR carried out the NWIP in 1993 to record water use for domestic, agricultural, industrial, and livestock use. Later on, under the Metering Project, actual pumping on was measured at selected wells. The data provided by NWIP has been modified based on the findings of the Metering Project.. Table _3 presents a summary of water usage in the Wadi Ma'awil catchment. At present, groundwater pumping from the lower catchment is estimated to be 94.93 MCM/yr. Water table decline and deterioration of water quality due to seawater intrusion reflects an over-abstraction which is estimated to be 40.43 Mm3/year.

Hardly any crop except date palm can sustain 6500 mg/l TDS. Considering this TDS limit, about 3860 hectares of land was calculated to be salinized by 1993. This value increased to reach 5330 hectares in 1999. The loss of cropped area was estimated to be 1172 hectare. Based on the salinity survey, it is concluded that on an average about 245 hectares of land is lost per year of which 54 hectare is cropped area. By the year 2020, if similar conditions continue, a total of 10,475 ha (104 km2) of land with 2304 ha (23 km2) of cropped area will be lost to salinization.

Water Demand

It is essential to quantify the availability of water resources with respect to the current and future water demand to work out an effective water management plan. Demand in Wadi Ma'awil catchment includes water use for domestic supply, livestock, industry and agriculture. Water demand is summarized in the Ttable –4 bellow:

Table -4 Consumptive Use of Groundwater in Wadi Ma'awil

No	Catchment	Agriculture	Livestock	Domestic & Municipal	Industrial	Total	Remarks
1	Upper	1.662	0.0072	0.722	0.078	2.469	Falaj supported supply
2	Mid lower & Lower Coastal	91.657	0.203	2.547	0.520	94.927	Well Abstraction
3	Total	93.319	0.2102	3.269	0.598	97.396	

Table -5: Water Balance Scenario (MM3/YEAR)

	%	Upper Catchment	M-Lower Catchment	Lower Catchment	Total (MM^3)
Rain Volume	100	50.38	41.65	12.98	105.01
Evapotranspiration	56.	29.69	20.82	8.43	58.94
Direct Recharge	29.8	17.63	10.41	3.25	3.25
Dam Recharge	2.8		2.9		2.9
Return Seepage					13.75
Indirect recharge			2.14		2.14
Wadi Flow	10.0	3.06	7.22	0	10.28
Sea water intrusion				8.44	8.44
Through flow (in flow)	2.0	12.46	1.3	8.29	22.05
Storage depletion					32.0
Consumptive Use		94.93			94.93
Water Balance					-40.43

Combined Supply And Demand Management Scenario

A combined supply and demand management approach has been proposed in this study to tackle the problem. These measures are briefly described below;

1. Modernization of Irrigation Practices: Switching over from conventional flood irrigation systems, presently practiced in 2333 hectares, to a modern irrigation system (MIS) will save 9.13 MCM of the water per year at an investment of 100 OMR/ha excluding subsidy with an additional 50 OMR/ha for public awareness and support

2. Crop Pattern Change: Changing from grass crops to vegetables in the freshwater zone (TDS<1500 mg/l) and to cereals in the brackish water zone (TDS>4000 mg/l) will save 8.8 MCM of water per year at an investment of approximately 500 OMR/ha to encourage the farmers through extension services of the Ministry of Agriculture and Fisheries.

3. Abstraction Control: Control of water abstraction from agricultural wells by installing water meters can save 11 Mm3 of water per year at an investment of 1 million OMR and an additional annual expense of OMR 30,000 for repair and maintenance. The cost of installing meters (1 million OMR) could be born by the Government while operating cost and any replacement cost should be born by the well owner. Thus, the government will spend only 105 Bz/m3 of water savings whereas well owners will spend 6.6 OMR/yr or 3 baisa/m3 of water saved

4. Wastewater Reuse: The projected total water demand comprising of domestic, livestock and industrial usage for the lower catchment is expected to be 5.77

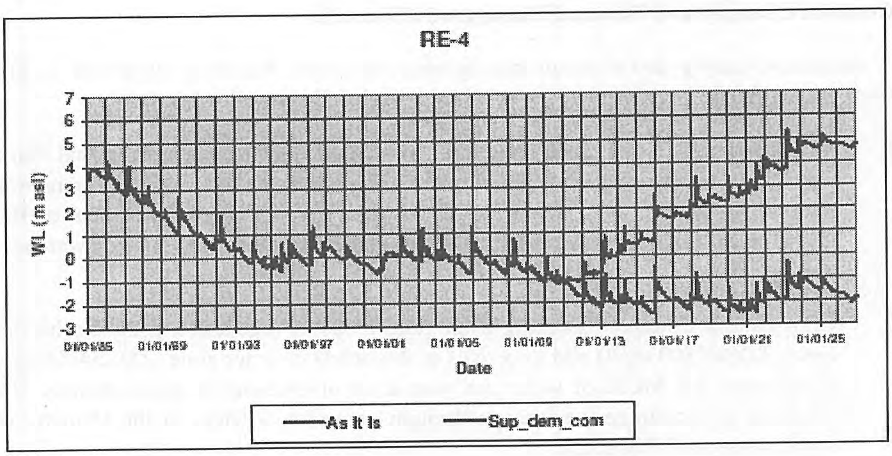
MCM/yr by 2020. This will generate about 5.2 MCM of effluent that has to be treated. Once treated, 4.9 MCM water can be beneficially used for irrigation and recharge of groundwater.

5. Desalination: Use of desalinated water for domestic and industrial purposes can save up to 5.77 Mm3 of abstraction per year. About 90% of this water can be reused by recycling in the treatment plant. The cost of the production may not exceed OMR 0.500/m3.

6. Change in Land Use through Land Purchase: If all the measures recommended above are applied, the water deficit may ultimately be surmounted around 2015. However, a ban has to be imposed on active agriculture nearest to the coast that has turned highly saline (TDS>6500 mg/l). Alternatively, Government may purchase the land in this zone and use for other development purposes.

According to the results of the groundwater model developed for this study it is expected that the saline front will be stabilized with the introduction of remedial measures by 2010. By 2025, water levels are predicted to rise above the 1985 water table and saline water intrusion will stop. The rising trend of water table is illustrated in Figure 3 below.

Figure 3: Effect of Management Measures on Groundwater Level



Socio-Economic

The objective of the socio-economic analysis is to study the agricultural conditions, both current and future, with and without the ICMP project. Agriculture is not possible in Oman without irrigation. Although Oman has an oil-based economy, it is essential to use all other resources efficiently for its sustainable growth. Water is one such resource. Growing water scarcities is a challenge to sustainable economic development.

About 96% of the total water demand is consumed by agricultural sector, which contributes only 3% to the country's economy. This gives the impression that water shortages do not have significant impact on the economy.

Considering the socio-economic and agricultural characteristics, Wadi Ma'awil is one of the important regions in the Sultanate and may be treated as an agriculture resource. Its tremendous economic potential has not been exploited due to inefficient use of water. Inappropriate management of irrigation has resulted in environmental problems including excessive water depletion, water quality reduction, and salinization. On the other hand, population and economic growth are boosting the water demand. To meet this growing demand efficient utilization and management of water resources is critical.

Low prices encourage wasteful and inefficient use of water. Therefore, the price elasticity of water is a very important parameter in water management and public policy. Despite the benefits of higher prices of water, it is difficult for the policy makers to implement. Therefore, different supply and demand management measures have been proposed in this study and their benefit cost ratio and social impacts have been worked out in the following table.

Ranking of Benefit Cost Ratio and Social Cost of Various Measures

Description	Benefit/Cost Ratio	Ranking	
		Benefit /Cost	Social Cost
Modernization of Irrigation System (MIS)	1.62	2	Low
Cropping Pattern change			
-Vegetables	0.96	4	Medium
-Cereals	1.40	3	
Abstraction Control	35	1	High

Planning and Implementing Steps

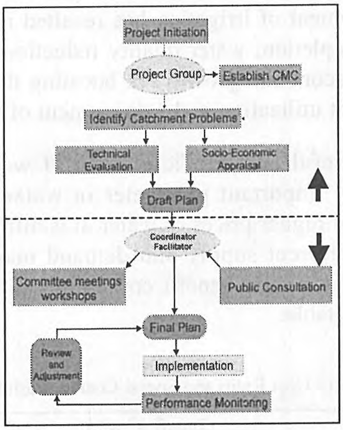
The plan is implemented at the local level with should be support of the Government. Performance monitoring and review are to be carried out over the planning period to assess and appraise the effectiveness of the plan. The plan should be reviewed periodically by the project group to update the plan as necessary based on the monitoring results. Planning And Implementing Steps

Putting The ICMP Into Action

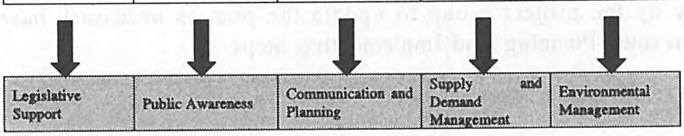
Major concerns of the catchment were grouped under five main headings below. Each of the actions in the following section attempts to address one or more of these concerns. Only the major actions are given in this summary, full action tables are available in the Main Report of the study.

Conclusion

Increasing demand for water to meet the increasing water requirement from different sectors has been a feature of the study area it appear likely to persist in present and future. Competition for water between agricultural and other sectors will continue if not effective water demand management is. Reducing water consumption rate is the only solution to solve the water deficit problem and to meet the future water requirement of an expanding of population through water demand management.



Legislation	Awareness	Communication	Water Management	Environmental
Establishing CMC Water quota allocation Comprehensive groundwater management policy development	Effect of over abstraction Water conservation Knowledge of current issues Protecting land & water	Inter-Ministerial cooperation Cooperation between the Government and the Community Support for farm planning Community education	Supply management in the catchment Irrigation scheduling Water conservation measures Water quota allocation Water tariff and water markets	Gravel quarrying Vegetation belts Solid and liquid waste disposals



Co-Sponsors :

- Arabian Gulf University
- Arab Organization for Agricultural Development (AOAD)
- International Atomic Energy Agency (IAEA)
- United Nations University (UNU)
- International Desalination Association (IDA)
- European Desalination Society (EDS)
- International Center for Bio-saline Agriculture (ICBA)
- Food and Agriculture Organization of the United Nations (FAO/RNA)
- The World Bank (WB)