WSTA 5th Gulf

WATER CONFERENCE

24-28 March 2001 - Doha, Qatar

CONFERENCE PROCEEDINGS Vol. II

Ministry of Municipal Affairs and Agriculture
Qatar General Electricity and Water Corporation
Qatar University

Secretariat General - GCC

Water Science and Technology Association (WSTA)

Under the patronage of

His Highness Shaikh Jassim Bin Hamad Al Thani

The Crown Prince of the State of Qatar

WSTA 5th
Gulf Water Conference

"Water Security in the Gulf"

29 Dhu'I-hijja, 1421 H – 3 Muharram 1422 H

24 – 28 March 2001

Doha – State of Qatar

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Organized by

Water Science and Technology Association (WSTA)
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Ministry of Municipal Affairs and Agriculture, Qatar
Qatar General Electricity and Water Corporation, Qatar
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Dr. Ali M. El-Nashar

ABSTRACT

The performance of several cogeneration options are considered in association with the MSF process for seawater desalination. Both full load and part load characteristics of each major commercial cogeneration process is reviewed both from a technical and economic viewpoint. A methodology for selecting the optimum cogeneration option to satisfy a given demand of power and water is described and an example is given for a cogeneration plant having a power rating of 300 MW and 50 MIGD of potable water. The life cycle cost analysis is used to select the optimum cogeneration system and the exergy analysis method is used for allocating the costs between electricity and water.

Keywords: Cogeneration, desalination, distillation, dual-purpose plants for power and water, economic analysis

INTRODUCTION

Most of the potable water and electricity in the Arabian Gulf countries are produced by cogeneration plants associated with multi-stage flash (MSF) desalination units operating on seawater. Although other distillation process such as thermal vapor compression and MED are started to find their way in the market, the MSF process is still considered as the workhorse of desalination industry. In spite of its limitations, this process has proven its reliability and flexibility over almost 50 years of plant design and operation. For large desalination capacity, say beyond 30 MIGD, the MSF process can be considered as the only candidate that can be considered commercially. However, on the cogeneration plant side, the situation is different in that several alternatives are commercially available to provide the required electrical power and steam for desalination. Among these alternatives are:

- Gas turbines associated with heat recovery steam generators (GT-HRSG),
- Back-pressure steam turbines (BPST) with the discharge steam directed to desalination,
- Controlled extraction-condensing steam turbines (ECST) where the steam for desalination is bled from a location on the steam turbine which matches the steam pressure required by desalination,
- Combined gas/steam turbine cycles where a heat recovery steam generator (HRSG) is used to produce steam at medium or high pressure that is supplied to a back-pressure steam turbine discharging into the MSF desalination plant, this system is referred to as CC(BPST),
- Combined gas/steam turbine cycles that are similar to the previous cycle except that a controlled extraction-condensing steam turbine is used, CC(ECST).

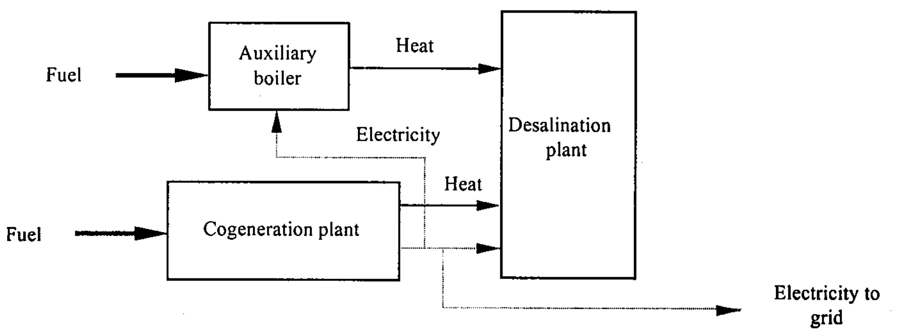
Each of the above cogeneration plants has its own characteristics in terms of their part-load performance curves, fuel requirement and capital and O&M cost needs. The matching of a cogeneration plant with a given rated power capacity to an MSF plant designed to produce a given amount of desalted water require knowledge not only of the technical performance and economic data of the different technologies, but also data on the annual variation of electrical and water demand on the site on which the plant is to be constructed.

The paper by Kovacik et al.(1987) and the two books by McMahan Jr.(1987) and Payne give useful information on the fundamentals of cogeneration technology and its applications.

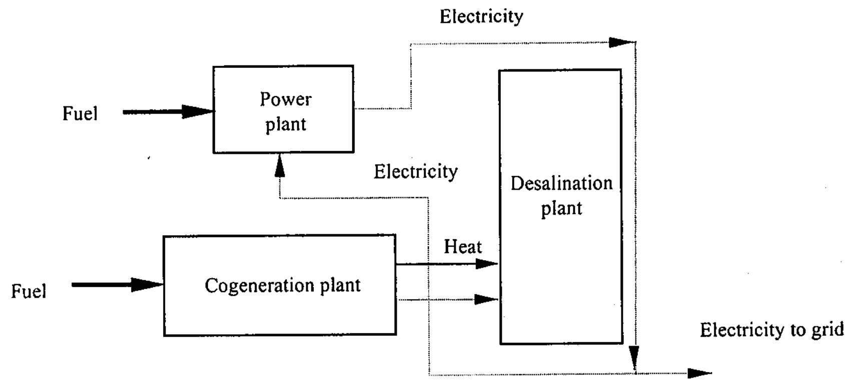
This paper reviews the technical and economic characteristics of the cogeneration plants currently commercially available as well as the cost parameters for both cogeneration and desalination plants. A methodology for selecting the optimal arrangement of cogeneration plant for a particular power and desalination capacity is described and an example is given to demonstrate the use of the method.

THE SYSTEM TO BE OPTIMIZED

Each cogeneration system normally operates in a rather narrow range of heat-to-power-ratio,λ_{cog}, which depends on the plant design and operating parameters [Awerbuch (1997)]. The demand heat-to-power-ratio, λ_d, is a time-dependent quantity which is influenced by the daily and seasonal demand variation for power and desalinated water and usually deviates from the corresponding value for the cogeneration plant. When the plant heat-to-power-ratio, λ_{cog}, is lower than the demand value, i.e. $\lambda_{cog} < \lambda_d$, with the demand power fully met but there is a deficiency in the heat supplied to desalination, an auxiliary boiler is then added to provide the shortfall in the steam demand by the desalination plant (see Figure 1(a)). On the other hand, if the plant heat-to-power-ratio, λ_{cog}, is larger than the demand value, i.e. $\lambda_{cog} > \lambda_d$, with the demand for steam supply to desalination fully met but the power generated is lower than the demand value, a single-purpose power plant is needed to supply the shortfall in power demand (see Figure 1(b)).



(a) Conceptual scheme and energy flows of a cogeneration plant with an auxiliary boiler



(b) Conceptual scheme and energy flows of a cogeneration plant with a single-purpose power plant

Figure 1 Unmatched cogeneration plants.

THE COGENERATION SCHEMES CONSIDERED

The six cogeneration schemes considered in this study are shown below:

- **(a)** Gas turbine + heat recovery steam generator (HRSG) + auxiliary boiler + MSF desalination plant
- **(b)** Gas turbine + supplementary fired HRSG + MSF desalination plant
- **(c)** Gas turbine + supplementary fired HRSG + back pressure steam turbine + MSF desalination plant
- **(d)** Gas turbine + supplementary fired HRSG + extraction - condensing steam turbine + MSF desalination plant
- **(e)** Back-pressure steam plant + MSF desalination plant
- **(f)** Extraction-condensing steam plant + MSF desalination plant

The gas turbines used in schemes (a) and (b) above produce power in a virtually identical manner to simple cycle gas turbines. However, the waste heat exhausting from the gas turbine is captured in an unfired HRSG (scheme (a)) and in a fired HRSG (scheme (b)). In an unfired HRSG, the amount of steam produced is a function of the gas turbine power output. As the load on the gas turbine is reduced, the corresponding HRSG steam production is reduced. Since desalinated water production must remain essentially constant, auxiliary boiler steam must be produced to maintain a constant steam flow to the desalination unit. In a fired HRSG (scheme (b)), as load is reduced on the gas turbine, gas-fired burners in the HRSG inlet duct are fired to keep the steam production for desalination constant.

Schemes (c) and (d) make use of gas turbine/steam turbine combined cycle systems with either a back-pressure steam turbine (scheme (c)) or an extraction-condensing steam turbine (scheme (d)). These units produce additional power from the same quantity of fuel but produce less heat compared to gas turbine schemes (a) and (b), thus, their heat-to-power-ratio is less for (a) and (b).

In scheme (e), natural gas can be burned in a fired boiler to produce high-pressure steam that can be fed to a back-pressure steam turbine to produce power. The exhaust from the steam turbine will feed the desalination plant. Feedwater heating utilizing extractions from the steam turbine are used to improve overall plant efficiency. Part-load performance can be achieved by bypassing some of the steam around the steam turbine. To reduce steam turbine load, steam turbine throttle valves must start to close to reduce the amount of steam flowing through the turbine. This plant would require a dump condenser to allow maximum power production when a desalination unit is out of service. This scheme has the highest heat-to-power-ratio.

In scheme (f) an extraction-condensing steam turbine is used which have a controlled-extraction port on the steam turbine to maintain proper steam flow and pressure for the desalination unit. Desalinated water production is reduced by about 40 percent compared to the backpressure steam turbine case, since some steam flow is required to flow through the low-pressure condensing section of the steam turbine, and since additional feedwater heaters are used to improve cycle efficiency.

PERFORMANCE CHARACTERISTICS OF COGENERATION PLANTS

PERFORMANCE PARAMETERS

Several performance parameters has been suggested by a number of investigators, see, for example, Porter et al. (1982), Antonini et al. (1991) and Baughn et al. (1985). Only the most popular ones are used in this study.

A first evaluation of the performance obtainable from different cogeneration plants can be made by considering the most commonly used parameters such as exergetic efficiency η_{ex} defined as

$$\eta_{ex} = \frac{P_{cog} + E_Q}{E_f} \tag{1}$$

where P_{cog} is the net power output of the cogeneration plant, E_Q is the exergy of the process heat extracted from the cogeneration plant and E_f is the exergy of the fuel input to the plant. The exergy of the process heat is calculated by multiplying the mass flow rate of process steam by its specific exergy:

$$E_Q = m_S\{(h - h_O) - T_O(s - s_O)\} \qquad (2)$$

where m_s is the mass flow rate of process steam, h is its specific enthalpy and s is its specific entropy. h_o and s_o are the coresponding values at ambient conditions (dead state). Note that for single-purpose power plants , $\eta_{ex} = \eta_{th}$ since $E_Q = 0$.

Another performance criterion developed for cogeneration plants involves a comparison between the fuel required to meet the given loads of electricity and heat in the cogeneration plant with that required in separate conventional plant designed to meet the loads, say in a conventional boiler to meet the heat load for desalination and a conventional power station to meet the electrical load. For a cogeneration plant producing net electrical power, P, and an amount of process heat, Q_p, and consuming an amount of fuel energy $(Q_f)_{cog}$, the fuel energy saved, ΔQ_f, is

$$\Delta Q_f = \frac{Q_p}{\eta_b} + \frac{P}{\eta_c} - (Q_f)_{cog} \qquad (3)$$

The fuel energy savings ratio (FESR) is defined as the ratio of the saving (ΔQ_f) to the fuel energy required in the conventional plants.

$$\begin{aligned} FESR &= \frac{\Delta Q_f}{\left(\frac{Q_p}{\eta_b} + \frac{P}{\eta_c}\right)} \\ &= 1 - \frac{\eta_c / \eta_{cog}}{[1 + (\lambda_{cog}\eta_c / \eta_b)]} \end{aligned} \qquad (4)$$

where $\lambda_{cog} = (Q_p/P)$ is the heat to power ratio in MW_{th}/MW_{el}, η_c is the thermal efficiency of the conventional power plant, $\eta_{cog} = [P/(Q_f)_{cog}]$ is the efficiency of the cogeneration plant, η_b is the efficiency of the conventional boiler. In this study, a combined cycle power plant having a thermal efficiency of 0.45, (i.e. $\eta_c = 0.45$), is assumed as the conventional power plant and an industrial boiler having an efficiency of 0.9 (i.e. $\eta_b = 0.9$) is assumed.

Another criterion of interest in cogeneration plants connected to desalination units is the power-to-water-ratio defined as the net power output in megawatt (MW) divided by the capacity of the desalination plant in million of imperial gallons per day (MIGD).

Typical values of these parameters for different cogeneration plants operating at design conditions are shown in

Table 1. It can be seen that the combined cycle system with the backpressure steam turbine (scheme (c)) possess the highest exergetic efficiency (0.48) as well the highest fuel energy savings ratio (0.19). The power-to-water-ratio (PWR) can be seen to vary between 5.1 (for the BPST) to 18.9 (for the CC(BPST)). These values are estimated based on an MSF performance ratio, PR = 8.0 which represent the average values of current MSF designs. Since the design PR values can range between 6 and 10, the design PWR for a cogeneration plant can be different from the values given in

Table 1. It can be shown that the PWR is inversely proportional to the PR value.

Type of cogeneration plant	Exergetic efficiency (η_{ex})	Fuel energy saving ratio (FESR)	Power-to-water-ratio (PWR) (MW/MIGD)
(a) GT + Unfired HRSG+ AB	0.44	0.15	8.7
(b) GT + Fired HRSG	0.44	0.15	8.7
(c) CC (BPST)	0.48	0.19	15.7
(d) CC(ECST)	0.47	0.14	18.9
(e) BPST	0.41	0.17	5.1
(f) ECST	0.38	0.04	9.2

Table 1 Typical values of cogeneration performance parameters at design conditions
(performance ratio of MSF unit = 8)

The part-load exergetic efficiency, η_{ex}, of the six cogeneration schemes is shown in Figure 2. Part-load performance parameters for the different cogeneration schemes were obtained from Bechtel(1996). With the exception of the extraction-condensing steam turbine (scheme (f)), all the other systems experience an increase in the value of η_{ex} with the increase in load thus achieving the maximum efficiency at full load. The combined cycle systems (schemes (c) and (d)), are shown to have the highest exergetic efficiency values compared with the other systems. Also to be noted is that the exergetic efficiency of the ECST system (scheme (f)) is shown to be the lowest particularly at high load ratios.

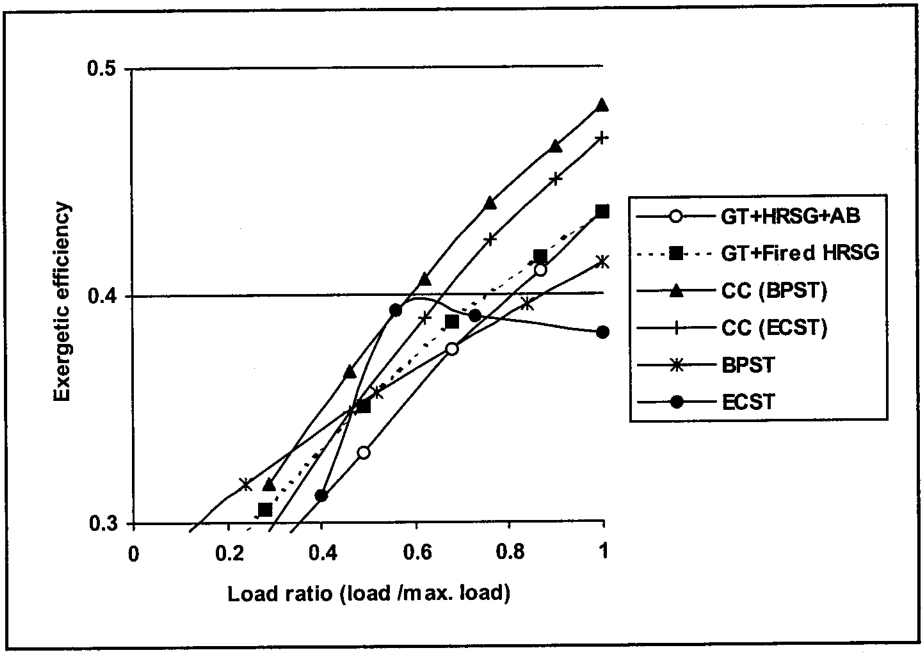


Figure 2 Exergetic efficiency versus load ratio for six cogeneration plants supplying constant steam flow to a distillation plant.

The thermal efficiency (defined as the net power output divided by fuel energy input) of the six cogeneration systems at part load are shown in Figure 3. The combined cycle schemes are also shown to offer the highest values of thermal efficiency while the lowest efficiencies are offered by the backpressure cogeneration system (scheme (e)).

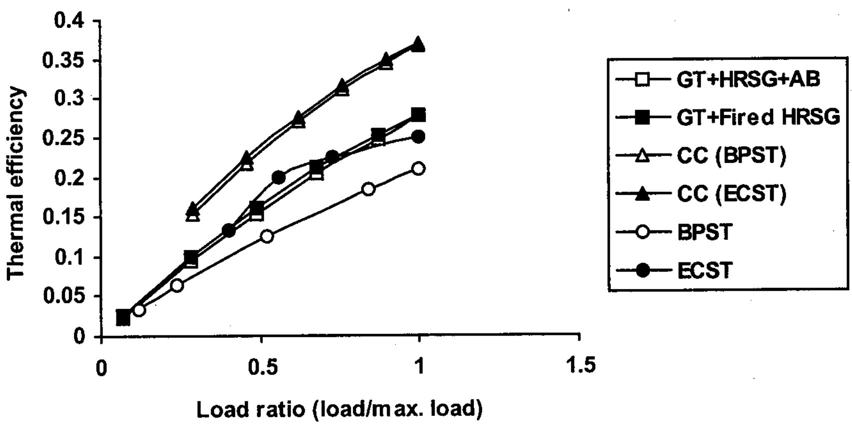


Figure 3 Thermal efficiency of cogeneration plants at part load.

The Fuel Energy Savings Ratio (FESR) at part load for the six systems is shown in Figure 4. It can be observed that, with the exception of the ECST scheme, the FESR tends to increase as the load increases reaching the highest value at full load. However, the ECST system behaves differently by showing a peak value for FESR at a load ratio of about 0.55. The combined cycle with the backpressure steam turbine (CC(BPST)) and the backpressure steam turbine (BPST) systems offers the highest potential for energy savings among the six schemes.

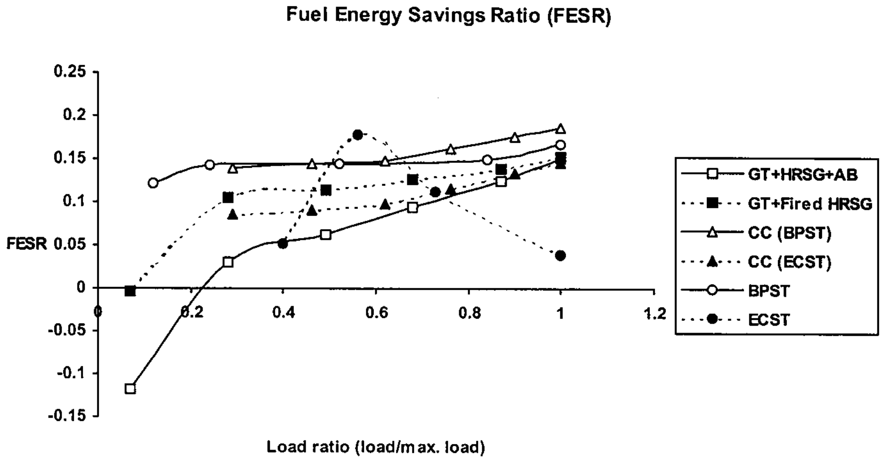


Figure 4 Fuel Energy Savings Ratio (FESR) at part load for cogeneration plants.

It should be noted that the potential energy savings at part load for schemes (a) and (b) becomes very small (or negative) as the load ratio is reduced below 0.2. This means that operating these systems at reduced load levels can result in either a small or no fuel energy savings.

COST ALLOCATION

The *equality method* of the exergy approach [Kotas (1995)] is used for costing electricity and steam from a cogeneration plant. Based on these costs, the cost of water from the desalination plant can then be estimated using the normal economic procedure which take into consideration capital, O&M and energy costs. In the equality method the production by the cogeneration plant of the two products (electricity and steam) is considered to have the same priority, so the capital, O&M and energy costs are split between both products according to the exergy content of each. Based on this method the cost of electricity per unit exergy, $c_e\varepsilon$, is equal to the cost of steam per unit exergy, $c_s\varepsilon$,:

$$c_s^\varepsilon = c_e^\varepsilon \tag{5}$$

The life cycle cost of a cogeneration plant which consists of capital amortization, O&M costs and energy costs can be split among the exergy content of the generated electricity and steam as follows:

$$(LCC)_{cog} = P.H.N.c_e^{\varepsilon} + m_s.H.N.\varepsilon_s c_s^{\varepsilon} \qquad (6)$$

where

P	average power production, kW
H	plant availability, hrs/year
ε_s	exergy content per unit of steam, kWh/ton
CRF	capital recovery factor
m_s	average steam production, ton/hr

This equation is used to estimate the unit exergy cost of electricity and steam. The unit cost of electricity (\$/kWh) is identical to the unit exergy cost of electricity (also \$/kWh) since the exergy content of electricity is identical to its energy content; i.e. $c_e\varepsilon = c_e$. The unit cost of steam on the other hand is different from its unit exergy cost since the exergy content of a mass of steam depends on its pressure and temperature. The unit cost of steam (\$/ton) can be related to its unit exergy cost (\$/kWh) by

$$c_s = c_s^{\varepsilon}\varepsilon_s \qquad (7)$$

Knowing the unit costs of electricity and steam, it is now possible to calculate the unit cost of water produced by the desalination plant from:

$$c_w = \frac{(LCC)_{tot} - c_e(\sum_{0}^{12} \overline{P}\Delta h)N.pf}{m_w.H.N} \qquad (8)$$

ECONOMIC CONSIDERATIONS

The prime movers most commonly used in contemporary large cogeneration systems, and likely to be the dominant prime movers for the foreseeable future, are steam turbines, gas turbines and combined cycle plants. Other prime movers such as fuel cells and Stirling engines are in various stages of development but are expected to see little or no general commercial application in the near future. In this paper, we will focus primary attention on the commercially available technologies, which account for essentially all major cogeneration facilities for power and desalination. Current capital and O&M cost information and expected lifetime for different cogeneration and desalination technologies are listed in Tables 10 and 11.

Table 2 Cogeneration technologies capital and operating costs.

Cogeneration Technology	Capital cost \$/kW	Fixed O&M cost \$/kW	Variable O&M cost \$/kWh	Expected Lifetime years
BP steam turbines	750 – 1000	40	0.0035	25 – 35
Ext./Cond. Steam turbines	650 – 900	32	0.0035	25 – 35
Gas turbine/ HRSG	450 – 600	43	0.0023	20
Combined cycle with BPST	550 – 850	43	0.0023	15 – 25
Combined cycle with ECST	500 – 800	43	0.0023	15 – 25

Table 3 capital and operating costs and expected lifetime of MSF plants (year 2000\$).

Capital Cost \$/GPD	O&M Cost $\$/m^3$	Expected Lifetime years
4.0 – 12.0	0.1	25

The determination of the preferred cogeneration system to satisfy a certain power and water demand involves examining numerous alternatives each for a particular configuration; the final system specification is the result of an extensive trade-off economic analysis. The work of Estey et al. (1980), Haaland et al.(1977), Kamal (1997)and Rohrer(1996) dealt with the selection of the most cost effective cogeneration scheme for industrial applications.

The economic value of a proposed cogeneration system typically is determined by predicting a series of future cash flows and then evaluating these cash flows according to an agreed-upon set of criteria or indicators. The *LCC* method is the most frequently utilized and widely accepted investment analysis technique. It is based on a discounted cash flow analysis without regard to eventual financing strategies. The *LCC* method recognizes that a capital investment by a utility is a cash outlay for the purpose of producing a stream of future cash revenues (or expense savings) sufficient to recover, or repay, the investment plus a return.

The economic analysis of a cogeneration facility, as any industrial project, requires a set of assumptions concerning general economic conditions and ground rules, current and future. The primary ground rules that must be established for an economic cash flow analysis are:

- The economic life of the facility
- The first year of operation
- The number of years of construction
- The general inflation rate
- The inflation rate for fuel and O&M expenses

The economic life of cogeneration systems is typically 15 to 25 years and is ultimately limited by the equipment life. This economic life is the period of time over which an investment is evaluated to determine its benefits and returns. The number of years of construction, the first year of operation, the general inflation rate, and other specific rates and escalations are parameters used to define the investment and operating costs of a cogeneration facility.

The life cycle cost analysis method is used to determine the most cost effective cogeneration option by comparing the total present worth of all costs incurred through out the lifetime of the plant. The life-cycle cost of a cogeneration plant producing electricity and steam is the sum of the initial cost plus the total present worth of annual costs. The initial cost includes the cost of the power generating equipment and the desalination equipment, engineering, installation and project management. Thus the initial cost can be expressed as

$$(IC)_{cog} = (1 + d + e + f)T_{cog} \qquad (9)$$

where T_{cog} is the total hardware cost (power generation plus desalination equipment), d , e, and f are cost ratios for engineering, installation and project management, respectively.

The present worth of annual costs include annual fuel costs and O&M costs. The present worth of these costs can be expressed as[Groumpos et al (1987)]:

$$PW_f = C_{f0}\left(\frac{1+g_f}{k-g_f}\right)\left[1-\left(\frac{1+g_f}{1+k}\right)^N\right]$$
$$PW_{om} = C_{om0}\left(\frac{1+g_{om}}{k-g_{om}}\right)\left[1-\left(\frac{1+g_{om}}{1+k}\right)^N\right] \qquad (10)$$

where

Pw_f = present worth of fuel costs, \$
Pw_{om} = present worth of O&M costs, \$
C_{f0} = fuel cost in the first year, \$
C_{om0} = O&M cost in the first year, \$
g_f = fuel escalation rate
g_{om} = O&M escalation rate
k = money interest rate
N = system life, years

The life-cycle cost of the cogeneration plant, $(LCC)_{cog}$, is obtained as the summation of the initial plant cost and the present worth of all annual recurring costs:

$$(LCC)_{cog} = (IC)_{cog} + (PW_f)_{cog} + (PW_{om})_{cog} \qquad (11)$$

where $(PW_f)_{cog}$ and $(PW_{O\&M})_{cog}$ are given by

$$(PW_f)_{cog} = (C_f)_o \left(\frac{1+g_f}{k-g_f}\right)\left[1 - \left(\frac{1+g_f}{1+k}\right)^N\right]$$

$$(PW_{O\&M})_{cog} = (C_{O\&M})_{cog}\left(\frac{1+g_{O\&M}}{k-g_{O\&M}}\right)\left[1-\left(\frac{1+g_{O\&M}}{1+k}\right)^N\right] \qquad (12)$$

where

$(C_f)_o$ fuel cost during the first year of operation, \$
$(C_{O\&M})_{cogo}$ O&M cost during the first year of operation, \$
g_f fuel escalation rate
$g_{O\&M}$ escalation rate of O&M expenses
k interest rate
N equipment lifetime, years

The life cycle cost of a desalination plant, consists only of capital amortization and O&M costs since the cost of steam and electricity used by the desalination plant is born by the system as a whole and does not involve the consumption of additional external resources:

$$(LCC)_{des} = (IC)_{des} + (PW_{O\&M})_{des} \qquad (13)$$

where

$(IC)_{des}$ initial capital of desalination plant, \$
$(PW_{O\&M})_{des}$ present worth of O&M costs of desalination plant, \$

The total life cycle cost of the combined cogeneration and desalination plants is the summation of $(LCC)_{cog}$ and $(LCC)_{des}$

$$(LCC)_{tot} = (LCC)_{cog} + (LCC)_{des} \tag{14}$$

SELECTION METHOD

The selection of the optimal cogeneration facility from among a number of options is usually carried out using a computer model. Estey et al. (1980). It is assumed that the nominal capacity of the required cogeneration plant is known (both power and water production rates) based on the estimated demand for power and water at the site. The performance and cost parameters of a number of possible plant configurations are among the input parameters that are required by the program. On any one computer run, the program computes the life cycle cost, *LCC*, of all discounted cash flows for a series of user-supplied unit sizes and the plant with the lowest *LCC* value is selected .

A simplified block diagram of the computational procedure is shown in Figure 19. The model used is designed to select the optimum cogeneration system size for a specific site where the electrical and water demands are specified throughout the year. The selected system is specified by the following three design parameters:

- The cogeneration system size.
- The number of cogeneration units.
- The size of the auxiliary boiler if needed.
- The size of the single-purpose power plant if required.

The selection is made from a user's desired spectrum of different sizes and different number of units that are to be investigated. In addition to the sizes and number of units, equipment data such as cost and performance of each piece of equipment; economic data such as financing method, fuel cost, etc; plant load data such as electrical and water load variation throughout the year; and weather data such as ambient temperature and pressure are also required as input to the program.

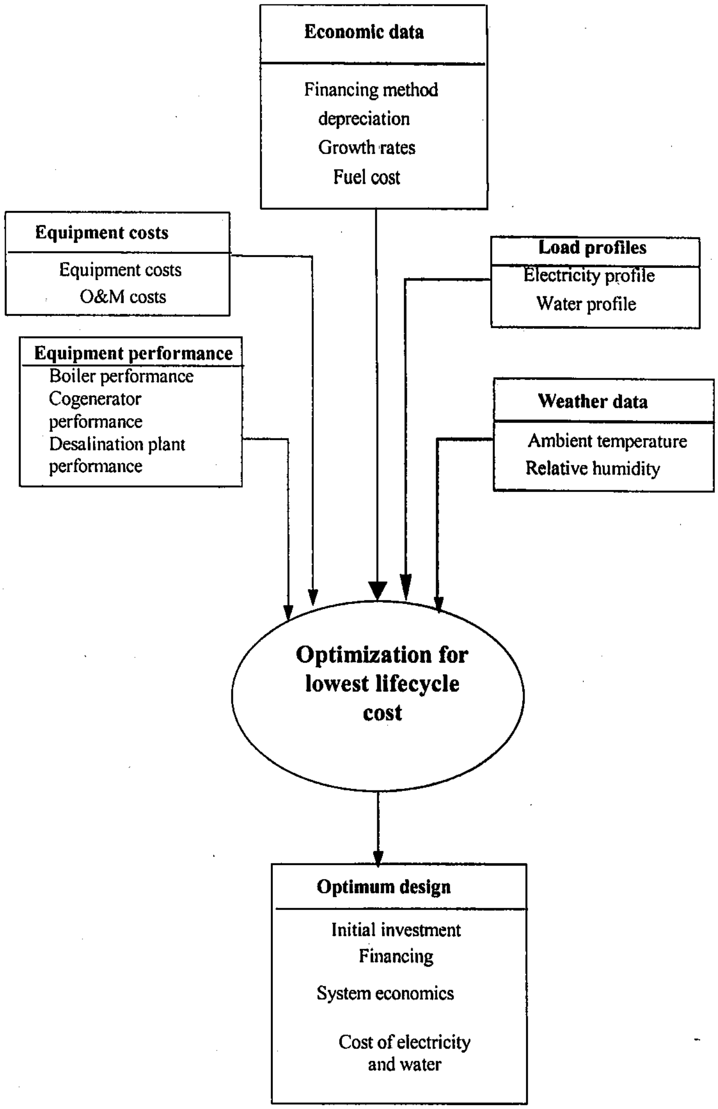


Figure 5 Block diagram of the optimization model for cogeneration plant.

CASE STUDY

To illustrate the procedure for selecting the optimum cogeneration system from among several options, let us consider an example where it is required to install a cogeneration plant having a rated power output of 300 MW and a rated desalted water production of 50 MIGD. The plant specification and economic parameters assumed are given in Table 4.

Table 4 Plant specification and economic parameters.

Parameter	Value	Unit
Rated power output	300	MW
Rated water production	50	MIGD
Performance ratio of MSF plant	7,8,9	
Fuel cost	1	$/GJ
Plant factor	0.9	
Escalation rate for fuel	0.03	
Escalation rate for O&M expenses	0.03	
Interest rate	0.08	
Plant lifetime	25	years

The options considered for plant configurations are as follows:

- Backpressure steam turbine connected to MSF (BP-ST)
- Extraction-condensing steam turbine connected to MSF (EC-ST)
- Gas turbine with heat recovery steam generator and auxiliary boiler connected to MSF (GT-HRSG)
- Combined cycle gas/ backpressure steam turbine connected to MSF (CC-BP)
- Combined cycle gas/extraction-condensing steam turbine connected to MSF (CC-EC)

The assumed specific capital cost and O&M cost for each of the above cogeneration options as well as the MSF plant are shown in Table 5.

Table 5 Specific capital and O&M costs of the cogeneration plants.

Cost parameter	BPST	ECST	GT-HRSG	CC(BPST)	CC(ECST)	MSF PR=7-9
Specific capital cost,	1000 $/kW	950 $/kW	700 $/kW	800 $/kW	800 $/kW	8-10 $/gpd
Fixed O&M cost	40 $/kWy	32 $/kWy	43 $/kWy	43 $/kWy	43 $/kWy	0
Variable O&M cost	0.0035 $/kWh	0.0035 $/kWh	0.0023 $/kWh	0.0023 $/kWh	0.0023 $/kWh	0.1 $/m^3

The electrical load on the cogeneration plant is assumed to vary throughout the year according to Figure 6. This load variation is typical for other cogeneration plants operating in Abu Dhabi, UAE, and is also similar to the load patterns in other Gulf areas. The MSF desalination plant is assumed to operate at full load throughout the year. The shortfall in steam production by the cogeneration plant is assumed to be supplied by an auxiliary boiler whose capacity depends on the configuration of the plant as well as the load variation.

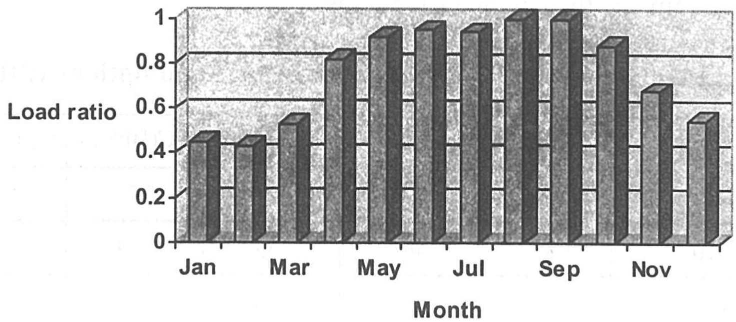


Figure 6 Electrical monthly load ratios assumed for the example problem.

The annual fuel consumption for each option was estimated as the summation of the monthly fuel consumption by both the cogeneration plant and the auxiliary boiler. The monthly fuel consumption by the cogeneration plant was estimated by multiplying the monthly-average plant heat rate and monthly average load. The heat rate obviously depends on the plant load that varies throughout the year. The monthly-average fuel consumption by the auxiliary boiler is estimated from knowledge of the shortfall in the steam required by the MSF plant that cannot be supplied by the cogeneration plant.

The capital cost, present value of fuel and O&M expenditures as well as the life cycle cost for each option are shown in Table 6 which indicates that the most economic alternative is the gas turbine- heat recovery steam generator with a life cycle cost of 1493 million $ to be followed with a small margin by the combined cycle- back pressure option. The costs shown in this table are estimated assuming that the performance ratio, *PR*=9.0.

The *LCC* for different values of *PR* is shown in Figure 7.which indicates that the *LCC* is quite sensitive to the *PR* value for each plant configuration. The BP-ST configuration shows an increasing trend for *LCC* with increasing the *PR*. However, the other configurations displays the opposite trend, i.e, the *LCC* decreases with increasing *PR*. As can be seen, the lowest *LCC* value is for the GT-HRSG option with *PR* = 9.0. The reason why the BP-

ST exhibits this trend is that when the *PR* of the desalination plant is about 7, the annual amount of steam discharged from the BP steam turbine is just enough to supply the required amount of steam required by the desalination plant. Thus the steam turbine will match the MSF plant reasonably well. On the other hand, when the *PR* is larger than 7, the steam turbine will produce more steam than required by the MSF plant and hence some of this steam will have to be condensed in a dump condenser which constitutes a waste of energy. Thus increasing the *PR* beyond 7 for this option brings only an increase in capital cost of the MSF plant and no extra benefit to the overall economy of the cogeneration plant.

Table 6 Life cycle costs of the different cogeneration options (PR = 9).

Cost parameter	BPST	ECST	GT-HRSG	CC(BPST)	CC(ECST)
Capital cost, $\$10^6$	800	803	727	772	775
PV fuel, $\$10^6$	552	465	533	439	459
PV O & M $\$10^6$	194	266	233	292	303
Life cycle cost $\$10^6$	1547	1534	1493	1503	1537

PV = present value

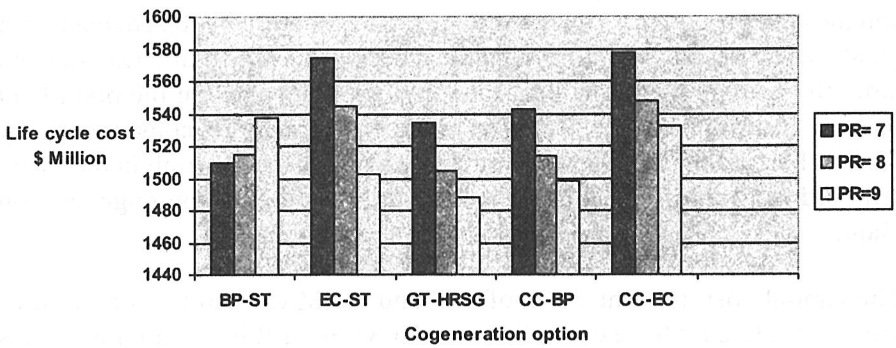


Figure 7 Life cycle cost for different performance ratio.

The situation with the other cogeneration options is the opposite to that of the BPST one in that for these options, the amount of steam produced is smaller that that required by the MSF plant which make it necessary to install an auxiliary boiler to supplement the shortfall in steam requirement. The additional capital, fuel and O&M expenses associated with this boiler contributes to a high *LCC* value for the whole cogeneration plant. The increase in the performance ratio of the MSF plant can relieve this situation by reducing the capacity of the auxiliary boiler required as well its associated fuel and O&M expenses thus help to reduce the *LCC*.

The unit cost of electricity and water is shown in Figure 8 and Figure 9, respectively. The optimum cogeneration option (GT-HRSG at *PR*=9) results in a unit cost of electricity of 2.16 c/kWh and a unit water cost of 1.13 $/m^3.

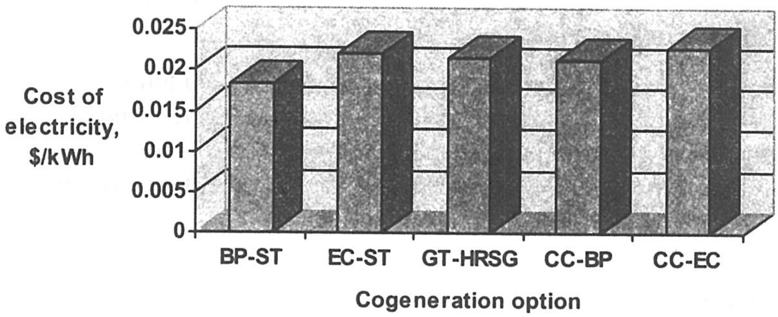


Figure 8 Cost of electricity for each cogeneration option (PR = 9.0).

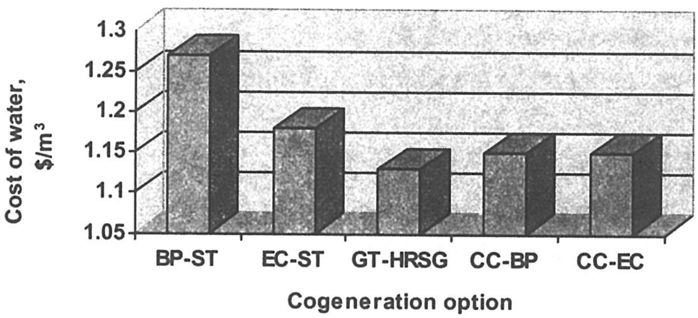


Figure 9 Cost of water for each cogeneration plant (PR =9.0).

CONCLUSIONS

- The wide variety of options available for combining cogeneration plants with desalination plants and the influence of the technical and economic performance parameters of each combination makes the use of system modeling using computer programming inevitable.
- The combined cycle options offer the highest values of exergetic efficiency and potential fuel savings.
- The potential fuel savings for most cogeneration plants depends on the electrical load with the highest savings achieved at full load.

- The optimum cogeneration option depends strongly on the load variation throughout the year. Both monthly electrical and water production loads should be input parameters to the computer model.
- The power to water ratio of the different combinations of cogeneration plants scans the range from 4 to 18 MW per MIGD with the lowest ratio for the BPST option and the highest ratio for the CC(ECST) option.
- The power to water ratio has a strong influence on the optimum selection of a cogeneration plant.
- The selection of the most economical cogeneration plant should be based on a life cycle cost analysis which should take into consideration the escalation rates of fuel and O&M expenses in order to arrive at an estimate of the total expenses for each option for the whole lifetime of its operation.

NOMENCLATURE

C_e	annual cost of electricity, $
c_e	unit cost of electricity, $/kWh
$c_e\varepsilon$	cost of electricity per unit exergy, $/kWh
C_f	annual fuel cost, $
C_{fo}	fuel cost during first year of operation, $
$C_{O\&M}$	O&M cost during first year of operation, $
CRF	capital recovery factor
C_s	annual steam cost, $
c_s	unit cost of steam, $/ton
$c_s\varepsilon$	cost of steam per unit exergy, $/kWh
E	exergy, kW
$FESR$	fuel energy savings ratio
g_f	annual fuel escalation rate
$g_{O\&M}$	annual O&M cost escalation rate
h	specific enthalpy, kJ/kg
H	plant operation time, hours per year
IC	initial capital cost, $
IC	initial capital, $
k	interest rate
LCC	life cycle cost, $
M_d	rated capacity of desalination plant, m^3/day
m_w	rated capacity of desalination plant, m^3/hr
N	plant lifetime, years

$O\&M$	annual O&M expenses, $
pf	plant availability, number of running hours per year divided by 8760
P	net power output, kW
$\overline{P}$	monthly average load, kW
PR	performance ratio
PW	present worth, $
PWR	power to water ratio, MW/MIGD
Q	thermal energy, MW_{th}
T	total hardware cost, $

Greek Letters

Δh	number of hours in a month
λ_{cog}	process heat to power ratio, MW_{th}/MW_{el}
λ_d	demand heat to power ratio, MW_{th}/MW
η_b	boiler efficiency
η_c	thermal efficiency of a conventional power plant

Subscripts

cog	cogeneration
des	desalination
e	electricity
f	fuel
fo	fuel for first year
net	net power
O&Mo	O&M for first year
OM	operation and maintenance
s	steam
w	water

Abbreviations

BPST	back pressure steam turbine cogeneration
CC(BPST)	combined cycle with back pressure steam turbine cogeneration
CC(ECST)	combined cycle with controlled extraction-condensing steam turbine cogeneration

ECST	controlled extraction-condensing steam turbine cogeneration
GT-HRSG	gas turbine and heat recovery steam generator cogeneration
IC	initial cost
LCC	life cycle cost
MSF	multistage flash
O&M	operation and maintenance

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Overview of Design Features and Performance Characteristics of Major Saline Water Conversion Corporation (SWCC) MSF Plants

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OVERVIEW OF DESIGN FEATURES AND PERFORMANCE CHARACTERISTICS OF MAJOR SALINE WATER CONVERSION CORPORATION (SWCC) MSF PLANTS

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ABSTRACT

The Saline Water Conversion Corporation (SWCC) is internationally recognized as the main producer of desalinated water. SWCC is currently operating 13 large MSF desalination plants with a total installed base load capacity of 658.27 MIGD. MSF plants are coupled with either back pressure or extraction condensing turbines for the simultaneous production of power and water. SWCC MSF distillers are characterized by a wide range of design features and performance characteristics. Distiller production capacity ranges from as low as 2.425 MIGD to as high as 10 MIGD in the SWCC's latest dual purpose plants. Performance ratios vary between 5.6 and 10.6 kg/2326 kJ.

SWCC's vast design and operating experiences are effectively being utilized to provide guidance for future planning and selection of improved MSF design. Overview of SWCC MSF plants' salient design features are presented in this paper. Most important operating parameters e.g. temperatures and mass flow rates, and fouling factors of MSF plants are reported. The paper also reviews other design features such as surface area, tube dimensions, materials of construction and energy requirements.

Performance characteristics of MSF plants, which have been in operation for more than twenty years are highlighted. SWCC successful attempts to improve the performance of its MSF distillers and reduce operating cost especially with regard to scale control are reviewed.

Key words : SWCC, Desalination, MSF, Design Features, Performance Characteristics

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INTRODUCTION

The Saline Water Conversion Corporation (SWCC) of Saudi Arabia is currently producing more than 16 percent of the total world production of desalinated water [1]. The majority of SWCC desalination plants are employing the multistage flash process producing more than 93 percent of its total water production. All large MSF plants are coupled with power generation facilities for simultaneous production of power and water.

The first two seawater condensers built in Saudi Arabia were installed in Jeddah during the year of 1928. They became known as "Al-Kandasa". In 1970 SWCC installed its first large scale MSF plant in Jeddah. Historical developments of SWCC desalination plants were recently reported [2]. SWCC MSF plants are located along the Arabian Gulf and the Red Sea coasts. In the east, along the Arabian Gulf, MSF plants are located in Al-Khafji, Al-Jubail and Al-Khobar. Along the Red Sea the large MSF plants are located in Jeddah, Yanbu, Al-Shugayg and Shoibah.

SWCC MSF distillers are characterized by a wide range of design and operating characteristics [3-6] .The objective of this paper is to give a comprehensive overview of the salient design and operating features of SWCC MSF desalination plants and to provide an insight into the basic relationship between the different designs parameters which can then be used together with the vast amount of operational experience as a guide for future planning of an improved MSF design and operation.

DESIGN FEATURES

All SWCC large MSF plants operate within the context of dual purpose facilities for the simultaneous production of power and water. Such co-generation arrangement uses either back-pressure or extraction condensing turbine. Some MSF/Power plants are also combined with RO plants (Jeddah, Yanbu & Al-Jubail) taking advantages of common seawater intake and outfall, common pretreatment facilities and the blending of permeate in appropriate proportions from MSF and RO plants if the need arises, as well as reducing the power to water ratio.

Salient features of SWCC MSF plants are shown in Table-1. All the MSF distillers are operating with brine recirculation modes and the majority is with cross flow configuration except Jeddah II & IV, which have long tube configurations. The number of stages varies from 16 in Alkhobar and Jeddah III up to 34 in the unique design configuration of Jeddah II. Distillate production ranges from as low as 2.5 MIGD in Al-Khafji and Jeddah II up to 10 MIGD in the mostly recently built plants. The design performance

Table -1: Salient Design features of SWCC MSF Plants

		1		2			3		4		5		6	
S. No.	SWCC-MSF plants	Production, MIGD (Base load Capacity)		No. of Stages	Top Brine Temp.	Design PR	Concentration Ratio		Design Fouling Factor $m^2K/W \times 10^{-3}$		M_{sw}/M_d	M_R/M_d	Brine Velocity, m/sec	
		No. of Distillers	MIGD/ Distiller	Total	^0C (max)	Kg./1000 kJ	(BR)	(BB)	BH	H. Rec.			BH	H Rec.
1	Al-Jubail Phase I	6	5.05	22	90.6	3.45	1.44	1.56	0.264	0.1761	11.87	12.64	2.0	1.87
2	Phase II C2	10	5.21	22	112.8	4.09	1.35	1.39	0.176	0.176	9.23	11.32	1.98	1.98
	C3	10	5.21	22	112.8	4.09	1.35	1.39	0.176	0.176	9.23	11.32	1.98	1.98
	C4	10	5.21	19	112.8	4.09	1.39	-	0.176	0.176	7.10	11.52	1.98	1.98
	C5	10	5.21	22	112.8	4.09	1.4	1.51	0.176	0.176	8.79	10.92	1.58	1.58
3	Jeddah Phase II	4	2.43	34	115	3.98	1.28	1.49	0.086	0.0861	3.27	3.63	2.1	1.54
4	Phase III	4	4.86	16	108	3.05	1.3	1.36	0.325	0.178	8.21	8.58	2.01	1.71
5	Phase IV	10	4.88	24	110	3.02	1.32	1.46	0.325	0.176	8.79	8.40	1.79	1.75
6	Al-Khobar Phase II	10	4.91	16	115	2.39	1.32	1.19	0.160	0.12	12.51	11.59	2.0	2.0
7	Phase III	8	7.70	16	105	2.79	1.11	1.23	0.264	0.106	13.35	10.16	2.0	2.0
8	Yanbu Phase I	5	4.76	24	121	4.57	1.33	1.52	0.1204	0.1504	6.87	7.48	1.91	1.81
9	Phase II	4	7.92	21	108	3.7	1.39	1.55	0.3	0.17	8.36	9.22	1.85	1.86
10	Shugayg Phase I	4	5.34	19	90	3.55	1.35-1.4	1.45-1.5	0.300	0.17	12.18	13.64	1.83	1.8
11	Shouaiba Phase I	10	4.91	19	90		1.44	1.57	0.3	0.17	9.90	12.34	1.77	1.82
12	Phase II	10	10	21	110	3.87	1.56	1.75	0.21	0.15	7.93	9.11	2.0	2.0
13	Khafji Phase II	2	2.52	22	112.8	3.52		1.7	0.279	0.279	4.61	5.55	1.56	1.55

Table -2: Tube dimensions and flash chambers specification.

S. No.	SWCC-MSF plants	Tube Dimension									Design Flash Chambers Specifications		
		BH			H Rec.			H Rej.					
		number	length (m)	ID (mm)	number	length (m)	OD (mm)	number	length (m)	OD (mm)	Width (m)	Stage Length (m)	Height (m)
1	Al-Jubail Phase I	2222	13.72	30.73	44517	14.73	32.0	6294	14.732	30.0	14.0	2.8 – 3.2	3.9
2	Phase II C2	1463	16.87	39.5108	29298	20.0	42.0	3501	20.0	36.0	19.4	65.2 *	3.8
	C3	1463	16.87	39.5108	29298	20.0	42.0	3501	20.0	36.0	19.4	65.2 *	3.8
	C4	1852	14.312	35.61	33014	20.0	40	6160	21.0	25.0	19.3	3.35 – 3.59	4.4
	C5	1665	14.5	39	31635	19.92	39	5562	19.934	29.0	19.6	2.51 – 3.03	4.2
3	Jeddah Phase II	2059	6.802	19	11036	24.54	20.8	1983	18.66	19	3 - 4	18.1 – 23.8 per module**	3.15 - 3.3
4	Phase III	4186	12.4	26.747	33698	12.817	28.575	7644	12.817	17.22	11.95	47 *	4.2
5	Phase IV	5400	14.65	17.2	21600	25.0	19.2	5000	25.0	17.2	8.5 – 9.0	25.1 per module**	4.8
6	Al-Khobar Phase II	4415	10.58	22	56732	12.186	22	3687	12.184	23.86	13.2	3.0 – 3.65	5.2
7	Phase III	2711	23.26	31	2715	15.99	31	2907	15.99	30.4	15.3	3.07-4.28	3.7-3.9
8	Yanbu Phase I	3147	8.125	23	67473	10.570	23	5490	10.650	21	10.2	2.5 – 3.2	3.73
9	Phase II	2030	17.73	35.7	2020	17.76	38.1	1749	17.76	35.6	17.76	2.9 – 4.7	4.2
10	Shugayg Phase I	2760	15.6	29.35	44160	17.8	31.75	6201	17.8	31.75	17.42	3.59 – 3.60	3.9 – 4.05
11	Shuaiba Phase I	3990	12.52	24.8	3957	15.1	25.6	2527	15.0	26.0	14.1	2.85-3.45	3.8
12	Phase II	3730	18.54	28.4	3732	17.82	30.4	3045	17.92	31.75			
13	Khafji Phase II	2278	12.013	22.9	43282	11.2	25.4	4491	11.6	25.4	11	-	3.4

* Total length of evaporator. ** Long tube configuration.

ratio ranges between 2.39 to 4.57 kg/1000 kJ. Table-1 also shows the blowdown and recycle concentration ratio mass flow rate of cooling water and brine recycle with respect to the distillate production and water velocities in evaporator tubes as well as the design fouling factors of the brine heater and heat recovery section.

The tube dimensions of the different sections of the distiller and flash chamber specifications are shown in Table 2.

The flashing brine flow rate per unit width of the flash chamber (shell load) is an important design parameter that affects the cost of shell as well as stage-wise non-equilibrium losses in the stage [7]. Table 3 shows the design shell load of major SWCC MSF plants. For cross flow arrangements, it ranges between 478,600.6 kg/(hr m) for plants with low rated capacity up to 965,207.6 kg/(hr m) in Shoaiba II plant. Using regression analysis the following linear relationship was obtained interrelating the shell load (SL) in kg/(hr m) of SWCC MSF cross flow distillers with rated capacity of each distiller (D) in MIGD: $SL = 457184 + 53432\ D$

Table –3: Shell Load of SWCC MSF plants

S. No.	Plant	Production (MIGD)	(Shell Load)$_{design}$ kg/hr m	(Shell Load)$_{equation}$ kg/hr.m	% Difference
1	**Al-Jubail I**	5.05	734664.8	708250	3.60
2	**Al-Jubail II**	5.21	673643.4	718650	-6.68
3*	**Jeddah II**	2.43	1183333.3	-	-
4	**Jeddah III**	4.86	681338.9	695900	-2.14
5*	**Jeddah IV**	4.88	1000000.0	-	-
6	**Al-Khobar II**	4.91	893939.4	699150	21.79
7	**Al-Khobar III**	7.70	939738.6	880500	6.30
8	**Yanbu I**	4.76	695588.2	689400	0.89
9	**Yanbu II**	7.92	773310.8	894800	-15.71
10	**Shugayg I**	5.34	743283.6	727100	2.18
11	**Shoaiba I**	4.91	825531.9	699150	15.31
12	**Shoaiba II**	10	965207.6	1030000	-6.71
13	**Al-Khafji II**	2.52	478600.6	543800	-13.62

* Jeddah II & Jeddah IV are long tube arrangements.

The difference between the design shell load values and those predicted from the empirical relationship is +21.79 and −15.71 percent.

Table 3 also includes the shell load of Jeddah II and Jeddah III which are long tube arrangements. The shell load of these two plants are not correlated by the empirical equation used for cross flow arrangements.

Another important design variable is the vapor release velocity at brine surface . It has a strong influence on the liquid carryover in the flash chamber as the higher the release rate the higher will be the amount of brine entertainment in the vapor [8]. The vapor release in a distiller varies from stage to stage as shown in figure-1 for selected three distillers representing Al-Jubail II, Al-Khobar II and Shoaiba I plants.

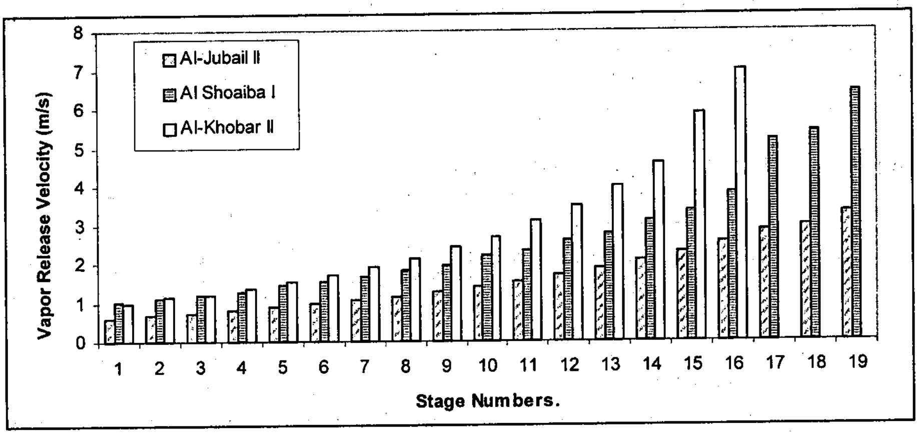

Figure-1: Variation of vapor release velocity with stage number

For the two latter plants the vapor release velocity varies from 1 m/s in the first stage up to 6.5 or 7 m/s in the last stage which is operating with low temperature and having high specific vapor volume. Al-Jubail Phase II distiller is generating low vapor release velocity compared to the two other plants and this is attributed to the low flash plan areas.

MATERIALS OF CONSTRUCTION

The most commonly used materials of construction in SWCC MSF plants are carbon steel, stainless steel, copper-nickel alloy and titanium [9]. The shell of brine heaters of all plants is made of carbon steel and the tubes are either 70/30 or 90/10 Cu-Ni except Al Jubail Phase I which is having titanium tubes.The material of construction of flash chambers is carbon steel with and without cladding. In some plants such as Al Jubail, Al-Khafji II and Jeddah III, the first high temperature stages are cladded with stainless steel. Module 1 of Jeddah II and the first two modules of Jeddah

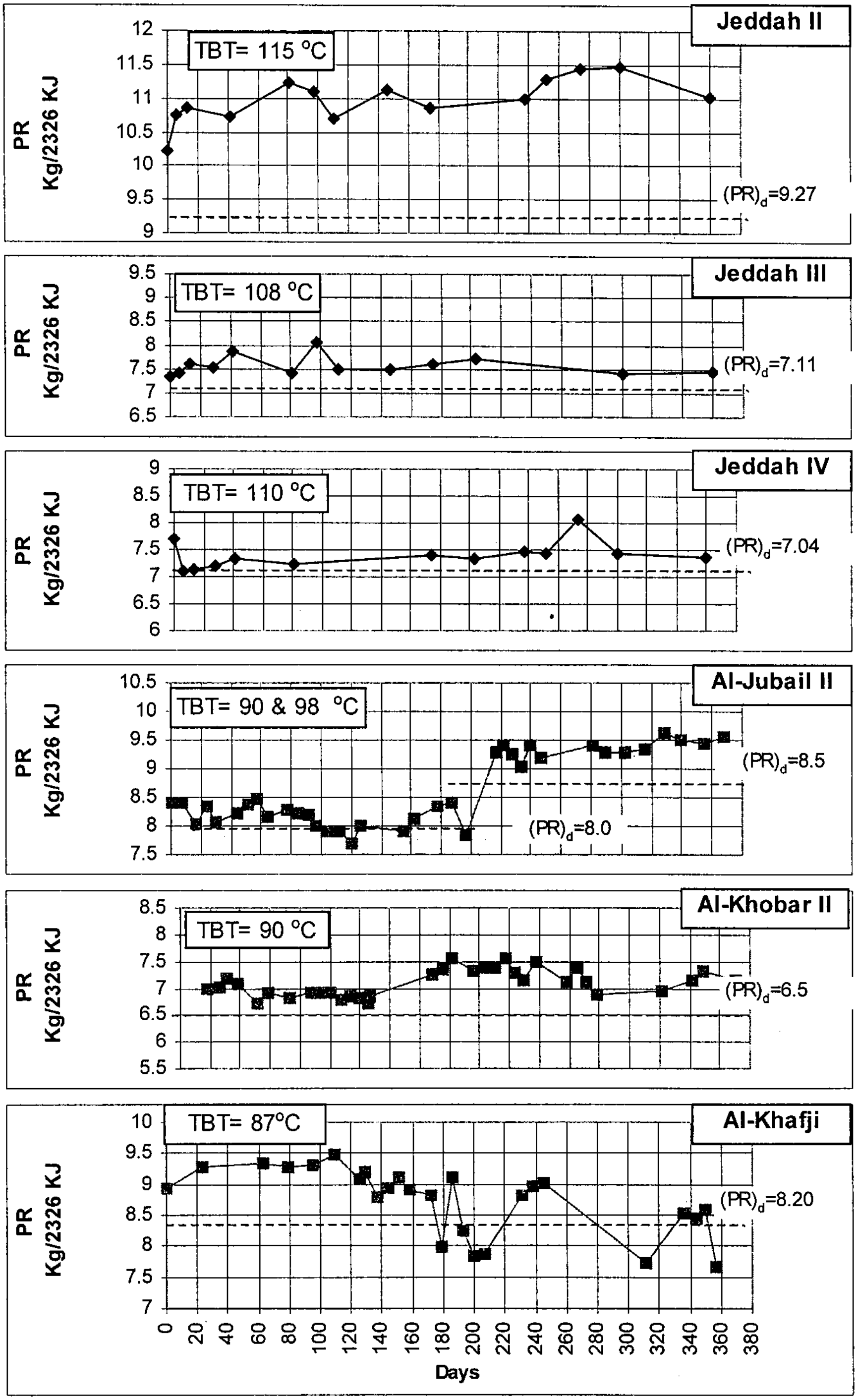

Figure-2: Operational performance of selected SWCC MSF distillers

IV are also cladded with stainless steel. Al Khobar II flash chambers are completely cladded with 90/10 Cu-Ni and Al Shugayg I is also totally claded with stainless steel. The materials of construction of the heat rejection tubes of all plants are made of titanium except Jeddah plants, which are having 90/10 Cu-Ni tubes.

OPERATING FEATURES

The operational performance of selected MSF distillers is shown in Figure-2 based on data collected during a period of 360 days. This figure shows the variation of performance ratio with time in days. Although these plants are in service for more than 18 years, the performance ratio is consistently equal to or in most cases much higher than the design value.

This could be attributed primarily to the generously available surplus heat transfer surface area and equally important to the strict operating and maintenance procedures followed by SWCC. In particular, the combined use of chemical additive and on-load tube cleaning contributed greatly in reducing the fouling factor which resulted in maintaining high overall heat transfer coefficient and reduced steam consumption and hence maintained high performance ratio.

SCALE CONTROL AND ON-LOAD SPONGE BALL CLEANING

Scale formation on heat transfer surfaces, which is one of the basic problems in the desalination of sea water, can be effectively controlled or minimized by the addition of chemical additives and use of on-load ball cleaning. A number of optimization tests have been carried out by SWCC thus leading to successful operation at low antiscalant dose rates [10–18]. Recommended dose rates to SWCC in 1981 were 12.5 and 4.5 for TBT of 110 and 90°C respectively [12] and are currently reduced to only 2.0 and 1.0 ppm for the respective temperatures. This significant reduction in dose rate is attributed to several factors such as improvement in chemical formulation, adoption of on-line sponge ball cleaning and plant operator awareness to reduce chemical dosing while maintaining effective plant performance.

Although the formation of scale is combated and controlled by threshold treatment with the use of antiscalant, its complete prevention is impracticable. Sludge or distorted scale is also formed as a result of threshold treatment, which gets deposited on tube metallic surfaces, and induce resistance to heat transfer. The combined use of chemical additives and on-line tube cleaning has been proved to be the most cost effective means to combat scale formation and to avoid acid cleaning [19–21].

Table-4: On-load sponge ball cleaning:

S. No.	Plant	Chemical Treatment	Ball/ Tube Ratio		Frequency of Ball Cleaning Operation	No. of cycles per operation
			BH	HRC		
1	Al-Jubail Ph. I	Antiscalant	0.450	0.427	3 Oper. / Day	8 Cycles / Oper.
2	Al-Jubail Ph. II					
	C2 & C3	Antiscalant	0.342	0.324	3 Oper. / Day	8 Cycles / Oper.
	C4	Antiscalant	0.270	0.257	3 Oper. / Day	8 Cycles / Oper.
	C5	Antiscalant	0.300	0.302	3 Oper. / Day	8 Cycles / Oper.
3	Jeddah Ph II	Acid	0.296	0.236	One/week	3 cycle/oper
4	Jeddah Ph III	Antiscalant	0.29	0.665	3 Oper. / Day	4 Cycles / Oper.
5	Jeddah Ph IV	Antiscalant	0.251	0.370	2 Oper./ Week	10 Cycles / Oper.
6	Al-Khobar II	Antiscalant	0.453	0.458	3 Oper. / Day	9 Cycles / Oper.
7	Yanbu I	Antiscalant	0.243	0.249	3 Oper. / Day	12 Cycles / Oper.
		Acid	0.243	0.249	One Oper./ Week	12 Cycles / Oper.
8	Yanbu II	Antiscalant	0.345	0.346	3 Oper/ Day	13 Cycles / Oper.
9	Al-Shugayg	Antiscalant	0.22	0.22	3 Oper/ Day	8 Cycles / Oper. (16 for high TBT)
10	Al-Shoaiba I	Antiscalant	0.251	0.253	3 Oper/ Day	3 Cycles / Oper.
11	Al-Khafji	Antiscalant	0.351	0.351	One Oper/ Day	9 Cycles / Oper.

Note: Al Khobar III and Shoauiba II are not included in this table because both plants are currently under commissioning and reliability tests.

All SWCC MSF plants are employing on-load sponge ball cleaning. The chemical treatment, ball to tube ratio and frequency of cleaning in different MSF plants are shown in Table 4.

The ball to tube ratio for plants using chemical additive treatment varies from as low as 0.22 in Al-Shugayg up to about 0.45 in Al Jubail Phase I and Al Khobar plants with average frequency of three ball cleaning operations per day for all plants.

The ball to tube ratio in SWCC MSF plants, thus in most cases, lie within the reported accepted range [17&19]. Larger number of ball to tube ratio may cause problems by several balls passing one tube simultaneously and getting stuck while smaller ratio will not be capable to reach all tubes.

The wide variation of ball to tube ratio reveals that ball cleaning operation is not yet well established. This can be attributed to its dependence on many interacting operating and design factors such as brine chemistry, type of inhibitor and control regime, ball type and MSF design parameters such as temperatures, number of stages and tube length, flow pattern and arrangement of ball injection points.

CONCLUSIONS

1. The common design features of SWCC MSF plants are that: they are all using brine recirculation mode with on-stream ball cleaning. They are also having cross-tube configurations except Jeddah II and IV which are having long tube configurations.

2. There is a wide diversity in the dimensions of the heat transfer tubes and flash chambers. The flashing brine flow rate per unit width for cross flow arrangements ranges between 478,600.6 and 965,307.6kg/hr m.

3. After more than 18 years of service, SWCC MSF distillers are currently operating with performance ratios equal to even higher than the corresponding design values. This attributed to SWCC strict maintenance and operating procedures.

4. Antiscalants dose rates are reduced to as low as 2.0 and 1.0 PPM for the low and high temperature operations, respectively.

5. All SWCC MSF plants are employing on-load ball tube cleaning with a ball to tube ratio in the range of 0.22 to 0.45.

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Discussion on the Optimization of Anti-Scale Chemical Dosing in MSF Plants

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DISCUSSION ON THE OPTIMISATION OF ANTI-SCALE CHEMICAL DOSING IN M.S.F. PLANTS

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ABSTRACT

At present, one of the most practical methods to produce a large mass of drinking water is by Multistage Flash (MSF) technology. The major operating concern in fresh water production by such technology is the scale formation and deposition on heat transfer surfaces. The build-up of scale affects the heat transfer mechanism, which result in reduction of production and poor performance of the plant.

Here, in Ghubrah Power and Desalination Plant, MSF technology is being used with two different designs (parallel flow type and cross flow type). Various types of anti scalant, at different dosing rates have been tried in order to inhibit scale formation.

This paper will discuss the scale control methods, which have been employed at Ghubrah MSF Plants and the use of the different anti-scalants with and without on-line mechanical cleaning.

Initially a low-temperature anti-scale additive was used in Evaporator No.1 (EVP-1) (parallel flow type), which was commissioned in 1976. Since 1979, an improved anti-scale chemical was adopted as scale inhibitor in EVP-1 and in all other units which were made later. The dosing rate of the improved chemical started at 4 ppm and was reduced at intervals to reach 1.5 ppm at all the units except EVP-1 and EVP-2, as unit no.1 has no on load mechanical cleaning system and unit no.2 was under trial test with a new anti scalant. Accordingly the cost of chemical consumption was reduced by 62.5%.

INTRODUCTION

Ghubrah plant started producing drinking water after the commissioning of evaporator no. 1 (EVP1) for commercial service in 1976.

The plant was made to employ MSF technology with long tube parallel flow design, which is built in 20 stages (2 of Heat Reject & 18 of Heat Gain). The other MSF units EVP2 to EVP6, which are cross flow type [3 Stages of Heat Reject & 13 stages of Heat Gain] were built in 1983, 1986, 1992 & 1997 respectively.

Evaporator no.1 is designed to produce 4 MIGPD with Polyphosphate and non guaranteed 6 MIGPD with Sulphuric acid. Decarbonator is provided to facilitate the efficient removal of CO2 and non condensable gases in case of acid dosing. This decarbonator had never been in use, as the plant was not operated with acid due to the fact that, acid dosing will enhance more corrosion to the plant interior structure. However a trail run with Sulphuric acid was conducted for few days.

Hence since the plant was taken over for commercial use, Polyphosphate was adopted as anti scale additive to give 4 MIGPD.

Polyphosphate can work only at temperature not more than 90 °C, as it is not stable in water with higher temperatures and revert to orthophosphate form. This orthophosphate reacts with calcium to form slugde, as Calcium Phosphate, which can strongly adhere to the heat transfer surfaces. Thus to improve the plant performance and output, Polyphosphate was replaced in 1978, by new improved chemical, based on Polymaleic Acid formation with pH~2 [PM-1] [See Table–1], which has the ability to retard the scale formation in desalination plants up to temperature above 113 Deg. C. Since then the total production in EVP1 was increased from 4 MIGPD to 5 MIGPD at Top Brine Temperature (TBT) of 90 °C & 104 °C respectively.

Table –1
Abbreviations for Additives

Abbreviation	Chemical Type
PM-1	Polymaleic Acid based – pH ~ 2
PM-2	Polymaleic Acid based – pH ~ 2
PM-3	Polymaleic Acid based – pH ~ 7
PH-A	Organic Phosphonic Acid based – pH ~ 10.7

Evaporator no. 2 to 6 designed to give 5 MIGPD with Polyphosphate and 6 MIGPD with improved chemical.

Unlike EVP1, a trail run with PM-1 was carried out during commissioning of EVP2 in 1983 and then adopted as scale inhibitor with the dosing rate of 4 PPM at TBT of 104 ° C, to give 6 MIGPD with on load mechanical cleaning. The same chemical was adopted in EVP3, EVP4 & EVP5 with the same dosing rate and Top Brine Temperature. The reduction in the PM-1 dosing rate was initiated with commissioning of EVP6 [3PPM] and continued, to reach 1.5 PPM at all the units except EVP1 & EVP2. As unit no. 1 has no on load mechanical cleaning system and unit no. 2 was under trail test with a new anti scalant. [Refer to table –2]

Table –2

Schedule of Belgard Dosing Rate Reduction

UNIT	DATE	DOSE RATE	TOP BRINE TEMP. ^{0}C
+EVP1	10.10.99	From 3 PPM to 2.5 PPM	104
*EVP2	11.10.99 09.11.99	From 3 PPM to 2.5 PPM From 3 PPM to 2.5 PPM	104
EVP3	21.11.99 18.01.2000 10.10.99	From 2.5 PPM to 2 PPM From 2.0 PPM to 1.5 PPM From 3 PPM to 2.5 PPM	104
EVP4	20.10.99 16.12.99 11.10.99	From 2.5 PPM to 2 PPM From 2.0 PPM to 1.5 PPM From 3 PPM to 2.5 PPM	104
EVP5	14.03.?2000 19.03.2000 06.05.99	From 2.5 PPM to 2 PPM From 2.0 PPM to 1.5 PPM From 3 PPM to 2.5 PPM	104
EVP6	12.10.99 15.04.2000	From 2.5 PPM to 2 PPM From 2.0 PPM to 1.5 PPM	104

\+ Further reduction could not be carried out due to aging of the plant and non availability of On Load Tube Cleaning system.

* From 31/12/1999, the plant is in trial run with new anti scale chemical [PH-S] for 10 month.

1.0 TRIAL RUNS OF VARIOUS ANTISCALANT

1.1 Evaporator no.1

This plant was commissioned in 1976 and operated with 4 PPM Polyphosphate additive, at 90 ^{0}C, Top Brine Temperature to produce 4 MIGPD. The plant is also designed to produce non guaranteed value of 6 MIGPD at Top Brine Temperature of 113 °C with sulphuric acid, but due to the risk of more corrosion, acid dosing had never been employed as scale inhibitor. Despite this fact the first trial test on this plant was conducted by using sulphuric acid on 16.08.78, at Top Brine Temperature of 114.5 °C with an average distillate production of 1130 M^3/Hr. (5.966 MIGPD).

The second trial run was carried out for 24 hours on 17-18, March 1979 with Polyphosphate, after the plant completed three years of service.

The treatment with polymer additive based on Maleic Acid [PM-1] started in 1978 with the dosing rate of 4 PPM at TBT of 104 °C to give 5 MIGPD.

Performance test with this polymer additive was carried out during the period from 09.02.1981 to 16.03.1981. The test was conducted at intervals with different dosing rates and Top Brine Temperature. From 9th February to 2nd March the test was done for uncleaned tube with different dosing rates 4, 5 & 7 PPM for Top Brine Temperature of 98 °C, 104 °C and 108 °C respectively.

At the brine top temperature of 98 °C, the GOR remained nearly constant at 6.07. For high temperature operation the plant was running normal and no considerable fouling was observed. The production was increased from the designed value 4 MIGPD for top brine temperature of 91 °C to 5.28 MIGPD at 108 °C.

On 3.03 1981 the plant was acid cleaned and it was put back in service on 04/03/81 for the continuation of the trial run with the same antiscale dosing rate and Top Brine Temperature. After acid cleaning the Gain Output Ratio and Heat Transfer Co-efficient increased from 6.02 and 2328 W/M^2 °C to 6.55 and 2820 W/M^2 °C respectively. These values remained virtually constant during the trail run. This indicates that PM-1 was effectively retarding and inhibiting alkaline scale formation. Then the plant continued with the same steady performance, producing 5 MIGPD for 354 days and before the shutdown GOR reached 6.14.

Another new anti scalant which is Carboxylic Acid based Polymer had also gone on trail test in EVP1. The performance test was carried out for 13 month from 06/11/1982 to 23/12/1983 with this chemical. The unit was acid cleaned on 15/12/1983 and the test was continued for another 36 days, from 12/01/1984 to 18/02/1984. The trial run started with a dosing rate of 6 PPM for Top Brine Temperature of 98 °C. The dosing rate was gradually decreased to 5 PPM and Top Brine Temperature was increased to 101.90 °C to produce 5 MIGPD. The Top Brine Temperature was further increased to 104.20 °C and the production reached 5.38 MIGPD. During the trial run the plant was steady, except the blockage of the dosing line. This chemical additive was forming a lot of sticky material in the solution preparation tank, which cause blockage of the dosing line. To over come this problem, a 100 mesh stainless steel strainer was fixed at the outlet of the preparation tank. This sticky material was found soluble in dilute Caustic Soda.

In the normal practice in MSF plants the additive treatment is accompanied by on line mechanical cleaning using sponge rubber balls. This method removes any soft scale deposited on the heat transfer surfaces. But on line mechanical cleaning system has never been in service in EVP1 which is still giving satisfactory performance.

1.2 Evaporator no. 2

Evaporator no. 2 had gone a trial run with two chemical additives, PM-1 and another anti scalant based on Maleic Acid with pH~ 2 [PM-2]. A trial run with PM-1 was carried out for 135 days during the period from 11/12/1983 to 24/04/1984. The G.O.R. values varied between 6.96 and 7.27 with average distillate production of 1138 M^2/Hr. The dosing rate of 4 PPM of PM-1 at Top Brine Temprerature of 100.6 °C, combined with on line mechanical cleaning (3 cycle per day, using 360 balls) was found to give good performance and retarded scale formation efficiently.

PM-2 antiscalant was also successfully, went on a trial run in EV2 for 94 days. The test showed, when the unit operated at Top Brine Temperature of 101.20 °C, with 4 PPM dosing rate, combined with sponge ball cleaning, a good performance was attained and the rate of alkaline scale deposition was effectively controlled.

Two chemical additive beside PM-1 and PM-2 were used in EVP2 namely Polyphosphate and Polymaleic Acid based new anti scalant with pH~7 [PM-3], [See Table 3]. Eventually and since 1995, PM-1was adopted as scale inhibitor in EVP2 and there after in all Ghubrah MSF plants.

Table-3

Anti Scale Chemicals used in EVP2

DURATION	CHEMICAL ADDITIVE	DOSING RATE	TBT °C
October 1982 to March 1983	Polyphosphate	4 PPM	90
April 1983 – November 1983	PM-3	6 PPM	104
November 1983 – September 1985	PM-1	4 PPM	104
September 1985 – May 1995	PM-2	4 PPM	104
May 1995 – Onwards	PM-1	4 PPM *	104

* The dosing rate was reduced in July 1997 from 4 PPM to 3 PPM and 2.5 PPM in October 1999.

2.0 REDUCTION IN THE ANTI SCALE DOSING RATE

Since 1995 PM-1 has been adopted as scale inhibitor in Ghubrah MSF plants. On 4th May 1995 PM-2 was replaced by PM-1 in EVP2 with a dosing rate of 4 PPM. Ultimately the same has been employed in all the other units. The performance of all the units remained the same as good as with PM-2. In EVP2 , the Brine Heater and Recovery Section resistance for the month of August' 95, (After three month of operation with PM-1) were 0.0003072 M^2 $^0C/W$ & 0.0002996 M^2 $^0C/W$ respectively in comparison with 0.000320 M^2 $^0C/W$ & 0.0002995 M^2 $^0C/W$, the average Brine Heater & Recovery section resistance for April' 95, just before the change over. Though the thermal performance showed that PM-1 has maintained the plant in good condition.

In March 1997 evaporator no. 6 was handed over for commercial service and PM-1dosing rate was adopted at 3 PPM. Consequently in July 1997 reduction in the anti scale dosing rate from 4 PPM to 3 PPM was carried out for EVP1 to EVP5. Further reduction in all the units was carried out on intervals to reach the target of 1.5 PPM as a minimum and optimum anti scale dosing rate [Table 3]. The reduction was done with close monitoring and daily watch of all the plants parameters.

EVP1 was exempted and the minimum dosing rate was maintained at 2.50 PPM.

In EVP2 the reduction in PM-1dosing rate was stopped at 2.5 PPM, as the plant has started a trial run with new chemical additive based on Organic Phosphonic Acid [PH-S] on 31/12/2000 for a period of 10 month.

EVP3 & EVP4 were the first two units to reach 1.5 PPM. At the end of August 2000 EVP3 had completed more than 7 months of service with 1.5 PPM. Therefore this study includes the performance curves [Fig. 1 A/B/C/D] of this unit with 4 PPM, 3 PPM and 1.5 PPM anti scale dosing rate.

2.1 Reduction form 4 PPM to 3 PPM

To compare the performance with these two dosing rates, EVP3 is considered.

Refer Figure 1 - A, it can be seen from the 3 PPM graph that heat transfer resistance for the Brine Heater and Heat Recovery section remained reasonably consistent with very slight increase. This increase is clearly occurred after the similar period of service with 4 PPM.

Similarly in Fig.1-B the Economy Ratio (Kg distillate/ Kg condensate) and Brine Heater Condensate flow graph for 4 PPM & 3 PPM dosing rate behave in the same manner, as the trend in 3 PPM curve for Economy Ratio and B.H. Condensate similar to that of 4 PPM over the same period of operation. Hence the reduction of PM-1dosing rate from 4 PPM to 3 PPM gave excellent scale control too.

2.2 Reduction from 3 PPM to 1.5 PPM

This reduction was achieved at gradual steps. EVP3 performance curves are considered.

2.2.1 Evaporator no. 3

The target of 1.5 PPM was reached on 18.01.2000 in this plant. Considering figure 1-C the heat transfer resistance curves for 1.5 PPM and 3 PPM dosing rate remained steady with a reasonable increase. In the 1.5 PPM curve, after completion of more than 6 month of service [July 2000] with 1.5 PPM dosing rate, slight increase in the heat transfer resistance can be seen, in comparison with 3 PPM curve over the same period of service. This small degree rise, in the heat transfer resistance can be obviously noticed in the case of Heat Recovery resistance curve and it is negligible as the overall process performance found to be satisfactory.

In figure 1-D the Economy Ratio curves for both 3 PPM and 1.5 PPM remained similar with slight decline over the same period of operation. Similarly the Brine Heater Condensate flow curves [Figure 1-D] for 3 PPM and 1.5 PPM dosing rate, behave at the same manner, indicating slight increase during the same period of service.

Hence from figure 1 [C/D] the behavior of EVP3 performance with 1.5 PPM [PM-1] dosing rate during the above mentioned period of operation found to be steady and quiet satisfactory, as there is no sudden hike either in heat transfer resistance or Brine Heater Condensate flow. Likewise Economy Ratio, which is found to be satisfactory too.

2.3 Reduction of Polymaleic Acid based Chemical [PM-1] & Trail of Phosphonic Acid based Chemical [PH-S] in EVP2

After year 1997, EVP2 had experienced one more reduction in PM-1dosing rate. On 11/10/99 the dosing rate was reduced from 3 PPM to 2.5 PPM and the plant was running with satisfactory performance till the annual shutdown on 09/12/99. From 31/12/99 on word, the plant is running on trail test with [PH-S] anti scalant. At the beginning of the trail run On Load Tube Cleaning system's number of balls charged every 4 days was increased from 200 to 360 and number of cycles from 2 to 3 cycles/day.

2.4 Reduction in Anti Scale Dosing Cost

Cost of PM-1 Anti Scalant = US $ 1857.6 / Ton

Consider the following table :-

UNIT	DOSING RATE	COST OF PM-1/ TON OF DISTILLATE
EVP2-EVP6	3 PPM 1.5 PPM	US$ 0.0147 US$ 0.00733
EVP1	3 PPM 1.5 PPM	US$ 0.0151 US$ 0.0126

SAVING PER DAY FOR THE TOTAL PRODUCTION OF EVP2 TO EVP6
= (0.0147 – 0.00733) x 1140 x 5 x 24 x = US$ 1008.216

SAVING PER DAY FOR THE TOTAL PRODUCTION OF EVP1
= (0.0151 – 0.0126) x 958 x 24 = US$ 57.48

THEREFORE THE TOTAL SAVING IN COST/ THE TOTAL PRODUCTION OF ONE DAY = 57.48 + 1008.216 = US$ 1065.696

which is a big saving and contributes to great extent on the total running cost.

CONCLUSION

Trial test of any anti scalant needs special arrangement i.e. experimental pilot plants. Such arrangement can facilitate comprehensive studies and information about the different type of chemicals available and their key roll to inhibit scale formation on heat transfer surfaces. However such facility is not available at Ghubrah Plant, many trials with different anti scale chemicals at different dosing rates were done and came out with useful results in selecting the proper additive and optimum dosing rate. In addition to that monitoring the plant parameter and the best way to keep it running with good performance and output.

Since the chemistry of Gulf water varies from area to another and ultimately the operational condition. The optimum dosing rate of any chemical additive at a certain area is not necessary to be applicable elsewhere.

Though from the wide experience with different chemical additive, PM-1 is found to be the suitable chemical additive, operational and cost wise to retard scale formation. As its dosing rate was successfully reduced from 4 PPM to reach the minimum and optimum level of 1.5 PPM.

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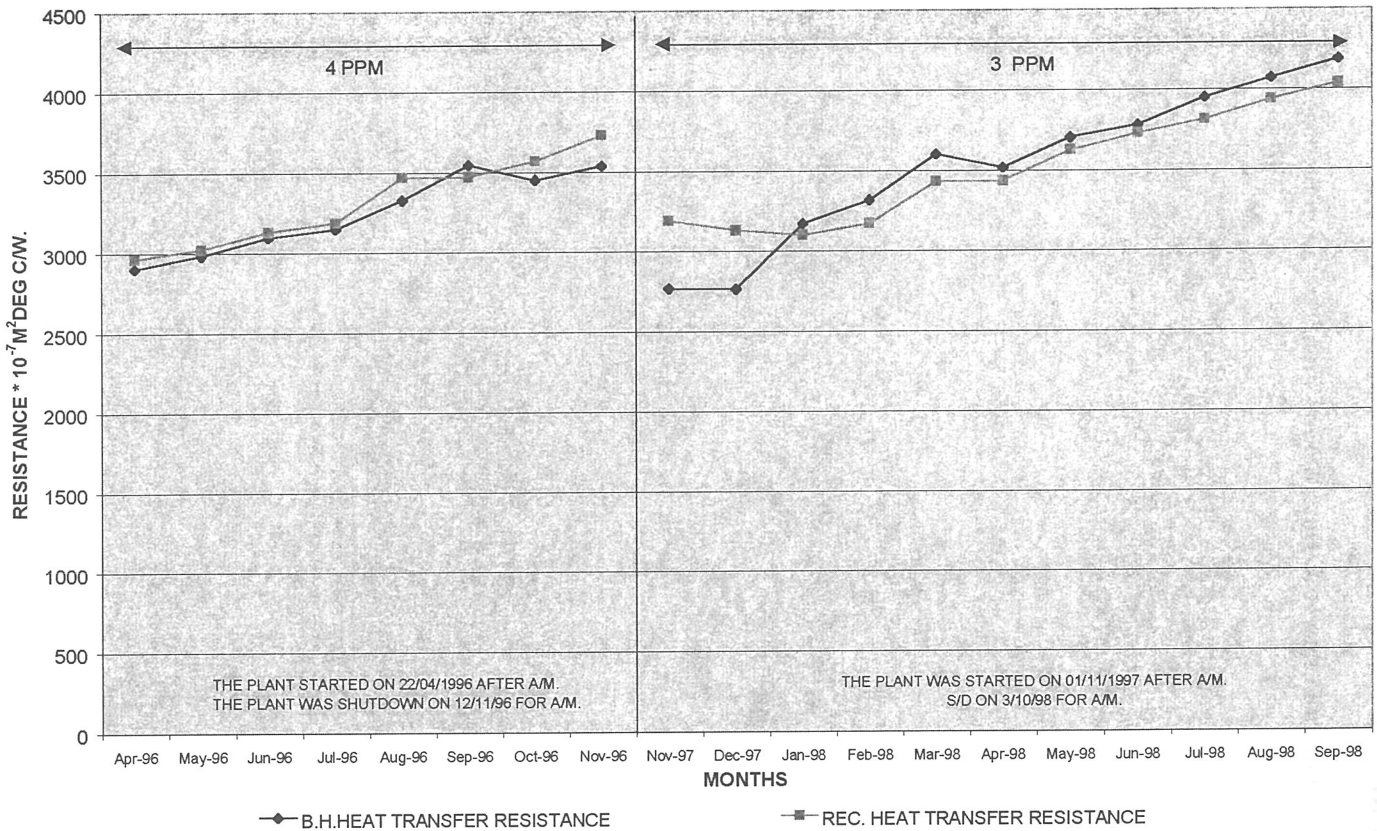

FIG. 1-A: EVP.#3- COMPARISON BETWEEN 4 PPM & 3 PPM (PM-1)
(PLANT PERFORMANCE)

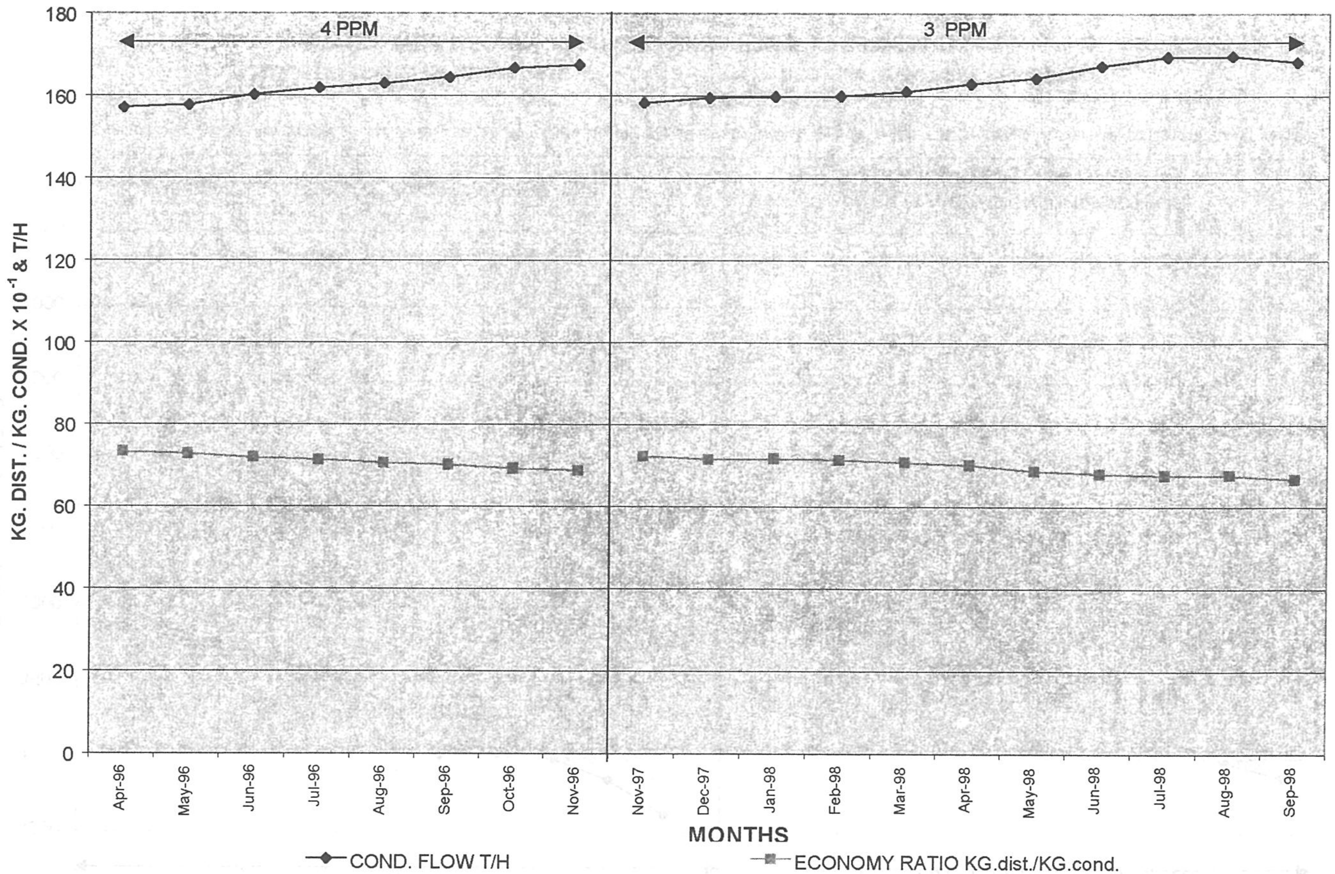

FIG. 1-B: EVAP. #3 - COMPARISON BETWEEN 4 PPM & 3 PPM (PM-1)

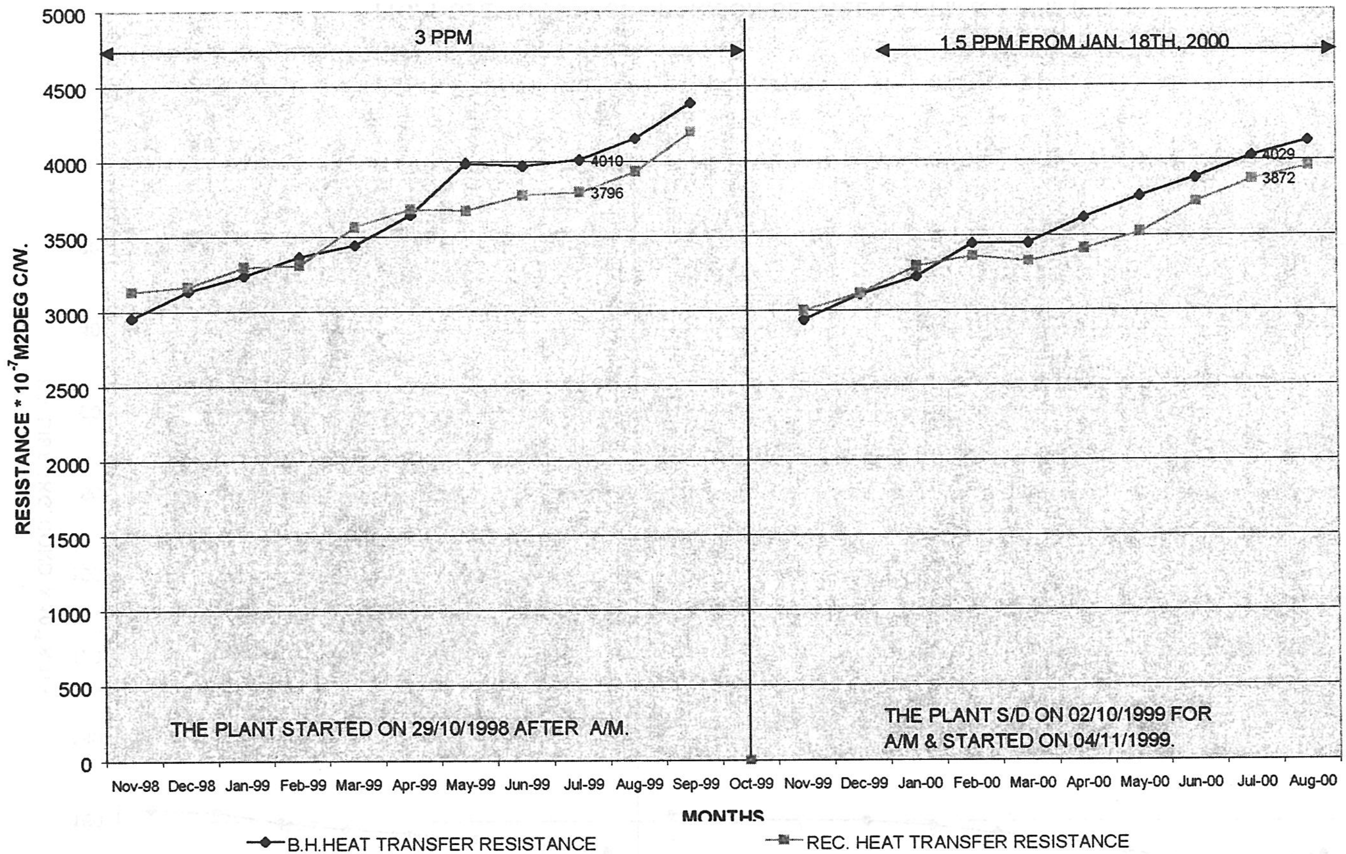

FIG. 1-C: EVP.#3- COMPARISON BETWEEN 3 PPM & 1.5 PPM (PM-1) (PLANT PERFORMANCE)

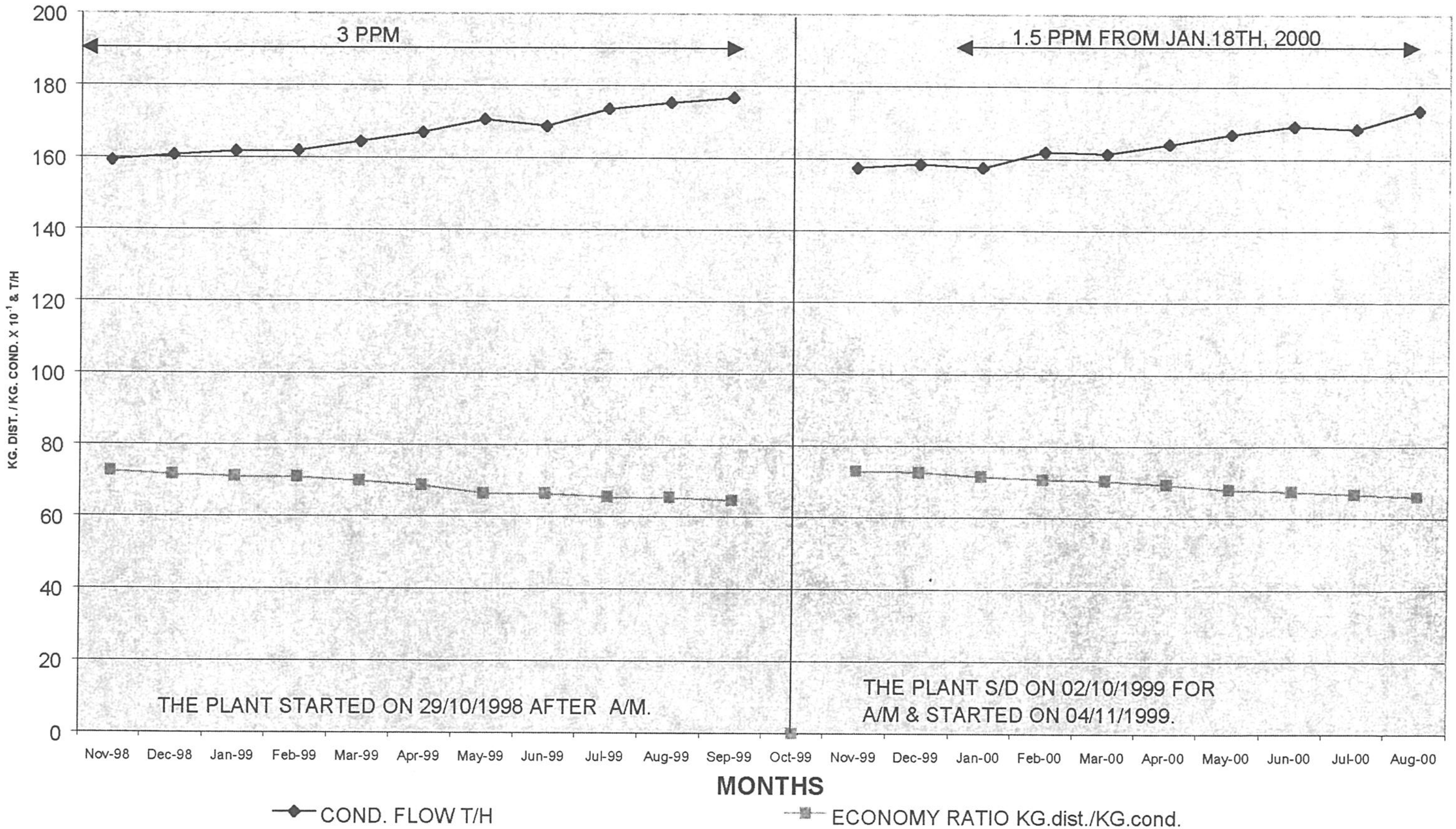

FIG. 1-D: EVAP. #3- COMPARISON BETWEEN 3 PPM & 1.5 PPM (PM-1) (CONDENSATE FLOW)

Corrosion and Mechanical Behavior of Fusion Bonded Epoxy (FBE) in Aqueous Media

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CORROSION AND MECHANICAL BEHAVIOR OF FUSION BONDED EPOXY (FBE) IN AQUEOUS MEDIA

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ABSTRACT

In recent years, there are many reported cases of corrosion failure in cement concrete pipelines. In the majority of cases, the failures have been attributed to rebar corrosion which is caused by the permeability of chloride from low resistivity soil and the subsequent attack on the passive layer of iron bar in the structure. As a possible alternative to cementitious materials, some organic coatings based on olefin, vinyl or epoxy-based polymers have been considered. However, due to the paucity of data on the behavior of these coatings in aqueous media particularly product water, the possibility of their application in water transmission systems in the Kingdom has not been fully exploited.

This paper deals with the studies carried out on the corrosion and mechanical behavior of fusion bonded epoxy (FBE) coating on steel in aqueous media which includes product water, distilled water and saline water. The mechanical testings on coating include adhesion, bending and cathodic disbondment testings. The corrosion studies include immersion testing under static and dynamic conditions, autoclave tests and accelerated (salt-fog) tests.

The analysis of results indicates chemical inertness of FBE coating in either of the aforementioned water used during testing, good adhesion and no damage to the coating during bending. Cathodic disbondment tests indicate that FBE coating sustains under cathodic protection (CP) conditions. In general, the results of the mechanical and corrosion tests indicate that FBE is a promising material for internal coating on steel in water transmission systems.

Key words : Fusion Bonded Epoxy, Mechanical testings, Immersion testings, Cathodic Protection, Cathodic disbondment, Steel, Product, Distilled, and Saline waters

INTRODUCTION

In recent years, there are several cases of pipe failure in the form of leakages, bursting or cracking of the pipelines [1-4]. In the majority of cases the failures have been attributed mainly to rebar corrosion which is caused by the permeability of chloride from low resistivity soil. The problem is acute in areas where the soil, besides having a high chloride content, has intermittent dry and wet spells. As a possible alternative to cementitious materials, in recent years, a number of olefin, vinyl and epoxy based polymer coatings have appeared in the market. Epoxy based paints or coatings have been employed as internal linings as well as external coatings in a considerable number of pipelines in the Kingdom. In recent years, polyethylene coatings having an FBE primer and a 2-layer polyethylene based material extruded over it has been used extensively throughout the world as an external coating for open or buried pipe lines [5,6]. In such coatings, the epoxy provides good adhesion to steel and good cathodic disbonding characteristics, which are combined with water barrier and the favourable mechanical properties of polyethylene. The combination has better adhesion, cathodic disbonding resistance, hydrolytic stability and impact strength than either coating used by it self. For internal coatings, fusion bonded epoxy (FBE) is a promising material and has been used for transmitting water though on limited scale [7].

For application in water transmission systems, using organic coating in general, and FBE in particular, the effect of chloride contamination on coating performance is an important objective of the study.

Soluble salt contamination can cause premature failure in virtually all types of coatings. Amongst the anions, chloride is the single most damaging anion because it migrates under coating film. Chloride containing solution has a high osmotic pressure contributing to moisture penetration, loss of adhesion and blistering. The source of chloride contamination could be from the environment around the metallic surface. Alblas and London [8] reviewed the literature concerning the effect of chloride contamination on the corrosion of coated steel surfaces [9].

Helvig [10], Weldon [11] and Flores [12] showed various correlations between the level of chloride and premature coating failures. These investigators applied the contaminant (chloride) in known quantities to the steel surface and applied the coating shortly thereafter. Niel and Whitehurst [13] used chloride contamination that remained in the micropits after sand blasting of a steel surface for studying FBE coating performance. They found that in the presence of a pitted surface, chloride contamination can cause serious loss of performance in FBE coatings in hot cathodic disbonding and hot water tests. For underground coatings and other immersion coatings in critical applications, a maximum chloride level of 2 mg/ cm^2 was suggested[14].

A review of the important work carried out on the performance of FBE coating as cited above reflects that the coating appears to have characteristic properties that are required for application in water transmission systems. However performance data about FBE coatings is sketchy and therefore, a systematic study of its corrosion and mechanical behavior is important and the work described in this paper was carried out by keeping this point in mind.

EXPERIMENTAL

(a) Materials

Fusion Bonded Epoxy FBE-X was employed for the studies. The coated species acquired commercially were of the following dimensions and quantity.

Thickness	:	~20-21 Mils	
Coupon size (mm) and quantity	:	100 x 50	50 Nos.
		200 x 200	50 Nos.
		200 x 25	25 Nos.

(b) Equipment

The following instruments were utilised for carrying out the experimental work : Salt spray cabinet; Holiday Detector, with Calibration Meter- model AP-W; Tinker and Rasor; Coating Thickness Meter, Elcometer Model 345 and Posi Tector-2000; Adhesion Meter from DYNA Proceq, Zurich, Switzerland; Cathodic Disbondment Testing Assembly.

MECHANICAL TESTING

(a) Adhesion Test

Adhesion tests on FBE coated steel samples were carried out at 25 °C using ASTM D4541 – 85 (Re-approved 1989) Technique.

Pull-Off adhesion test and/or crosscut methods were employed to determine the adhesive strength of the coatings. An adhesion tester consists of dollies made of aluminum, which are glued perpendicular onto the coated surface of the samples. After the curing of adhesive (glue) testing apparatus is attached to the loading fixture and aligned to apply the tension normal to the test surface. The force applied to the loading fixture is then gradually increased and monitored until either plug of coating material is detached or a specified value is reached.

The relative stress applied to each coating can be calculated as follows:

$$X=\frac{4F}{\pi d^2}$$

where

X : Greatest mean pull-off stress applied during the pull off strength achieved at failure (psi)

F: Highest force applied to the test surface (pounds)

d: Equivalent diameter of the original surface area stressed (inches)

In the crosscut adhesion test, a crosscut was made with the help of a utility knife on the coated surface deep into the metal substrate. At the crosscut the blade of the knife was inserted under the coating and with a levering action, force was applied to chip off the coating. The chipped off area was observed under a microscope (magnification x 40) to see the extent of removal of the coating from the substrate.

(b) Flexibility Test

Mandrel Bend Machine was used to test the steel specimens coated with FBE, at room temperature. The main objective of the bending test is to determine the strength of the coating under bending condition. Prior to bending, the specimens were inspected visually for any visible defect followed by holiday test. The FBE coated samples were tested by pulse type detector set at 2500 ±50 Volts. The thickness of the specimens was also measured before the test. The specimens were then clamped in the holder and bent flat-wise at 60 °C over a thick shoe, bending was accomplished in approx. 30 seconds. After bending was completed, specimens were again inspected by holiday detector to confirm any cracking in the coating after bending the specimens.

It was programmed to carry out the flexibility test by increasing the mandrel radius step by step until the coating stops failing. The smallest available mandrel shoe was 87 mm radius. The present strain was calculated as given bellow:

$$\%Strain = \frac{100\ t}{(2\ r + t)}$$

Where:

t = Effective thickness of the specimens (DFT + Metal)

r = Radius of the mandrel shoe

DFT = Dry Film Thickness

The percent strain is directly proportional to the effective thickness "t" of the specimen.

(c) Cathodic Disbondment Test

The tests were carried out with FBE specimens for duration of 7 days (40 ^{0}C) and 4 weeks (25 ^{0}C) employing CAN / CSA-2245.20 M92 method.

This test provides accelerated adhesion assessment and determines resistance of the coating to cathodic potential and current flow. Coated steel samples of dimension 200 x 200 mm were used for the tests. In the middle of the coated specimen a hole of 3.2 mm diameter was drilled through the coating to expose the substrate. A 200 mm long plastic pipe of 100 mm diameter was glued on to the specimen with the holiday at the center of the tubing. A cathodic disbondment test cell was assembled with a DC power supply, platinum wire as anode, high resistance volt/ amp meter and a calomel reference electrode. The DC power supply was designed and fabricated at the research and development center by the instrumentation section. The advantage in using this power supply is that it keeps the applied potential constant irrespective of current flowing through the cell. Test specimens glued with the plastic pipe were kept on a hot plate and 900 ml solution of 3% NaCl was poured in each plastic pipe. The temperature of the hot plate was raised to maintain the temperature of the NaCl solution at 40 °C.

The negative lead of the power supply was connected to the coated plate and positive lead to platinum anode. After 7 days, the electrolyte was drained out and the test cell was immediately dismantled. The coated plate was cooled down to room temperature. The blade of a hard utility knife was inserted under the coating near the holiday edge and using a levering action, the coating was chipped off. This action was continued till it became impossible to flake off the coating. Radius of the disbonded area from the holiday edge was measured along seven different directions and the average was taken.

CORROSION TEST

(a) Salt Spray Test

Salt spray tests were carried out in a salt spray fog chamber following ASTM B117 – 90. During salt spray tests, the development of corrosion on some abraded area was studied. In one set of samples, scratch lines (scribes) were made through one corner of the samples to the diagonally opposite corner of the sample, i.e. "X" shaped. One side of the coupons was scribed while the other side was left unscribed. The specimens, without the scribe mark, were weighed before starting the salt spray test.

In the salt spray chamber the specimens were placed meeting the following conditions:

(i) All the specimens were supported parallel to the principal direction of horizontal flow of fog

(ii) Specimen holder was made of plastic and, therefore, specimens were not in contact with one another or with any metallic material

(iii) A 5% solution of sodium chloride was atomized by compressed air in the chamber

(iv) The temperature of the chamber was kept at 38 °C

Specimens were exposed to the above-mentioned conditions for 25, 50, 75 and 100 days, respectively. After the required exposure period, the samples were examined as per ASTM D1645-71a (Re-approved 1984). This method provides a means of evaluating and comparing basic corrosion performance of substrate, pretreatment, or coating system, or a combination thereof, after exposure to corrosive environment. The specimens were carefully removed from the holder and gently washed in clean running water, to remove salt deposits from their surfaces, and then immediately dried. Exposed surface at the scribes was cleaned with brush to remove all the rust. Mean creepage from the scribe and failed area was measured and rated as per ASTM D1654-71a. Similarly, measurements were also carried out for the blisters appeared on scribed and unscribed sides.

(b) Close Circuit Loop Test

Specimens of FBE, were fixed in coupon holders and installed in an indigenously designed and fabricated close circuit loop. The experiments were carried out under the following conditions : Temperature: 40 °C; Medium: Distilled water; Duration: 4 Weeks; Flow Rate: 60 GPM

(c) Autoclave Test

The test was carried out on an autoclave at 1500 psi, 40 °C in distilled water for 48 hours. The specimens were half immersed in the test solution during the test. The thickness of coating was measured before and after each test using a electromagnetic thickness gauge Posi-Tector 2000 at 6 different places (3 in aqueous and 3 in vapor phase) on the specimen. After completion of the test the samples were assessed visually for swelling and blistering etc. The pull-off adhesion test was also carried out on each phase i.e., vapor and aqueous phases.

The test were carried out under the following test conditions :

(i)	Pressure	:	1500 psi
(ii)	Temperature	:	40 °C
(iii)	Atmosphere	:	Nitrogen gas
(iv)	Test medium	:	Distilled water
(v)	Duration	:	48 hours

RESULTS AND DISCUSSION

1. Adhesion Test

In all the tests the dolly was detached at the coating/ dolly interface. This confirms that the bonding between the metal substrate and coating was more than the coating and dolly. The adhesion test results were not consistent i.e. a large difference among the data was observed. The maximum adhesion strength between coating and dolly was 345 psi. Adhesion test was also carried out on samples used for autoclave test at AQPC facility. Here again the strength of the glue and coating was not enough to pull-off the coatings. Maximum adhesion strength of glue used to fix the dollies to the coatings was around 500 psi. Pull off adhesion test results obtained from DYNA adhesion test with 50mm φ dolly are as follows :

S. No.	Coating Type	Adhesive Strength (psi)		
		#1	#2	#3
1.	FBE-X	150	210	345

2. Flexibility Test

After bending the samples were examined visually followed by holiday test at the bend site. No defect was found either visually or by holiday tester on any of the samples . The tests show that with a thickness (DFT + metal) of 4.5 mm, FBE can sustain up to 2.41% strain.

3. Cathodic Disbondment Tests (CDT)

An increase in pH value from 4.5 at the start of the test to 8.5-9.0 at the end of all tests was recorded. Radial Disbondment (RD) results obtained from the CDT at 40°C are given below. The FBE coated samples showed the average RD value of 1.96 mm.

S.No	Coating Type	Sample #	Average DFT (mils)	Radial Disbondment (mm)	Average Disbondment (mm)
1.	FBE-X	A	23.0	1.7	1.966
		B	24.4	2.2	
		C	27.7	2.0	

At 25°, FBE showed almost negligible average RD. However, no deposits on or underneath the coated surface were found.

4. Salt Spray Tests

Specimens with and without scribe exposed to the salt fog were evaluated with respect to mean creepage (from scribe) and blistering. The salt spray results for FBE coating are summarized below:

Coating	Exposure (days)/ creepage of coating (mm)				Rating	Visual Examination Remarks
	25	50	75	100		
FBE-X	0.09	1.27	1.57	2.06	6	No blistering

FBE coating (green) shows little creepage (0.09 mm) after 25 days of exposure but it is increased considerable (2.06 mm) after 100 days exposure although no blistering in the coating was found. The scribed samples show a number of blisters where as unscribed samples are devoid of any blister. It is interesting to note that, a decrease in number of pits was found from 50 to 100 days of exposure in the salt spray chamber. The maximum number of blisters were found on the scribed side.

5. Close Circuit Loop Test

Coupons of FBE were exposed to distilled water in the close circuit loop at 40 °C for 1 month under flowing condition (Flow rate 60 GPM). All the samples were intact and no remarkable change in the physical condition of the coating was observed.

6. Autoclave Test

The autoclave tests were carried out in order to know the behavior of coatings under high pressure and temperature. The test duration was 48 hours and the temperature was fixed to 40°C. The pressure of the test vessel was kept at 1500 psi. After the test samples were examined for color, blistering, loss in adhesion strength and thickness. The data which present the thickness of FBE-X coating in vapor and aqueous phases, respectively are summarized below:

S.No.	Phase	Before Test (mils)	After Test (mils)	Change in thickness (mils)	Average change in thickness (mils)
1.	Vapor	20.26	20.93	+ 0.67	+ 0.53
2.	Vapor	21.26	21.66	+ 0.39	
3.	Aqueous	20.56	21.93	+ 1.36	+ 0.93
4.	Aqueous	21.40	20.90	+ 0.50	

A slight increase in thickness can be seen in both phases. FBE coatings do not show any loss of color in autoclave test. While carrying out Pull-Off adhesion test on the panels of FBE after the autoclave test, again failure (at 500 psi) of dolly and coating was observed as shown below:

S.No.	Sample #	Physical Appearance	Adhesion Test Gas Phase	Aqueous Phase
1.	1	No Blistering or Swelling	Glue Failure	Glue Failure
2.	2	No Blistering or Swelling	Glue Failure	Glue Failure
3.	3	No Blistering or Swelling	Glue Failure	Glue Failure

CONCLUSIONS

(i) The results of adhesion tests carried out on FBE, show that the bonding between the metal substrate and the coating was more than the coating and dolly.

(ii) The flexibility test (bending test) carried out on FBE shows no defect or presence of holidays at the bending site. The coating can sustain up to 2.41% strain.

(iii) The results from salt fog tests show following behavior of FBE coating:

(a) In scribed samples, the creepage increases with increasing exposure time: from 0.09 mm (25 days) to 2.06 mm = (100 days).

(b) No blistering was observed after 100 days exposures

(iv) The pull off adhesion tests carried out on coated samples after autoclave tests show that the adhesive strength of FBE, coatings is greater than 500 psi.

(v) The results of the autoclave tests indicate that FBE coating shows very small variations in thickness in vapor phase and a definite increase in aqueous phase. Moreover, no loss of color was found.

(vi) Close circuit loop tests results of 1 month exposure in distilled water indicate no marked change in the color and texture of the coating. There was no perceptible change in weight.

(vii) FBE has good mechanical properties, low water permeation, no chemical degradation and good corrosion resistance. Moreover, FBE shows small increase in radial disbondment under applied potential (1.5 volts Vs SCE) thus indicating its stability towards cathodic disbondment. This combination of properties provide FBE as a suitable choice for internal and external lining material for steel pipes in water transmission systems.

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Impact of Four Different Operating Trials on Performance of a Seawater Reverse Osmosis Plant on the Red Sea Coast

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IMPACT OF FOUR DIFFERENT OPERATING TRIALS ON PERFORMANCE OF A SEAWATER REVERSE OSMOSIS PLANT ON THE RED SEA COAST

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ABSTRACT

Serious membrane fouling has been experienced in a seawater reverse osmosis (SWRO) plant situated on the Red Sea coast of Saudi Arabia. Four different operation trials were made (in four phases under normal and modified process conditions) to find out the optimum conditions that would minimize membrane fouling. The first trial included normal plant operation, where the coagulant (polyelectrolyte) is dosed upstream and sodium metabisulfite (SBS) downstream the media filter, ahead of the micron cartridge filter (CF). During the second trial the coagulant dosing point was shifted close to the seawater intake pump providing increased residence time for proper coagulation prior to filtration. In the third trial, to avoid any biological growth within CF, the SBS dosing was shifted to the downstream of CF. Chlorine dosing was at the intake chamber during all these trials. In the fourth trial, the plant was operated without chlorination but with the coagulant dosing as in the case of second trial. The first two trials did not improve either the pretreated seawater quality or the membrane flux, but the latter two gave some positive results. A significant decrease in the rate of DP build-up across cartridge filter with increased rate of membrane fouling (DP) was observed during the third trial. Compared to earlier trials, the 4th trial resulted in an increase in the rate of coarse and cartridge filter clogging, a high SDI value, a lower rate of DP build up across membranes and, a lower decline in flux rate. However, when this trial was repeated for a longer period on a latter occasion, the clogging of coarse and dual media filters as well as the SDI values were within acceptable limits, indicating variability of water quality during these two test occasions. The difficulty encountered in pretreating the low quality feed water by the existing pretreatment facilities resulted in low plant availability. Otherwise, membrane performance was better when plant was operated without chlorine compared to that with chlorine. Successful operation may be possible by rectification of feed water system and other process deficiencies, e.g., back-up cartridge filters, improved backwash and filtration systems. The paper discusses the results obtained from these trials and recommends measures to alleviate the fouling and associated problems encountered in the present modes of plant operation.

Key words : SWRO Fouling, Chlorination/Dechlorination, Pretreatment, Feed Water Quality

INTRODUCTION

Al-Birk is a small town situated on the southern Red Sea cost of Saudi Arabia, about 450 km south to the port city of Jeddah. A 2270 m^3/d SWRO plant, utilizing DuPont B-10 6840 polyamide membrane, later changed to B-10 6835 in the first and B-9 in the second stage, was constructed and commissioned in the year 1983. Though the initial performance of the B-10 membrane was acceptable, within three months of operation DP exceeded its maximum value of 4 bar due to membrane fouling. Consequently, salt rejection as well as productivity of the plant dropped considerably. A concern on the seriousness of fouling in this plant has been reported in a number of papers and reports [1-4]. Recently, the intensity of fouling became so alarming that it necessitated membrane cleaning and cartridge filter (CF) replacement at 10 days interval [5].

The phenomenon of fouling is site and membrane specific. Whereas in one of the site the addition of cationic polyelectrolyte was assumed to be the cause of membrane fouling [6], in another site the cationic polymer was found to improve pretreated water quality [7]. Experiments in the Arabian Gulf coastal seawater showed that chlorine increases the SDI value of pretreated feed by 1 unit due to some interaction with seawater [8], but in the Red Sea coast SWRO plant, chlorination reduced the SDI of pretreated feed [7]. According to some authors, chlorination results in the formation of a micronutrient called assimilable organic carbon (AOC) that acts as food for the surviving bacteria following the dechlorination step, which in turn leads to an increase in bacterial growth as well as biofouling potential[9]. Adsorption of organics on membrane surface was found to decline permeation [10-11]. The heavy metal content in the coagulant ($FeCl_3$) was found to degrade SWRO membrane [7].

The site and seasonal variation of seawater quality have stimulated substantial diversity in the SWRO pretreatment process design, operation technique and ultimately trouble shooting steps. Depending on localized situations, most of the SWRO membrane fouling problems are solved by developing localized procedures, either by modifying process design or developing alternate mode of operation. Site specific study is required prior to proposing any modification that would help to overcome operational troubles and membrane fouling. Many site specific operation trials with manipulation of disinfectant usage have been reported. Intermittent chlorination [12-13], no chlorine operation [14-15] and operation with alternate disinfectants [16-17] are some of the trials reported to be successful in different sites. Four operation trials were considered to overcome the fouling problems experienced at Al-Birk plant. In the first two, manipulations in pretreatment system and in the second two, some modifications in disinfection usage were considered. The study was carried out jointly with DuPont, the manufacturer and supplier of B-10 membrane used at Al-Birk SWRO plant. This paper presents the results of these trials and discusses effect of the different operation modes on filtrate quality as well as performance of the membranes. A set of recommendations are proposed to tackle the fouling and associated problems in the Al-Birk plant on a long-term basis.

BRIEF DESCRIPTION OF AL-BIRK SWRO PLANT

A schematic flow diagram of the plant is given in Figure 1. The plant consists of two individual trains (Train 100 and Train 200), which are fed by seawater received through a concrete open intake chamber at a depth of 6 to 7 meters below water level and about 300 meters from sea shore. The incoming seawater feed is screened ahead of seawater pump by a traveling band screen of mesh size 1 mm. Downstream seawater pump, the flow is split into two streams, one to train 100 and the other to train 200. The entire experiments were conducted on train 200 (see Figure 2). Each train has individual feed pretreatment and desalination section. The desalination is conducted in two stages. The first stage utilizes DuPont B-10 HFF SWRO membranes, while the second stage utilizes DuPont B-9 brackish water membranes. The first stage of each train has six identical banks/ racks to hold membranes. Each rack normally contains 18-19 permeators. The second stage contains one bank per train with three rejects staging. Normally, 25 permeators are installed in the second stage bank. The number of permeators may vary in all stages, lower in number when membranes are new and is higher in number after long service (approx. one year) to maintain steady product flow. Part of the second stage reject is used as feed to first stage. Final product is transferred to product tank via suck-back tank. The designed product recovery for first stage is 35%. Permeate flushing of membranes actuates automatically, at any loss of pressure during emergency or normal shutdown, at a flushing rate of 40 m^3/ h.

Normal Operating Conditions

Seawater from an open sea sub-surface zone is chlorinated at the intake chamber (Figure 1) by dosing about 3.3±0.2 ppm calcium hypochlorite Ca(OCl) followed by about 2.5 ppm coagulant at up-stream of dual media filters (DMF). The coagulant is a polyelectrolyte (PE), magnifloc C-573, a solution of poly quaternary amine in water. The filtration is done initially by coarse filters (average sand grain size 1.35 mm, depth 105 cm and diameter 2.8 m) followed by DMF which consists of 0.55 mm grain size sand for a depth of 40 cm above which is 0.8-1.6 mm particle size anthracite for a depth of 40 cm, bottom layer is 1.35 mm particle size coarse sand for a depth of 30 cm, vessel diameter is 3 meters. Automatic back-washing follows the program: 5 minutes drain, 5 minutes air scouring, 7 minutes backwash, 3 minutes filling and 5 minutes settling time. Programmed back-washing frequency for coarse filters is 1 to 2 days and for DMF every 3 to 5 days. The backwash pump (BWP) has a capacity of 210 m^3/h at 17.5 bar head. The filtrate was dechlorinated by dosing SBS up-stream of CF as shown in Figure 1. The water is then streamed through 5 micron CF unit for fine filtration prior to desalination. The 1st stage desalination is operated at a feed pressure of 60 bar, feed flow 150-160 m^3/hr at ambient temperature (29-33°C). Second stage is operated at feed pressure of 28 bars.

OPERATION TRIALS AND RESULTS

First Trial

Many precautions were taken for this trial. To improve the feed water quality, new filter media were introduced in train 200 (experimental train). Filters were soaked for four hours in 5 ppm chlorine solution then flushed and back-washed. CF was replaced with a new one. Seawater inlet pipe from intake chamber to intake pit was flushed. To avoid bio-growth, intake sump was covered to prevent exposure to sunlight. Bacteriological analysis and collection of operational data were carried out for one month under normally followed operating conditions as mentioned above. Data on pretreated seawater SDI values, coarse filter backwash frequency (FBWF) and cartridge filter replacement frequency are plotted versus operation time (Figure 3). The average SDI values for pretreated seawater ranged from 2.7 to 3.3, FBWF was once per day and the CF replacement frequency was thrice per month. The permeate conductivity from all racks of the two trains was very high >9000 ms/cm, while average DP across membranes was >7 bar. The daily increase in DP was about 0.6 bars. The average permeate flow from individual racks was as low as 3 to 4 m^3/rack. The membranes were in service for about one year and were considered old. Membrane cleaning frequency was within 12-15 days. Membrane chemical cleaning procedure, recommended by DuPont has been followed.

Second trial

In order to achieve a longer residence time for coagulation, the coagulant dosing point for train 200 was shifted about 84 meters up-stream from its previous location as shown in Fig. 2. Sixteen old membranes having permeate conductivity lower than 5000 ms/cm were selected from train 200 and were mounted on rack #1 of train 200 to evaluate their performance and compare the same with former and subsequent results at other trials. The chlorine and SBS dosing locations as well as other operating parameters remained the same as they were during the first trial. The unit was operated for two weeks under these operating conditions. Plant performance data were monitored daily. No significant changes in the values of SDI, FBWF and CF replacement frequency were observed during this trial, their values were almost similar to the first trial (Figure 3). The permeate conductivity and ΔP (old membranes) were very high (same as in the 1st trial). Permeate flow was very poor. The ΔP from old membranes in rack #5 of train 200 were compared to those obtained from the new membranes which were operated during the 3rd and 4th trials as will be addressed in the following sections. Figure 4 shows the ΔP from old and new membranes installed in rack #5 of train 200. The trend in rise of ΔP values across all membranes was very sharp. During this trial, the initial ΔP (old membranes) value of 2.5 bar increased to >7 bar within 7 days, at an approximate average rise of 0.64 bars (9.3 psi) per day.

Third trial

To eliminate fouling potential within the CF as well as in the process piping, two modifications in SBS dosing location were implemented. In the first case, the SBS dosing point, one static mixer and the oxidation reduction potential (ORP) sensor of train 200 were shifted from their original locations, after the dual media filter, to downstream of the CF, as shown in Figure 2. CF was replaced with a new one. In the second case, the SBS dosing point was further moved from earlier location to adjacent at high pressure pump (HPP) and the ORP sensor was placed at a point downstream of HPP (Figure 2). The second SBS dosing point was 13 meters downstream of earlier location. Static mixer was not shifted in the second case, as the HPP was considered a good mixer. Higher than earlier dosing rate of chlorine was maintained to keep enough residual (0.5-0.6 ppm) in the feed stream prior to dechlorination. Consequently, the SBS dosing rate also increased. Location of PE dosing and other operating conditions, in both cases, remained as they were in the second trial. Two racks of train 200 were loaded with 26 new B-10- 6835 membranes (15 in rack #4 and 11 in rack # 5). The rest of the racks (racks #1, #3 and #6) were operated with old membranes, while rack #2 was isolated due to high conductivity of permeate. The unit was operated for four and two weeks, respectably, utilizing the two different SBS dosing locations

When the SBS was dosed at downstream of the CF, the rise in differential pressure across the CF was at a lower rate and consequently, no replacement of CF was required within one month. Prior to this modified SBS dosing, differential pressure of CF exceeded 2 bars within 10 days of operation, which necessitated its replacement (Figure 3). The average SDI values of pretreated seawater ranged from 2.8 to 3.6 and the FBWF was similar to those during the first trial, it was at an average of once per day (Figure 3). The DP of new membranes (two racks in train 200 and one rack in train 100) exceeded 3 bars within 12 days. This was similar to the trend of ΔP rise in old membranes. The initial ΔP of 0.3 bars went up to 3.4 bars within 12 days of operation for the new membranes in rack #5 (Figure 4). This is compared to a rise of 0.2 to 0.3 bars in ΔP per day for all membranes. After the 1st chemical cleaning, ΔP values for membranes in rack #5 decreased to 1.0 bar. Second cleaning was carried out after only 9 days of operation when ΔP went up to 5.1 bars. This suggests higher membrane fouling during the period. After 8 days of the 2nd cleaning there was a shut down for a few hours. Following this, ΔP fell significantly (Figure 4) due most probably to permeate flushing at shut down (SDF). Permeate flow and conductivity from new membranes in racks #4 & #5 of train 200, are plotted versus operation time (Figure 5&6). Permeate flow and permeate conductivity of these membranes were similar in both racks. The permeate conductivity from these membranes remained approximately steady during this trial (Figure 6). They were within the recommended range of 355-486 ms/cm and 318-375 ms/cm in the same order as mentioned above. Permeate conductivities of old membranes were higher than 9000 ms/cm (not shown). Dosing of SBS about 13 meters up-stream of high pressure pump (HPP) or adjacent to it showed similar results.

Fourth trial

This was the main and final trial of the experiment. During this trial the plant was operated without chlorination/dechlorination but coagulant dosing through new location and other operating conditions remained unchanged. Due to severe coarse filter clogging, the plant operation without chlorine was terminated, initially (1997), after 5 days and the plant reverted to chlorination. Later on, it was again implemented for 24 days. Prior to no chlorine operation the intake chamber and intake pit were cleaned, a few backwashes were conducted with chlorinated (0.5 PPM) pretreated seawater as well as without chlorine. Operation of both trains without chlorination was continued for 11 days from 18/7/1998 to 28/7/1998. The rest of the days (19 days) the plant was run with intermittent chlorination rather than without chlorine.

An increased rate of rise in differential pressure across coarse filters of both trains was observed during no chlorine trial. Consequently, an increased FBWF, from 1 to 2 times/two days to twice/day, was required from the 2nd day of starting the no chlorine trial. The FBWF became more severe, about 2 to 3 times/day, on 4th day of the trial. Pretreated seawater SDI values as well as CF replacement frequency of train 200 also went up during this period. The initial average SDI value of 3.4 increased to 5.5. Figure 3 shows the abnormal FBWF and CF replacement frequency values during no chlorine operation. The increased frequency of FBW resulted with an extra demand on filtered water and ultimately remarkable increase in the coarse and dual media filtration rates. On the 5th day both the main and standby coarse filters of train 200 were clogged together and differential pressure limit of 1.2 bars for coarse filters went up to 2.2 bars, instantly. Pretreated seawater feed to HPP was restricted by coarse and dual media filters and suction pressure of HPP was reduced to lower limit of its low-pressure interlock. These circumstances compelled the plant to shutdown and the no chlorine trial operation was discontinued from the same day (5^{th} day). The no chlorine trial was resumed again in *July 1998* for a total of 24 days (in two parts). The FBWF (Figure 3) and ΔP across dual media filters were observed to be within the tolerable limit during this later operation. The FBWF was once per 2 to3 days and backwash for the DMF was needed every 3 to 5 days. This is similar to the case when the plant was operated with chlorination/ dechlorination during the first two trials. The average SDI values were within the range of 3-4. Remarkable variation in the SDI values was observed during the two periods of no chlorine trials. Seawater qualities during the two occasions were different. Differential pressure (ΔP) across membranes, permeate flow and conductivity from membranes of racks #4 and #5 are plotted versus operation time (Figure 4, 5 and 6). The observed average rise in ΔP during the no chlorine trial was less than in earlier trials of operation with chlorine. It was about 1.5 psi/day or less compared to 3 to 5 psi/day during operation with chlorine in earlier trials. The membrane flux decline rate was also very low. On the average, it was about <0.39% per day of the initial permeate flow compared to 0.86% in earlier trials when the plant was operated with chlorine (Figure 5). Permeate conductivity, however, remained steady and was not different from the chlorinated trials (Figure 6).

DISCUSSION

To ensure a better SWRO membrane performance, an efficient feed pretreatment is a precondition. If variability of intake seawater quality is not visualized in feed pretreatment design, frequent membrane fouling could occur. In a conventional coagulation filtration pretreatment system, filtration rate is the prime influencing factor that controls the efficiency of media filters. Al-Birk plant data reveal that the flow velocity (0.4 m/min) along coarse filters seems to be higher than the values suggested in the design and reported in literature. No other SWCC SWRO plant has such a high filtration rate. Operation at the highest rates (0.122 m/min to 0.244 m/min) has been reported on higher quality feed water which is yet far less than the case of Al-Birk plant (0.4 m/min). Furthermore, the velocity of filtration increases yet further during filling of backwash tank (vol.=130 m^3). Filling of the backwash tank requires increased flow rate by 30 - 40 m^3/h in addition to normal feed flow through the coarse and DMF filter. The increased flow velocities enhance the stripping of suspended particles through media filter beds into the CF and onto the SWRO membranes. This happens after every backwash, hence initiating differential pressure build-up across CF and membranes. So, the higher filtration rate calls for an investigation to rectify present situation. The removal of suspended solids by filtration is a complex process involving a number of phenomena. Attempts to develop theories, which quantitatively predict solids removal performance with sufficient precision and versatility to be of use in practical filter design, have met relatively little success. Current practice is directed towards using a greater volume of the filter bed to maximize run lengths and minimize wash-water usage. This practice may be applied on the media filtration system at Al-Birk plant by introducing more filter bed. Effective media size, media depth and optimum filter bed are to be investigated to improve present filtration efficiency.

In addition to the filter bed deficiency, the plant has no intermediate storage facility (clear well) for pretreated seawater to feed the plant during backwashing. Normalization of pretreatment (i.e., effectiveness of coarse and media filters) requires draining of "First Filtrate" immediately after backwash until pretreated seawater SDI value comes down to ≤3. If FBWF were to increase abnormally, as was observed during initial stage of the no-chlorine operation, the plant would require to be fed from a clear well. Otherwise, reduction of plant availability has to be accepted. More filter bed and pretreated seawater intermediate storage facility appear to be essential requirements for the plant at present stage. An abrupt rise in ΔP across CF was also known to occur after each backwash due to carry over of silt materials on to the CF. This could be negated by dumping the 'first filtrate', as mentioned above. High frequency of CF replacement affected the plant availability, as there was no standby CF housing in the plant. Standby CF housing can eliminate plant shutdown during CF replacement period. Availability of all these components should ensure high plant availability and steady operation. In the present situation, operating the plant at low capacity during rough sea conditions may alleviate frequent clogging of pretreatment filters. But, high water demand does not allow the plant to operate at low capacity. A backwash strategy emphasizing dumping first filtrate has already been suggested. It may be a temporary solution to frequent clogging of CF at least during calm sea conditions. Deficiencies in backwash system and flow rates also existed in the plant. The expected flow at 90% efficiency of BWP is about 189 m^3/h. It may be noted here that the feed flow through media filter during backwash tank filling is about 180 to 190 m^3/h, which is almost similar to the backwash flow rate value. Normal backwash flow is observed to be about 3 times higher than feed flow. Such low backwash flow ratio is not observed in any other SWCC SWRO plants. In addition to that there is no inter connection between the two backwash tanks of individual trains. It could help to backwash one train with water from other train at emergencies as was experienced during the no chlorine operation. *The above discussion reveals that deficiencies of filter bed, filtrate storage facility, standby CF housing and poor backwash facility are resulting in low plant availability and high membrane fouling.*

The increase in FBWF during the no-chlorine phase of operation is not usual. No other SWCC SWRO plant reported this type of abnormal filter clogging. During this short period of operation without chlorine, the physical appearance of seawater was muddy with fishy smell and contained plenty of living organisms (copepods) [18]. Investigation on physical character of the intake seawater showed that the SDI values were much above the measuring limit. Within 3-5 minuets, the 0.45m SDI paper was almost blocked. The SDI value of pretreated seawater was also high (at the rate of 4.6). However, during the second no-chlorine operation, the SDI values were lower than earlier readings. This confirms the variable water quality of Al-Birk plant intake. Therefore, physical, chemical and biological analysis of seawater from the vicinity around the plant intake and its neighboring area are required prior to any attempt of major modification either of the intake or pretreatment system. Clogging of coarse filters is a pretreatment problem and it is to be solved by improving pretreatment efficiency. If larger clogging particles could be screened out ahead of coarse filter it may constitute a remedy for filter clogging. To improve intake water quality, *alternate intakes* [19] are suggested to ensure better plant performance. *Relocation of intake chamber to a better quality water site could be an alternate to present water source.*

It is evident from the analysis of experimental data that introduction of improved operation techniques are expected to eliminate operation problems to some extent. From plant design it appears that the maximum water velocity was about 11.3 m/min in the 30" diameter and about 300 meters long intake pipe from the intake chamber to the intake pit. The minimum total residence time for chlorine along the 30" intake pipe was about 26.5 minutes when the plant was operated with a flow rate of about 300 m^3/h. Considering the residence volume of intake pit (about 420 m^3 at high tide) the total residence time of chlorine in seawater up to the suction of the seawater pump is more than 90 minutes. The residence time of chlorine, if it is neutralized at its regular location up-stream of CF, is only less than two minutes longer than if it is neutralized at the seawater pump. This added time of chlorine residence is felt to be of a negligible effect on disinfection efficacy because the intake water had already been in contact with chlorine for 90 minutes. Therefore, the dechlorination point could be shifted away "up-stream" from existing location (up-stream of CF). It was seen from bacteriological study [18] that shifting the SBS-dosing point up-stream will reduce biofouling potential of SWRO feed water. S*o, an alternate SBS dosing location at upstream of coarse filters is recommended.*

Compared to initial flow values the respective rises in permeate flows after cleaning were only 3.5 and 5.5%. This poor restoration percentage of membrane performance suggests the *inefficiency of chemical cleaning* agents and the chemical cleaning procedure used in this plant. Alternate coagulants improved feed water quality in some sites [7]. Inorganic coagulant could be an alternative to present organic coagulant. There are evidences that organic part of coagulants enhance biofouling. Also no effect of existing coagulant has been observed on pretreated water quality. *So, the use of an alternate coagulant is suggested for Al-Birk.* The ΔP build-up across membranes showed a general trend of decrease after shutdowns. This is attributed to flushing by stored product water. This observed behavior of membranes after several shut downs are shown by arrow marks in Figure 4. *Membrane flushing could be practiced on a regular frequency (e.g. 15-20 min/day) to see if it would produce a continuous decline in* ΔP *build-up across the membranes.* A number of SWRO plants throughout the world have successfully increased product water flushing to minimize membrane chemical cleaning and enhanced plant availability [14]. Should the plant continue operation without chlorine some sorts of disinfectant shock dosing treatment may be recommended to keep the system piping clean from biological fouling. Iodine shock treatment as an effective biological control agent has been applied in a forest ranger water treatment station in U.S.A. Iodine is also reported to be successful in reducing chemical cleaning frequency by about six times, from every 15 days to 90 days, in a SWRO plant in UAE [14].

The discussion points to the need for further investigation of the operational problems of the Al-Birk plant. As such an investigation will place a burden on the plant to meet local consumption demands, it is preferred to carry out all recommended trials in a pilot SWRO unit that mimic the main plant.

CONCLUSIONS

1. In several trials, it appeared that successful continuous operation of Al-Birk plant without Cl_2 depends on sea water conditions (source water quality) at Al-Birk.

2. Operation without Cl_2 improved membrane performance but increased SDI value, media filter clogging and CF replacement. However, operation period was short.

3. Should conditions be smoother for a longer duration of operation without chlorine, smooth operation of the plant could be obtained through gradual refinement and process manipulations.

4. Modified PE dosing did not improve feed water quality (SDI). Dosing of SBS at modified location decreased CF replacement frequency but did not improve membrane performance.

5. Process evaluation reveals some deficiencies in pretreatment.

6. Seawater SDI values was very high and variable.

7. Present coagulant seemed to be ineffective.

8. Membrane performance restoration by chemical cleaning was not adequate.

RECOMMENDATIONS

1. *Site evaluation* : An evaluation of the intake site seawater quality around the vicinity of the plant is required to take decision on relocation or modification of the present intake.

2. *Alternate intake* : The probability of a beach-well as an alternate intake needs to be investigated.

3. *Upgrading of present pretreatment* : (a) Introduction of extra media bed and proper media size, (b) Intermediate pretreated seawater storage facility (i.e., clear well), (c) Standby CF housing, (d) Extra SDI monitoring facility.

4. *Modification of processing techniques* : (a) Low capacity operation during rough sea condition. (b) Intermittent permeates flushing operation. (c.) Occasional shock dosing of disinfectant. (d) Improved back-washing strategies.

5. *Trial of modified operation modes* : (a) Intermittent chlorination, (b) Dechlorination at up-stream of coarse filters, (c.) Trial of coagulant other than PE or even without coagulant, (d) Operation without Cl_2 during favorable weather months (Nov. -April). (e) Trial with coagulant aid.

6. *Alternate membranes* : Use of organic repulsive and fouling resistant membrane.

7. *Mobile pilot plant* : May be provided for future test.

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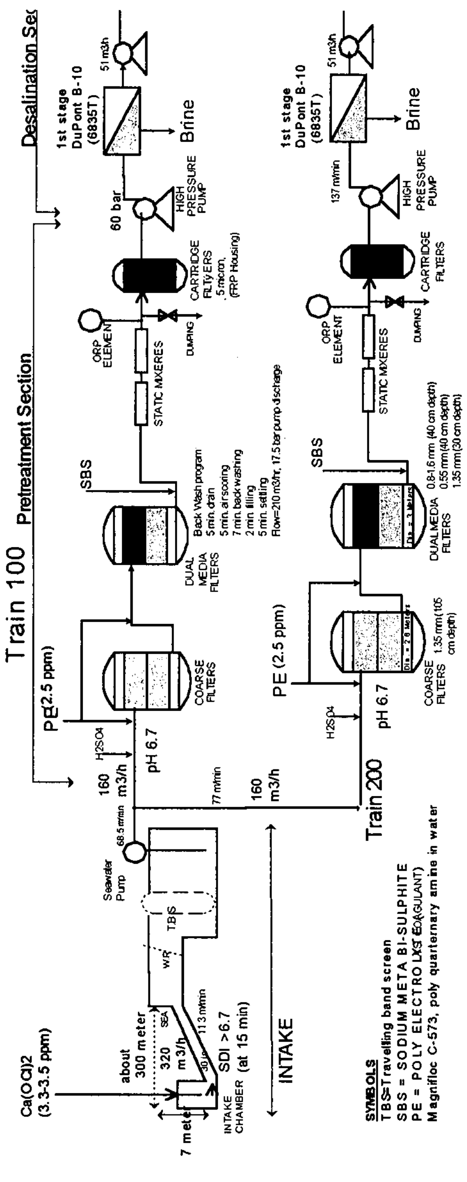

Figure 1: Schematic Flow Diagram of Al-Birk SWRO Plant Showing the two Trains, Normal Coagulant (PE) and SBS Dosing Points and dosing rates.

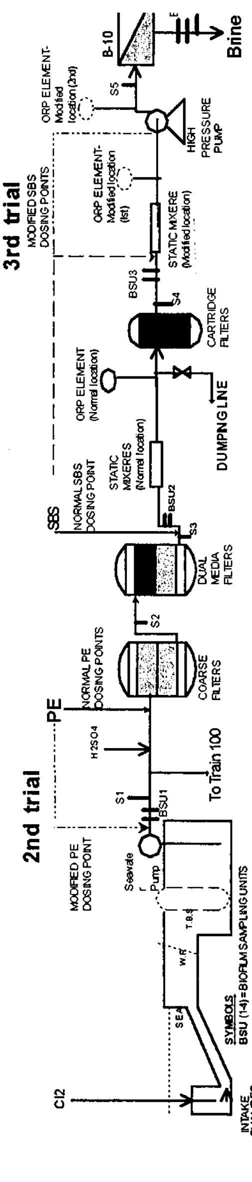

Figure 2 : Schematic Flow Diagram of Al-Birk SWRO Plant,Train 200, Showing Sampling Points (S1-6), Locati Bio-film Sampling Units (BSU 1-4), Normal and modified Coagulant (PE) and SBS Dosing Points.

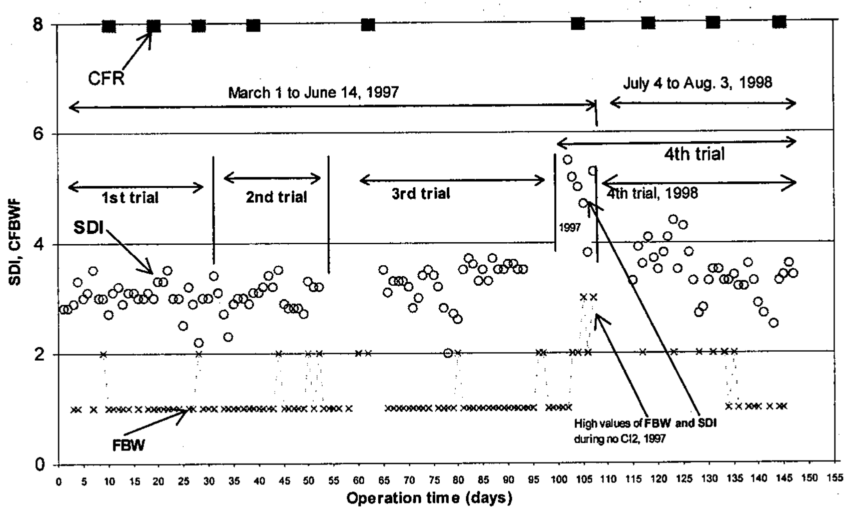

Figure 3: Pretreated Seawater SDI Values, Coarse Filter Backwash Frequency (FBWF) and Cartridge Filter Replacement Frequency (CFRF) versus Operation Time.(Al-Birk SWRO Plant).

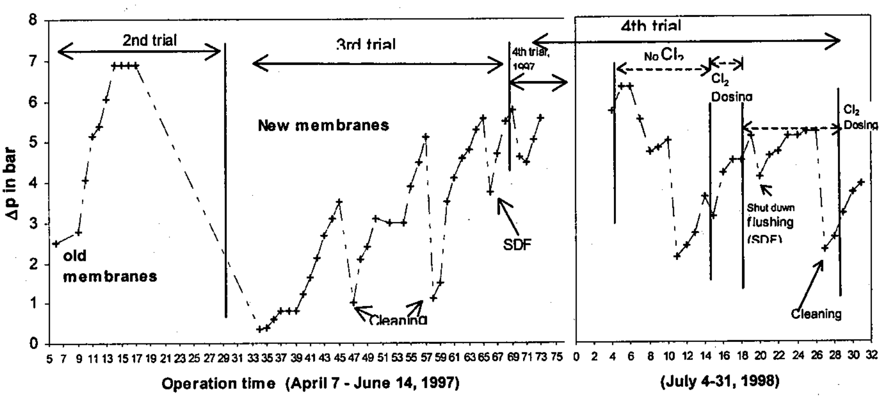

Figure 4 : Δ p Across Old and New DuPont B-10 Membranes, Rack #5 of Train 200, Al-Birk SWRO Plant versus Operation Time.

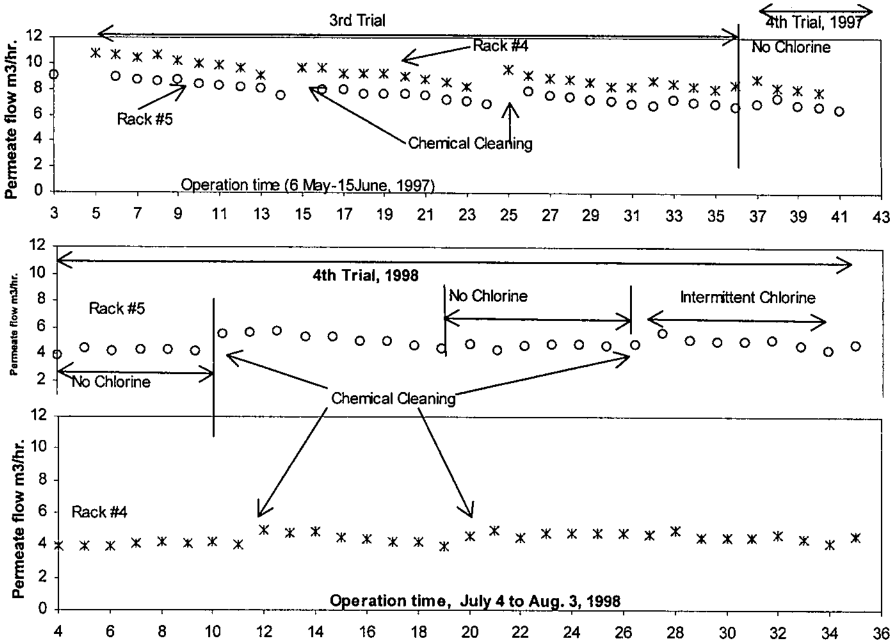

Figure 5 : Normalized Permeate Flow from Membrans of Racks #4 and #5, Train 200, Al-Birk Plant versus Operation Time.

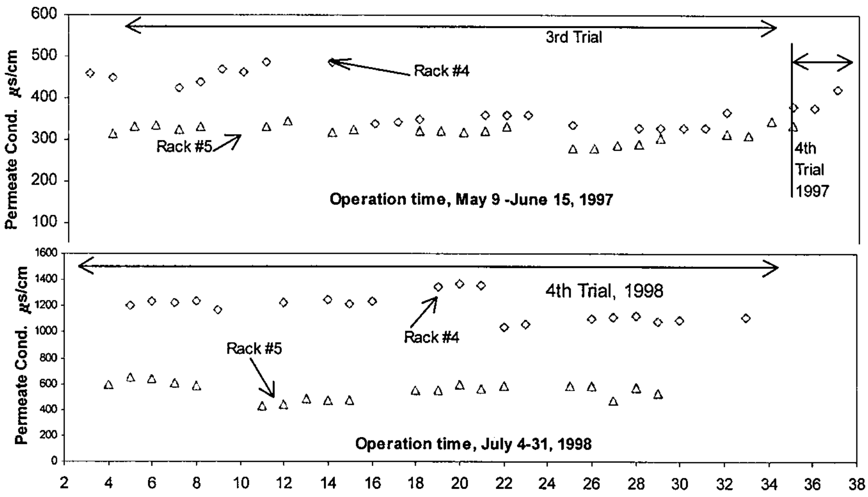

Figure 6: Permeate Conductivity from Membranees of racks #4 and #5, Train 200, Al-Birk SWRO Plant versus Operation Time

Plates – The Next Breakthrough in THERMAL Desalination

Mr. Carlos Legorreta and Mr. Steen Hinge
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PLATES – THE NEXT BREAKTHROUGH IN THERMAL DESALINATION

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ABSTRACT

Desalination systems employing the distillation process depended on basic shell and tube heat transfer exchanger technology for long time. This basic form of heat exchanger has been acceptable in the past but as the economics of systems becomes more important, and as waters for desalination become more diverse, then the total cost aspect of the project becomes essential. An alternative heat transfer technology is now becoming more dominant in the desalination market – plate heat exchangers.

The plate type desalination systems have been in operation since 1967 and some 70% of the world's ships and offshore installations are using plate technology due to the advantages inherent in the system design. The plate concept applied in Alfa Laval Water Technologies' Multi-effect plate (MEP), Multi-effect thermo vapour compression (TVC) and Vacuum vapour compression (VVC) has distinct advantages over earlier technologies and offers owners and operators significant total cost savings.

The high heat transfer coefficient achievable in the plate concept leads to relatively low heat transfer areas. This, along with the inherent low hold up volume leads to an extremely compact design giving a small footprint and corresponding to low civil costs. For most applications Alfa Laval applies Titanium plates inside a Duplex Stainless steel vessel. Neither the Alfa Laval plate nor tubular units are designed to operate with waters containing high calcium sulphate. All heat transfer surfaces suffer if subjected to high sulphates, but the significant difference between the plate and the tube is that the plate can be completely cleaned on both sides. In this manner the heat transfer area always remains totally effective, removing any need to incorporate redundancy in design.

Also, the plate concept configuration can be configured to allow future addition of the heat transfer surface. This makes it possible to plan for future plant capacity expansion with minimal investment today. While membrane systems may be able to allow for the addition of extra membrane for planned increases in capacity and so does the plate system. This has not before been possible with any thermal desalination process, thereby the new plate configuration offers a very high design flexibility.

INTRODUCTION

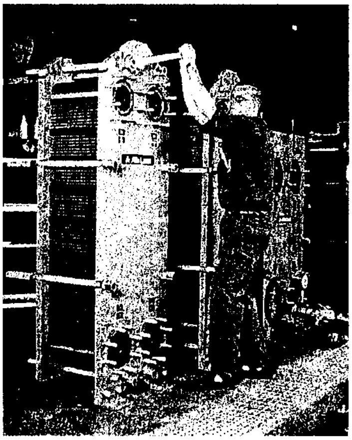
It is recognized that **Plate Heat Exchangers (PHE)** are optimal solutions for traditional liquid/liquid heat exchangers.

Still some applications with high temperatures and/or high pressures utilize shell and tube heat exchangers, but apart from that Plate Heat Exchangers are by far the primary type of heat exchangers used today.

The Plate Heat Exchanger design provides certain advantages over the traditional shell and tube heat exchangers, amongst others :

- Typically the heat transfer coefficients in plate heat exchangers are 2 to 3 times those obtainable in traditional solutions, achieved by possibility for optimizing design and characteristics of the plates

- Low material consumption, the corrugated structure of the plates and the densely distributed supporting points require only limited thickness of plates with savings in material and less resistance of heat conduction. Typically the consumption of material to achieve the same heat transfer area is reduced by 50 % compared to traditional shell and tube solutions



- The standard plate heat exchanger concept does not require machining, welding or other forms of crafting apart from the pressing; thus it is possible to utilize all sorts of metallic materials for plate production

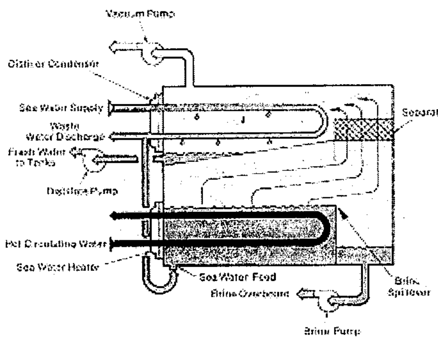
- The high grade of flexibility provided by the plate heat exchanger insures the satisfaction of the exact design requirements of a certain heat exchange application. In traditional shell and tube designs, one is forced into a selection of design. This design will never be the optimal compared to the requirement of the application. In most cases this gives an excessive use of heat transfer surface and a heat exchanger which does not provide the best design in terms of performance and ability to be self-cleaning.

- The plate heat exchanger is the only heat exchanger that can be fully disassembled and cleaned. Traditional heat exchangers can be cleaned by Cleaning in Place (CIP) only. If fouling and/or scaling occur and cannot be removed by chemical agents, it has to be removed mechanically on the inside of the tubes; should it happen on the outside of the tubes, the only solution is a complete re-tubing of the heat exchanger.

TRADITIONAL SOLUTIONS FOR DISTILLATION PROCESSES

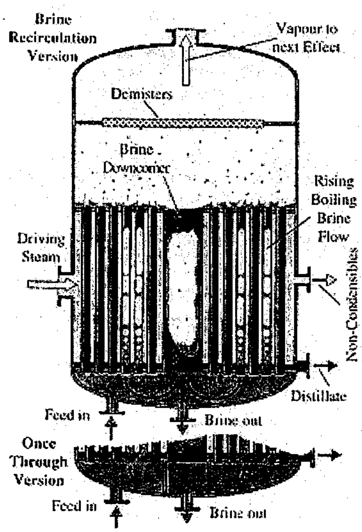
Submerged Tube

In submerged tube evaporators, tube bundles are submerged into the brine, thus the name. The heating medium, either condensing steam or hot water, flows inside the tube, thus making the brine evaporate.



The submerged tube is rather sensitive to scaling and requires low operating temperatures in order to avoid precipitation of calcium carbonate, calcium sulfate and magnesium hydroxide. Due to this fact the submerged tube configuration is essentially not used today.

Vertical Tube Rising Film, VTRF

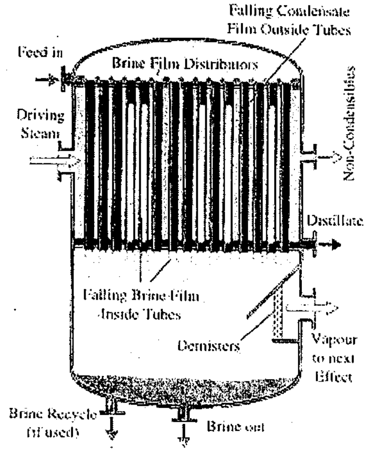


In the rising film system the brine flows up-wards inside the tubes; the heating medium, either condensing steam or hot water, flows on the outer surface of the tubes. This causes the brine inside the tubes to evaporate, vapour is released and flows upwards in the tube in plug flow. This causes a thin film of brine to be established and constantly be renewed on the tube wall.

In the rising film the boiling takes place inside the tubes; thus, if scaling occurs it is relatively easy removed. Though, if calcium sulfate scaling occurs, the tubes have to be cleaned mechanically by drilling or brushing the layer of scaling.

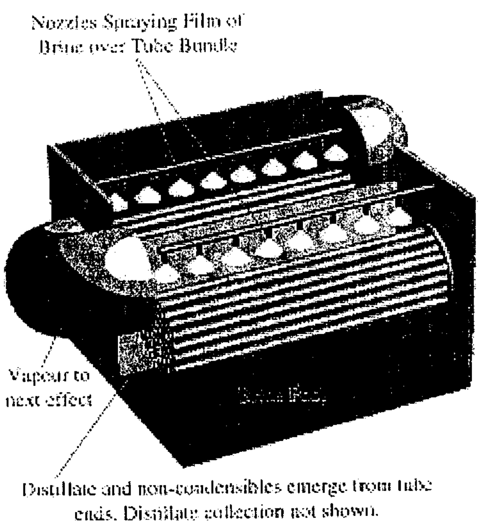
Vertical Tube Falling Film, VTFF

In the falling film system the brine flows in a falling film down and normally inside the tubes. The heating medium, - either steam or hot water - flows on the outside of the tubes which causes the brine to boil. The generated vapour flows together with the surplus brine downwards in the tubes.
The falling film has to have even distribution of brine around the circumference of all the tubes and along the full length; otherwise, dry spots and consequently scaling can occur. This distribution is sometimes achieved by fitting in distribution caps on the top of each tube; this, however, is sensitive to blockage.



Horizontal Tube Falling Film, HTFF

The Horizontal Tube Falling Film is the most popular system used for traditional shell and tube multi-effect and vapour compression desalination plants.



The falling spray film is established by spraying brine through nozzles or spray plates above a horizontal tube bundle, and a film is established around the outside circumference of the top tube rows by the spray nozzles, and on the lower tube rows by dripping from the upper tube rows.

The heating medium, normally steam, flows on the inside of the tube bundle. The steam condenses on the inside of the tube and its latent heat is transferred to the brine film on the outside. The heat transfer causes a portion of the brine film to evaporate. This vapour condensed in subsequent lower temperature effects produces the distilled water.

PAST PLATE CONCEPTS

Bavex system

The Bavex system offered by the German company Deutsche Verfahrens Technik (DVT) is, in fact, not a plate concept but merely a different configuration of the horizontal tube falling film. This hybrid configuration makes a sort of 'tube bundle' in which the heating steam flows and condenses. On the outside of this 'tube bundle' the brine distributed by spray nozzles flows vertically in a falling film.

The brine film in the lower areas of the evaporator surface is not dependent on dripping, as is the case with the traditional horizontal tube falling film concept, but rather by flowing downwards on the heat transfer surface.
The system essentially uses corrugated plate surfaces and offered the potential of better control of brine wetting by removing the dripping from tube row to tube row. However, initial distribution of the seawater to the top of the plate is problematic.

Rosenblad Dimpled Plate system

In the Rosenblad dimpled plate system, the heat transfer surface is established by spot welding together two plates with a "quilted" contour. Thus a heat exchange channel is established between the plates welded together and on the outside of the plates.

A brine falling film is established on the outside of the plate cassette, and steam flows and condenses on the inside of the plate cassette; thus some of the brine evaporates.

Disadvantages

The above concepts have not been successfully applied for water desalination because in both cases, when scaling cannot be removed by chemical cleaning in place, the heat exchange surface has had to be removed and replaced. This disadvantage has created reluctance by the operators for its acceptance. There have been other attempts to make plate concepts for desalination and/ or concentration of liquid waste streams. However, they have not yet proven performance in the market.

New Plate Concept For Evaporative Processes

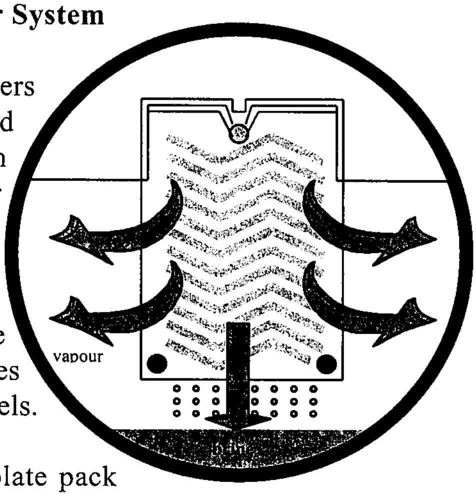
As inventor and technological leader of Plate Heat Exchangers, Alfa Laval puts continuous efforts and resources into research and development of new applications where the advantages of the Plate Heat Exchanger concept can be provided.

Desalination has been an important area for Alfa Laval. The first commercial desalination plant from Alfa Laval with plate heat exchangers was introduced for Marine and Offshore applications in 1967. Since then the concept has taken over the market and is by far the dominant solution today.

With recent R&D activities Alfa Laval has developed a plate concept which also can be used for larger capacity thermally driven desalination plants of the Vacuum Vapour Compression (**VVC**), Multi-Effect (**MEP**) and Thermo Vapour Compression (**TVC**) types.

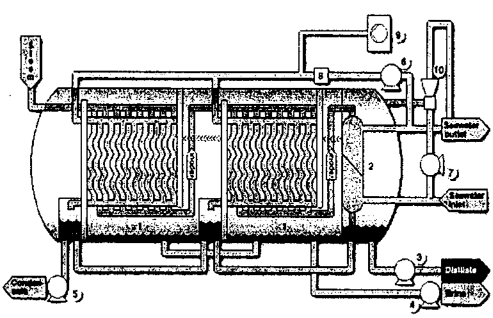
The design of the plate, the arrangement of fluid distribution devices and flow control corrugations represent a significant investment in proprietary technology for the benefit of the customers. For this reason detailed images of the plates cannot be included in this text.

The Alfa Laval Plate Evaporator System



The evaporator plate heat exchangers consist of a number of corrugated titanium plates, which have been especially developed for desalination. All the plates are similar; however, there are two gasket configurations, one for plates forming the evaporator plate channels and another one for plates forming the condensing plate channels.

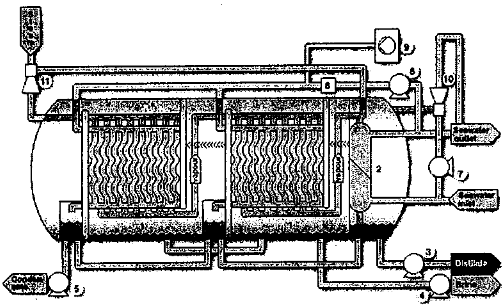
The plates are assembled into a plate pack having a support system inside the vessel. Pressure plates and tightening bolts are used for pressing the plate pack together as in a traditional plate heat exchanger.



Multi-effects System

The brine to be desalinated is introduced into the plate pack and distributed evenly in a falling film on the plate surface in each second channel created by the plate assembly.

On the opposite side, the condensing side, the steam flows into the condensing channels of the plate pack. By giving its latent



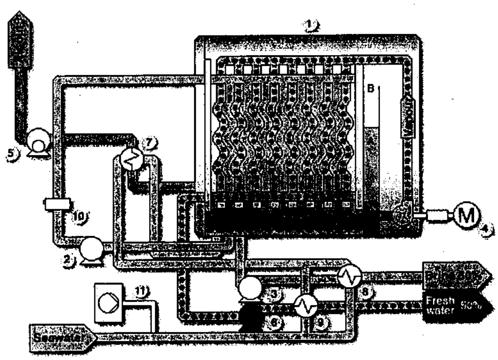
Thermo-compression System

heat of condensation, it causes some of the brine to evaporate. The generated steam and the surplus brine leave the evaporator channels of the plate pack into the evaporator compartment of the desalination plant vessel.

The Alfa Laval plate concept is designed specially for Multi-Effect, Thermo-Vapour Compression and Mechanical Vapour Compression plants.

The special plate design made by Alfa Laval for desalination has basic differences from the typical liquid/liquid plate heat exchanger applications.

This concept has been specifically designed for a two-phase flow operation on both sides of the plate; however, one side is for evaporation and the other for condensation.



Mechanical Vapour Compression System

Plates geometry, size and effective heat transfer area are designed to be integrated in modular plant sizes to meet production capacities from 1 to 10,000 m^3/day per unit or even larger. Both Multi-Effect and Mechanical Vapour Compression units commonly share vessel geometry and dimensions. This allows modular and cost effective manufacturing, resulting in competitive market prices.

Plate production is achieved by capital- intensive, sophisticated metal pressing technology, which secures unmatched quality in terms of plate surfaces, thickness, pattern, groove depths and cutting process.

Titanium

The plates in the Alfa Laval systems are made of pure Grade 1 titanium. This material is used in a cost-effective manner because of the relatively low requirement of material in the plate concept.

Titanium is recognized as the best material for corrosion resistance in seawater environments, both at low and elevated temperatures. In fact, the resistance of titanium is equal to that of platinum.

Gaskets

Gaskets made of high-quality Nitrile do the sealing between the plates. The gaskets are fastened to the plate by gluing.
The Nitrile gasket material is selected amongst the numerous variations of Nitrile to sustain a long lifetime at the specific conditions of the evaporative process.

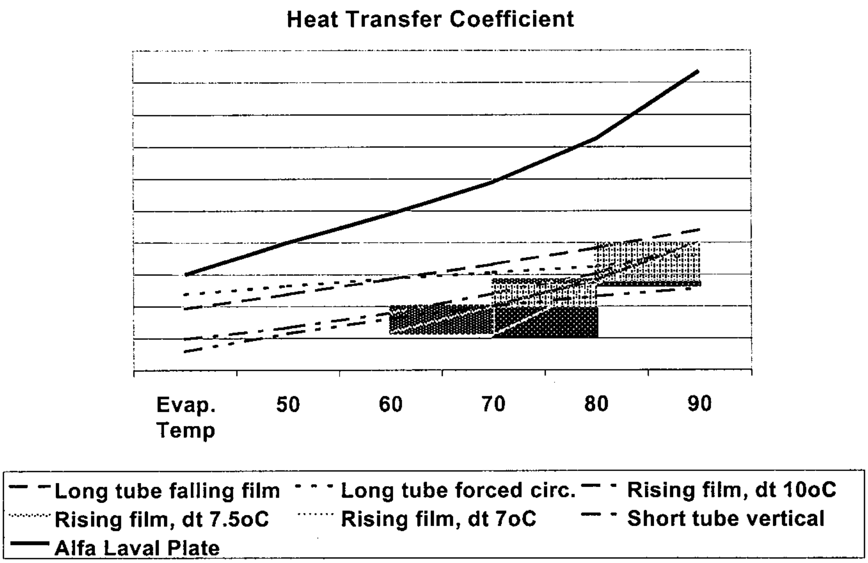
While gaskets can last indefinitely in the air and light-free environment inside the vessel, it is recommended they be changed after 10 years because of natural decay of compressibility; however, this point has never been a major maintenance problem. Change of gaskets can readily be done on site.

COMPARING ALFA LAVAL PLATES WITH TUBES IN WATER DESALINATION

Heat Transfer Efficiency

The thermal efficiency of the Alfa Laval plate concept by far out-performs the traditional shall and tube distillation plants.

The below curve shows a qualitative comparison of the Overall Heat Transfer Coefficients



(OHTC) achieved by the Alfa Laval plate compared to various traditional systems in the market. The curve shows the values at various evaporation temperatures.

As can be seen, the heat transfer coefficient is much higher for the Alfa Laval plate compared to any of the traditional systems available in the market. This is achieved by the particular controlled conditions of velocity and film thickness. Further, the lower material thickness used in the plate concept enhances the performance.

Materials

The main issues to consider in the selection of materials for a desalination plant are cost and the materials resistance to corrosion in the very aggressive seawater environment. Another parameter to consider for the heat transfer surface is, of course, the thermal conductivity, which together with the thickness of the material gives the thermal resistance of conduction.

The standard shell and tube systems in the market today mostly use Aluminum-brass or Copper Nickel as tube material. Some manufacturers utilize even lower grade materials such as Aluminum.

Substitution of MSF Desalting Plants with Absorption Air Conditioning in Co Generation Power Plants in Gulf Area

M. A. Darwish

SUBSTITUTION OF MSF DESALTING PLANTS WITH ABSORPTION AIR CONDITIONING IN CO-GENERATION POWER PLANTS IN GULF AREA

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ABSTRACT

INTRODUCTION

The hot climate in most Arabian Gulf countries AGC makes cooling air conditioning A/C a necessity during long periods of the year (e.g. at least 7 months in Kuwait and 9 months in Jeddah, Saudi Arabia). In Kuwait, one of the AGC, the average maximum temperature in summer is 48°C, and A/C systems consume more than 75% of their produced electric power. It increased from 5100 million kWh in 1975 to 31,576 million kWh in 1999 due to increase of population and standard of living, and generous government subsidizing cost (about 70%). Since electric load follows power consumed by A/C machines (or ambient temperature), vast variation in load is observed. In 1999, the maximum load reached 6160 MW while minimum load was 1650 MW when A/C is not needed. See Ministry of Electricity and water (MEW) statistical book on water and electricity productions in Kuwait (2000).

Also Kuwait, like most Gulf countries, depends on desalting seawater to satisfy fresh water needs by using Multi Stage Flash (MSF) desalting system. The MSF units in Kuwait consume on the average 300 kJ thermal energy in the form of steam at moderate pressures (2-3 bars) and 16 kJ pumping energy to produce one liter of desalted water. This is huge amount of energy if one thinks about 78797 million imperial gallons (358.53 millions m3) desalted water produced in 1999. This production consumes 107.56 millions GJ thermal energy, beside 1809 million kWh pumping energy.

It is wasteful, from thermodynamics point of view, to use fuel to generate low availability steam (at low temperatures and pressures) required by the MSF units, compared to steam generated for power plants to produce work. So co-generation power-desalting plants (CPDP) are used to produce electric power and process heat for desalting. In these plants, steam is generated and fed to turbines at high pressure (about 140-160 bars) and temperature (about 538-560°C). The steam expands in the turbine and produces work before its extraction (partially or totally) to the MSF units at relatively low pressure (about 2 bars). Good percentage of fuel energy is saved by the use of CPDP compared to two separate plants, one for desalting and one for power. This can be shown by an example from one of Kuwaiti plants:

1. The plant consumes 779 MW fuel energy when it produces 300 MW electric power only

2. It consumes 869 MW fuel energy when it produces same 300 MW electric power, and 196 MW process heat for desalting. This shows that less than 100 MW fuel energy consumed by steam generator to produce 196 MW process heat for desalting.

The domestic daily desalted water consumption per capita increased from 30.6 IG (imperial gallons) in 1975 to 107.3 IG (488.2 l/d) in 1999. The cost of the energy used to produce the 383.2 million m3 of desalted water produced in 1999 can be estimated:

1. The thermal energy consumed to produce the 383.2 millions m3 desalted water is 115 millions GJ (based on 300kJ/kg distillate). In fact little more than half of this amount, say 52%, i.e. about 59.8 millions GJ is supplied at the steam generators when co-generation plants are used. If $30 is the cost of one barrel of oil that producing 6 GJ, then the fuel energy cost is $ 299 millions.

2. The cost of 1810 million kWh mechanical energy used for pumping based on 16 kJ/kg distillate) is $131.25 millions (the actual production cost of electric power is $0.07/kWh).

This shows that the energy cost only to produce one m3 is $1.123. This does not include the operating and capital costs to be added to get the cost of desalted water production.

The water and power industries are concerned with the high cost of their products due to the encountered de-efficiencies resulted from:

1. Continuous increase in demand for both desalted water and electric power, and consequently continuous need for building new power and desalting plants,

2. Large variations on electric power demands due large variation of the power required by A/C units.

3. Operation of power plants at low capacity factors most of the year, due to part load operation except few hours at peak demands in summer. This means in-efficient use of fuel energy.

4. Continuation of desalting seawater by the in-efficient MSF desalting method combined with power plant operation as compared with the more energy efficient Reverse Osmosis desalting system. The Reverse Osmosis desalting system consumes only mechanical (pumping) energy. The consumed energy is usually less than guaranteed 7.5 kWh/m3 with the use of energy recovery. This is in the range of 1/3 the equivalent mechanical energy consumed by the MSF desalting system which amounts to 22 kWh/m3.

5. Inefficient use of electric energy and desalted water by the population and the real needs to rationalize their use.

ANOTHER APPLICATION FOR PROCESS HEAT

The in-efficient energy method of MSF seawater desalting is losing grounds to the more competitive and energy efficient Reverse Osmosis (RO) desalting system. Sooner or later, RO desalting plants are to be installed either as new plants or substitute to the present MSF plants. About 30% of desalted water are produced by RO system in Saudi Arabia (the largest producer of the desalted water in the world) after 100% production by the MSF desalting system in the last decades.

In a country like Kuwait, all power plants are designed as co-generation plants to produce both electric power and process heat for desalting plants. When RO desalting system substitutes the MSF system in future, the process heat from the co-generation plants would be banned. This reduces the efficiency of these plants when it works as single purpose power plants. This is one of the reasons delaying the adoption of the more efficient RO seawater desalting system.

For the existing co-generation plants to work efficiently, production of electric power and process heat should continue. The process heat can be used for other purposes, other than desalting. One good usage of this process heat is to drive Absorption Water Chillers for district Air Conditioning needed in Kuwait. Fortunately, absorption water chiller machines, (like Water-Lithium Bromide type), are driven by steam or hot water in the temperature range of 100-130°C, similar to conditions required by the MSF desalting units. A schematic flow sheet of an existing co-generation power and process heat with conditions used in Kuwait is given in Figures 1a and 1b.

The use of this plant to produce electric power and chilled water for summer A/C can result in better utilization of fuel energy, and help in changing the MSF desalting method to the more efficient RO system. It leads also to better usage of the available equipment. The benefits of using this approach as compared to the use of conventional motor-driven mechanical vapor refrigeration MVC machines and producing power to drive them are illustrated.

CO-GENERATION POWER-PROCESS HEAT STEAM PLANTS

It is more efficient for co-generation plant, as the one shown in Figure 1, to produce both power and process heat as compared to power production only. The plant maximum power output is 300 MW with or without process heat production. When the plant produces 300 MW electric power and 196 MW process heat (QP), the steam generator produces 298 kg/s of steam to be fed to the high-pressure (HP) turbine. When 300 MW power is only produced, only 261 kg/s steam is generated. This keeps about 20% of the steam generator capacity unutilized and working at part load with lower efficiency.

The reduction of the HP turbine flow rate reduces its power output, but the losses kept the same, and this decreases its efficiency. Adiabatic efficiencies of the HP turbine with and without process heat production are 0.8712 and 0.8432 respectively. Also when both electric and process heat are produced, the steam flow rate leaving the low-pressure (LP) turbine is 119.8 kg/s, compared to 170.5 kg/s for power production only. This decreases the leaving (kinetic) energy losses from the LP turbine about 30%. Moreover, the advantage of getting process heat by steam extraction while consuming about half of this amount as fuel energy to steam generator is lost when power only is produced. So, it is much better to continue the production of both electric power and process heat by the plant and finding another application for this process heat.

This study suggests transfer of the process heat, in the form of hot water from the existing co-generation plants, to Water-Lithium Bromide Absorption chillers to generate chilled water for air conditioning. The absorption machines can be installed in building to be cooled. Hot water can also be supplied to the air handling units when heat is required in winter or for other domestic use. In case of dismantling an MSF desalting unit, its brine heater can serve as a heat exchanger to produce hot water from the extracted steam. There are two brine heaters combined with each 300 MW turbine. Each brine heater is designed to heat 3450 kg/s from 83 to 90°C. It can operate to heat 1700 kg/s from 116 to 130 °C to suit the conditions required by absorption machines as illustrated in the next section.

ABSORBTION WATER CHILLERS

The absorption refrigeration cycle, sometimes called thermal vapor compression (TVC) cycle, see Figure 2, is similar to mechanical vapor compression (MVC) cycle. Both cycles have three main components, namely: evaporator, condenser, and expansion device. In both systems, vapor is generated in the evaporator at low pressure by extracting heat from the medium to be cooled. This vapor should be moved, by raising its pressure, to the condenser operating at higher pressure. They differ only in the way this vapor is transferred from the evaporator to the condenser. In MVC cycle, a mechanical compressor is used. In, TVC cycle, vapor leaving the evaporator is absorbed first by strong absorbent that has high affinity to the vapor. The resultant solution is pumped as liquid to a generator where it is heated to drive the refrigerant as a vapor to the condenser. An example of absorption refrigeration machine is the Water–Lithium Bromide machine producing chilled water at 6.7 °C shown in Figure 2. The conditions around the cycle are given in the figure, Jones (1980). Water, the refrigerant, is evaporated at say 4.4°C and low absolute pressure in the evaporator, i.e. high vacuum. The vapor leaving the evaporator is absorbed into the absorber (operating at 40.6°C) by strong Lithium Bromide Li-Br solution of high affinity to water vapor. Dilute (weak) Li-Br. Solution is resulted due to absorption. The vapor liquefaction into the solution releases its latent heat and this heat should be rejected from the absorber. In another words, the absorber needs cooling by water. As shown in the figure, both evaporator and absorber are equipped with pumps to re-circulate and spray the refrigerant water and the strong solution on the heat transfer tube bundles in the evaporator and absorber respectively to enhance the heat transfer processes. The weak solution is pumped (needs little amount of work compared to MVC) to the generator, where heat is added to drive off much the water from of the solution as vapor to the condenser. The vapor is condensed and expanded to the evaporator through an aperture. The cycle is completed by the return of the water vapor from the evaporator, and the strong solution from the generator to the absorber. Heat, main energy input to the cycle, is supplied to the generator by steam at 1.841 bar or hot water leaving at 117.8°C to heat the dilute solution kept at 104.4°C and 101.4 bar. Heat exchanger is used to heat the weak solution going to the generator and cool the strong solution returning to the absorber. In the example given the machine consumes about 0.7 kg of steam / kW refrigeration at full load or 1.54 kJ of heat/ kJ of refrigeration. This gives coefficient of performance, COP, (refrigeration energy/ supplied energy) around 0.65. The COP of the absorption machine is very low compared with the COP of MVC refrigeration (in the range of 3-4). This is about 6 times as much as the COP of the absorption machine. However, this does not mean that MVC machines are 6 times more efficient than the TVC machines. The reasons are outlined in the next section.

Absorption refrigeration (TVC) machines are better choice for air conditioning of buildings than electrically driven mechanical vapor compression (MVC) systems in some cases such as:

1. availability of unused boiler during summer months

2. cheap heat energy source is available

3. When 100% standby electric generator is required and there is need to reduce its capacity like in hospital. In such cases, central absorption air conditioning machine with little electric energy consumption can reduce the standby generator cost.

4. Absorption refrigeration machines are more quite and vibration free compared with MVC machines

Proven types absorption machines are commercially available in different capacity ranges, e.g. Carrier Company produces machines in the range of 70-815 tons, and the York Company produces machines in the range of 114-1378 tons. Both products are using water-cooled condensers and absorbers.

ENERGY CONSUMPTION BY MVC AND TVC REFRIGERATORS

Direct use of the COP to compare the efficient use of energy by MVC and TVC machines is wrong since the energies supplied to the two systems are of different qualities. The MVC uses work while the TVC uses low availability heat. Both energies cannot be directly compared. Three rational ways of comparison are given here:

1. The first is a practical method for the present case of the co-generation cycle given in Figure 1. Fuel energy supplied to the steam generator is 877 MW to produce 300 MW electric power and 196 MW process heat. This fuel energy decreases to 779 MW when 300 MW power only is produced. So, only 98 MW fuel energy is consumed at the steam generator to produce 196 MW low available process heat. If the overall efficiency of the power plant is 0.37, then the 98 MW fuel energy added to steam generator would produce only 36.26 MW electric power energy. This is the actual equivalent work of the 196 MW process heat. The refrigeration capacity produced by 196 MW process heat is 196 x 0.7 (typical COP) = 137.2 MW refrigeration. Then, the COP based on the equivalent mechanical work is 137.2/36.26 = 3.78. This is in the range of COP for MVC refrigeration system.

2. The second method is to calculate the mechanical energy loss due to steam extraction just before the low pressure LP turbine to produce 196 MW process heat. According to the given flow sheet, 139.31kg/s steam enters the LP turbine at 2944 kJ/kg enthalpy, and 241°C temperature. There are two steam extraction to LP two feed heaters one has 7.778 kg/s flow rate and 2772 kJ/kg enthalpy, and the second 11.72 kg/s flow rate and 2611 kJ/kg enthalpy. So, the LP turbine output is :
   $W\ (LP) =$

   $$139.31\ (2944 - 2333.3) - 7.78\ (2772 - 2333.3) - 11.73\ (2611 - 2333.3) = 78508 kJ/s$$

Multiplying this value by 0.98 (mechanical efficiency), 0.98 (generating efficiency), and by (1- 0.025 (end losses) gives output work of 73.51 MW. Since this comes from flow rate of 139.31 kg/s to the turbine while 77.17 kg/s is extracted for process heat, then the equivalent work of the process heat is 73.51x (77.17/139.31) = 40.78 MW mechanical. This brings the COP of the absorption machine, based on the equivalent work loss due to steam extraction to 137.2/40.78=3.364. The difference in the values of COP between 3.364 and 3.78 obtained by the two methods came from the fact that in the first method, all the benefits of using CPDP was allocated to the refrigeration process.

3. The third method is based on the availability analysis and is not related to any specific case. Since heat is supplied to the refrigeration machine at 117°C (390 K), and heat is rejected to condenser can be at 37°C or 310 K, the efficiency of Carnot cycle operating between these two temperatures is (1 – 310/390)= 0.2. Practical cycle can have efficiency equal to 0.75 that of Carnot, i.e. 0.16, and the 196 MW process heat is equivalent to 196x0.16 = 31.36 MW equivalent mechanical work. Then the COP of the absorption machine based work calculated by this method is 127.28/31.36 = 4. This is a high COP value when compared with those calculated previously. The reason is the extracted steam from turbine at actually at higher temperature than 117 °C. In fact the extracted steam is throttled and de-superheated before its supply point of the refrigeration machine.

This concludes that the energy consumed by absorption machines TVC is comparable with MVC machines, and COP of both systems are close to each other when the equivalent work of heat added to the TVC system is used as energy consumed.

THERMAL STORAGE SYSTEM

The electric power demand follows the A/C cooling load, and the maximum load lasts for only few hours in hot summer days. So, the installed capacities of both power plants and A/C systems are much higher than average demands. The process heat rate to absorption machines (QR) can vary with the cooling demand by changing the flow rate of hot water supply while keeping its temperature constant. This requires variation of the extracted steam flow rate when cooling load is varied. The existing co-generation plants were designed to supply constant rate of process heat (QP) to desalting system as this insures full utilization of the plants in terms of flexibility and efficiency. Beside the use of full process heat capacity QP, the use of hot water storage at the end of the district piping distribution system can improve the cooling system. At peak demand time, refrigeration machines can be supplied by heat rate (QR) higher than the heat rate produced by the co-generation plant (QP). In fact QP would be the heat rate required at the average cooling load. This enables the use of higher capacity cooling machines supplied during peak hours by QP from the co-generation plant plus energy from hot water storage. Better usage of the piping system by flowing constant rate lower than the maximum flow rate. The piping system is used as a part of the storage capacity.

A further step is the use of cold water storage. This decreases the capacity of refrigeration machine to match the average load and allows continuous operation of these machines at full load. Continuous operation of cooling machines at full load gives better performance ratio, especially at night with low condensing temperatures.

SUGGESTED DISRTICT COOLING SYSTEM

A new university campus with residential and commercial areas surrounding the campus is planned in Kuwait. A previous study, Darwish et al (1987), showed that the maximum cooling load demand is 504 MW, while the average cooling demand is 190 MW. This is based on average to maximum demands of 0.21, 0.65, and 0.34 for classrooms, residential area, and commercial area respectively. The Ministry of Electricity and Water (MEW) has set conservation measures that define the maximum allowable electric power to be drawn for air conditioning A/C and lighting in different types of buildings based on minimum acceptable COP of 3 for MVC air conditioning system. So the MEW is required to provide additional installed power capacity for the A/C system only equal to 168 MW if conventional MVC air conditional system is adopted. If absorption A/C is used the maximum parasitic electrical power needed is estimated by 20% that of the corresponding MVC system of the same capacity (Suri et al.), i.e. 33.6 MW. The peak cooling demands of 504 MW would be satisfied if absorption A/C machines of 150,000 tons (524 MW) total cooling capacity. The process heat extracted from existing two 300 MW steam turbines, equal to 392 MW, should satisfy the thermal energy input for these machines if hot water thermal storage tanks are used. By assuming 10% thermal energy loss, the average process heat supply is equal to 352.8 MW thermal energy. The average cooling capacity produced by this amount of process heat, for COP=0.7 (typical value for commercial m/c) is equal to 247 MW refrigeration. This is more than the required average cooling load required. Since the required heat to absorption machines at peak load is 720 MW and while only 392 MW is directly available from extracted steam, then a thermal storage capacity of 328 MW is required. It may be noticed here that the assumed 0.7 for COP is conservative number. The COP of double stage absorption machines is in the range of 1.0-1.2.

The advantages of the proposed system over the conventional method of using MVC air conditioning system can be outlined as follow:

1. The reduction of needed power plant capacity of 168-33.6=134.4 MW. This can be expressed by capital cost saving of \$120 millions, based on \$900/kW power plant cost. It also decreases the electric power demand at the peak time by 134.4 MW.

2. The electric power consumption when using MVC refrigeration machines is 82.33 MW to generate the same average 247 MW refrigeration machines (by using COP=3) provided cold water storage is used. Without the use cold water storage, the MVC refrigeration would work at part load most of the time at lower COP. The rate of energy input added to produce 83.33 is equal to 82.33/0.35=238 MW.

3. To extract 392 MW process heat from the turbine of the co-generation plant, the rate of fuel energy added at the boiler 196 MW. This extracted heat of 392 MW can produce an average refrigeration rate of 247 MW by using TVC water chillers on the assumption of 10% heat loss from the storage and piping system and COP = 0.7. The parasitic energy consumption of the absorption machines is 20% of the power consumed MVC refrigeration machines of equivalent capacity. This is equal to 82.33 x 0.2 =16.47 MW. The fuel energy required to produce this parasitic power consumption is 16.7/0.35=47 MW. Then the total fuel energy rate for the operation of the absorption machines is 47+196= 243 MW. This is almost equal to the fuel energy used when MVC refrigeration machines are used.

4. This arrangement gives heating air conditioning in winter, where the outside temperature in Kuwait can reach 0°C.

CONCLUSION

The adoption in district cooling system in Kuwait would utilize the existing co-generation steam turbine power plant producing heat for MSF desalting plants when the use of these MSF plants is stopped. It should slow the demand for continuous increase of the power plants capacity to meet the continuous demand for increasing the load for air conditioning machines. The use of hot water storage should shave the power peak (the real cause of low capacity factor) and insure the operation of the refrigeration machines at high performance ratio specially when they operate at low condensing temperatures at night. The absorption water chillers are well proven commercial products available in the market.

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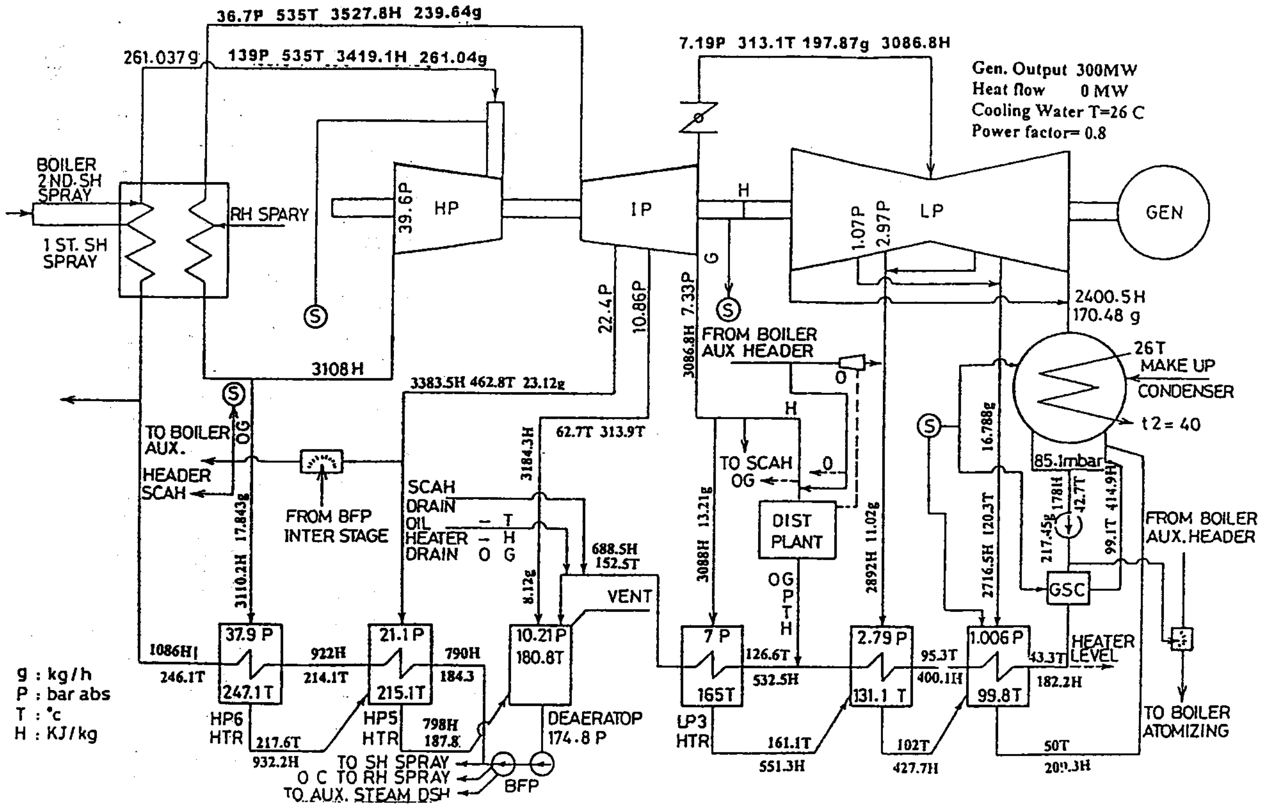

Figure1a: Co-Generation Power-Process Plant Operating in Kuwait Flow Sheet Producing 300 MW only.

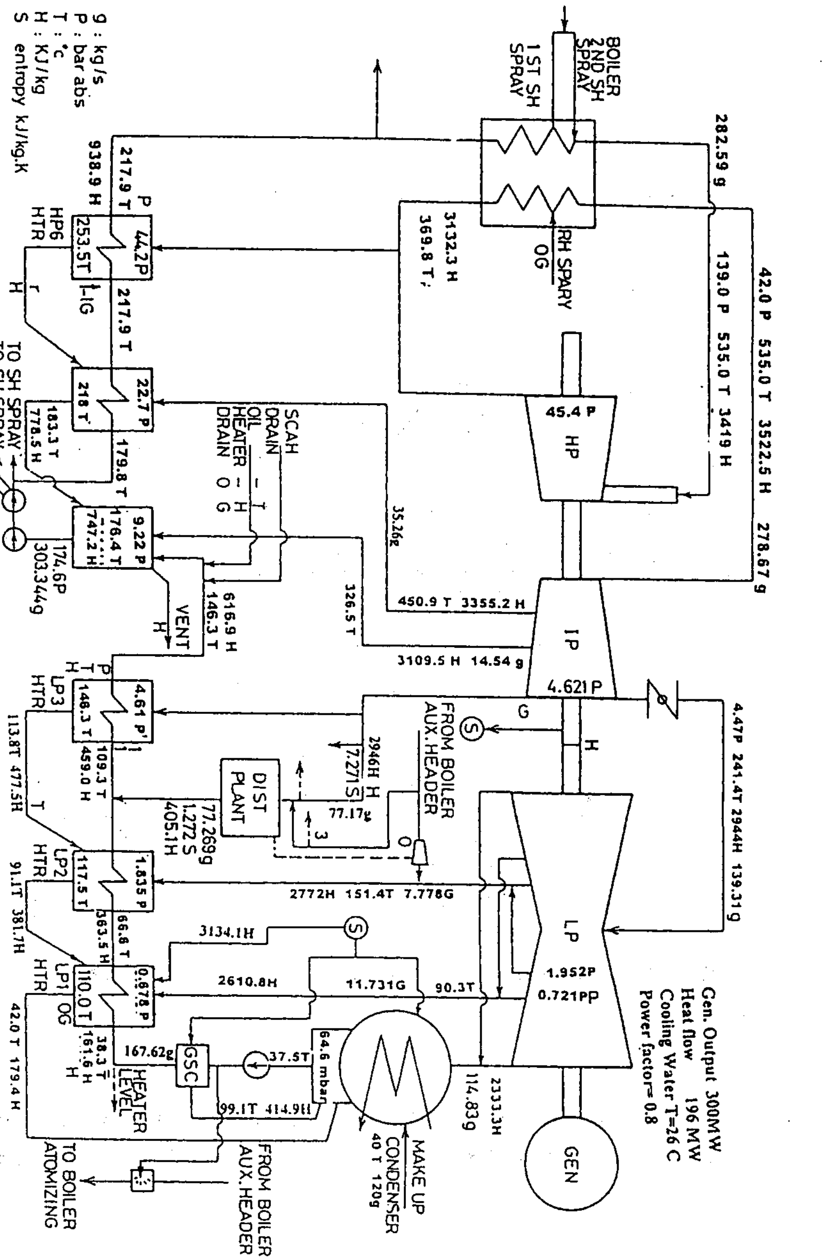

Figure 1b: Co-Generation Power-Process Plant Operating in Kuwait Flow Sheet Producing 300 MW and 196 MW Process to MSF Desalination System.

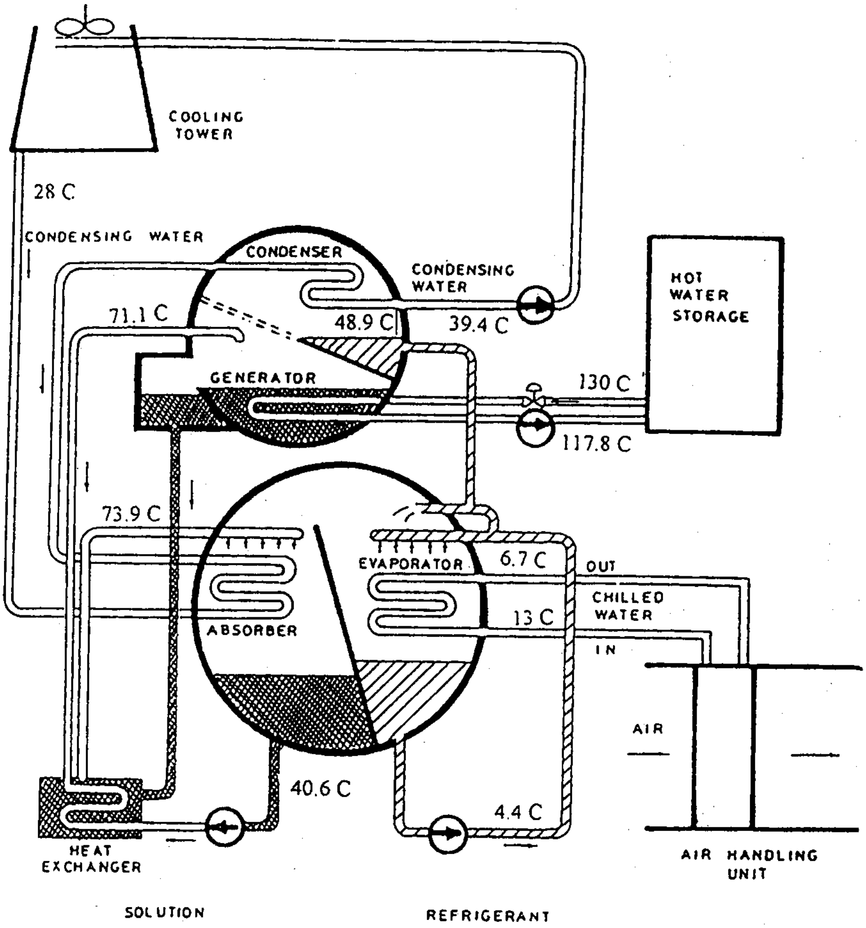

Figure 2: Flow Sheet of Water Lithium Bromide Refrigeration Machine Producing Chilled Water for Air Conditioning.

Sea Water Environmental Monitoring

Tidal Characteristics and Flow Pattern in the Abu Dhabi Lagoon System

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TIDAL CHARACTERISTICS AND FLOW PATTERN IN THE ABU DHABI LAGOON SYSTEM

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ABSTRACT

Abu Dhabi Island, United Arab Emirates has a unique tidal hydraullic system. This tidal system provides water for power and desalination plants. Therefore, understanding of the characteristics of tidal waves and flow pattern in the system will enable to optimize the performance of the power and desalination plants and to minimize the impact of such plants on the environment. Understanding the tidal system will help in the establishment of effective measures to prevent oil contamination at seawater intakes. Water level measurements in the tidal system was carried out and analyzed. Based on these measurements and analysis, the tidal propagation in the system was obtained. A 2-D numerical flow model representing the tidal system around Abu Dhabi Island was developed using the Delft-3D software package of Delft Hydraulics. The model was calibrated and verified using the available data. This paper presents the nature of the tidal characteristics and flow pattern in the complex system around Abu Dhabi Island based on field measurements and results from the numerical model.

Key words : Tidal Hydraulics, Tidal wave propagation, Numerical flow model, Flow pattern.

1. INTRODUCTION

In 1997 a hydrographic survey program in the tidal waters around Abu Dhabi Island was carried out. Hydraulic data of water levels and water flows was processed and analyzed by the Umm Al Nar Hydraulic Laboratory (UHL), Abu Dhabi Water & Electricity Authority. Data collection program was also carried by the UHL from 1995 to 1998 using their tidal self-recording gauges. This data was used in describing the tidal characteristics around Abu Dhabi Island and to calibrate and verify the Umm Al Nar 2-D numerical flow model (UEM), which covers Abu Dhabi Island and surrounding waters. This model was nested from the United Arab Emirates model, which covers the entire Emirates region. The UEM flow model is capable of simulating the tidal phenomena in the tidal system around Abu Dhabi Island.

After a brief description of the tidal hydraulics of the complex sea-lagoon system around Abu Dhabi Island, the paper will describe in detail the tidal propagation and flow pattern in the system.

2. DESCRIPTION OF THE TIDAL SYSTEM

Tidal hydraulics in the surrounding waters around Abu Dhabi Island is a complex system as it can be seen from Figure 1.

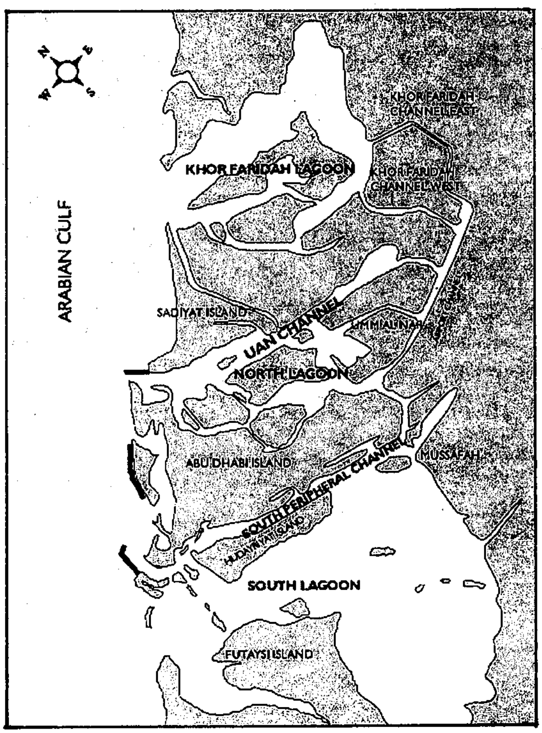


Figure 1 Tidal system around Abu Dhabi Island

As it can be seen from the figure the hydraulic system around Abu Dhabi Island consisting mainly of North and South lagoons. The North lagoon is fed from the Arabian Gulf through the Umm Al Nar channel, while the South lagoon is fed from the Gulf through South Peripheral channel. North and South lagoons are interconnected through Maqta channel. The nature of tidal wave in the part of the Arabian Gulf is of mixed, predominately semi-diurnal type, but with a significant diurnal component. The tidal wave at the part of Arabian Gulf propagates along the coast from North-East to South-West. Propagation and deformation of tidal waves in the lagoons are governed by the geometry (length, width, intertidal areas and bed roughness) characterizing the lagoon system.

3. ANALYSIS OF TIDAL WATER LEVELS

The location of the water level measuring stations is shown in Figure 2 and the period of measurements is given in Table 1. The available local water level data of at least one-month record at each measuring station was processed and analyzed. The analysis aimed at obtaining the tidal constituents based on the measurements. These constituents are used in predicting the astronomical water level for anytime at the specified location. Using tidal constituents has an advantage of eliminating the possible meteorological effects on tidal waves (i.e. wind effect) and to fill the gabs of the data due to the malfunction of the instruments, which may occur during the survey period caused by i.e. emptying the charge batteries of the instrument.

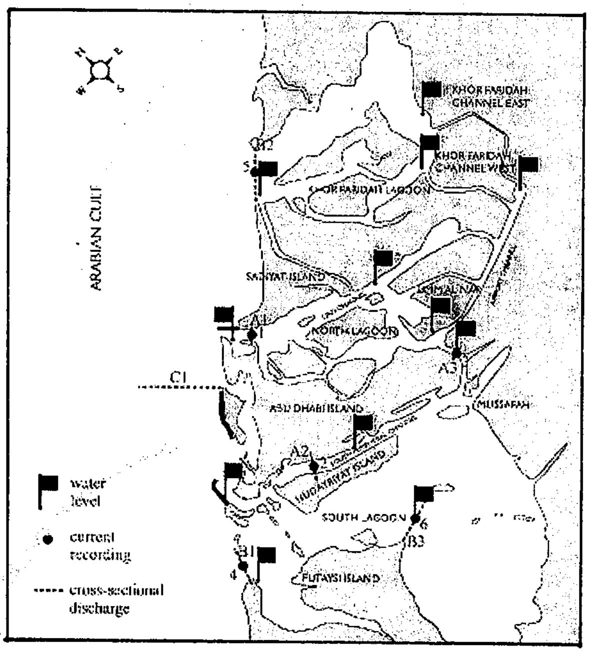


Figure 2: Location of the measuring stations and cross sections.

Table 1 : The time and period of water level measurements at the stations (Year 1997)

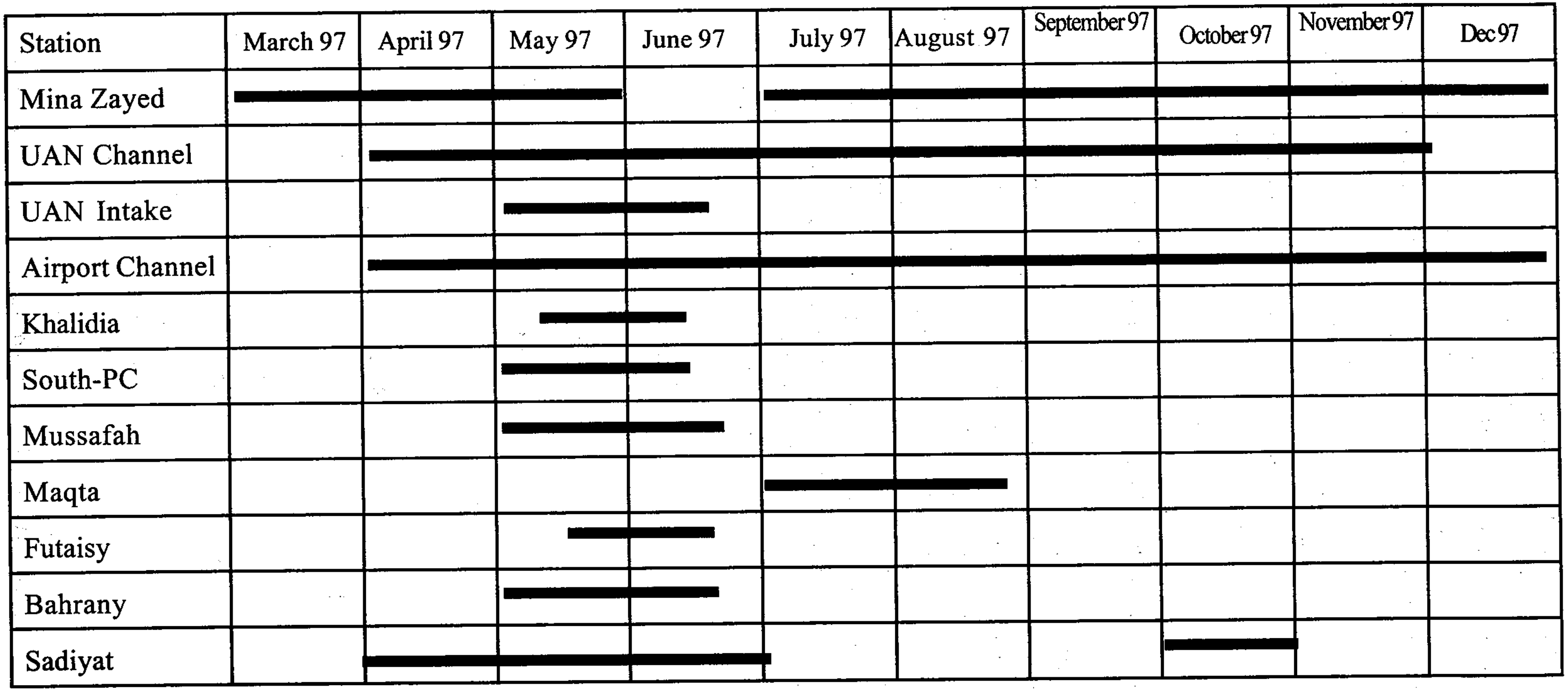


Table 1 : (continued) The time and period of water level measurements at the stations (Year 1998)

Station	Jan 98	Feb 98	March 98	April 98	May 98	June 98	July 98	Aug 98	Sept 98	Oct 98	Nov 98
Mina Zayed											
UAN Channel											
UAN Intake											
Airport Channel											
Khalidia											
South-PC											
Mussafah											
Maqta											
Futaisy											
Bahrany											
Sadiyat											

3.1 The main tidal constituents

From the tidal analysis of the water level measurements it was found that three groups of tidal constituents are the most important and have big effect on the amplitudes and phases of the tidal harmonics. These groups are as follows;

Principle tide, which consists of semi-diurnal components, M2 and S2 Declination tide, which consists of diurnal components, K1, O1 and P1 and semi-diurnal component K2.

Elliptic tide, which consists of diurnal component Q1 and semi-diurnal components N2 and L2.

Table 2 presents the values of the amplitudes of the mentioned tidal constituents as obtained from the available measurements using the tidal analysis technique. The so-called Form Number (F) is also given in the table. This Form Number determines the type of the tidal wave as follows:

$F = (H_{O1}+H_{K1})/(H_{M2}+ H_{S2})$; in which H is the amplitude of the specified tidal constituent.
and if:

$F < 0.25$, the tide is semi-diurnal
$0.25 > F < 1.5$ the tide is mixed with mainly semi-diurnal tide
$1.5 > F < 3$, the tide is mixed with mainly diurnal tide
$F > 3$, the tide is diurnal

Table 2 the amplitudes of harmonic constituents and the determined Form Number

Station	Amplitude of harmonic constituents									Form number (-)
	M2	S2	K1	O1	P1	K2	Q1	N2	L2	
Mina Zayed	42.3	17.5	33.1	18.8	11.0	4.7	3.2	8.8	2.7	0.87
Umm Nar Ch.	37.0	14.4	38.8	17.4	10.0	3.9	2.3	7.8	3.1	1.06
Umm Al Nar intake	39.2	12.4	27.3	14.9	9.1	3.3	2.1	7.7	2.4	0.82
Airport Ch.	36.0	14.0	28.9	15.8	9.7	3.7	2.5	7.1	3.3	0.89
Mussafah	35.6	14.8	28.5	14.9	9.5	4.0	3.7	7.5	4.4	0.86
Bahrany	36.4	15.2	30.1	15.8	10.0	4.7	3.4	7.5	10.1	0.89
Sadiyat	18.2	5.1	18.3	10.3	6.1	1.3	5.1	13.7	5.5	1.20
South Per. Ch.	36.0	9.8	27.2	15.3	9.0	2.6	1.8	1.6	1.7	0.89
Futaisy	24.8	9.1	23.1	11.8	7.7	2.5	2.2	4.9	3.4	1.02
Khalidia	42.0	14.5	29.5	18.6	9.8	3.9	2.3	8.6	1.2	0.85
Maqta Ch	33.1	12.7	27.7	17.8	9.2	3.4	3.2	6.8	3.6	1.0

It can be seen from the table that the Form Number for the measuring stations falls in the range between 0.25 and 1.5. This leads to the fact that the tide in the hydraulic system around **Abu Dhabi** Island is of mixed, predominately semi-diurnal type, but with a significant diurnal component. This means that two high and two low waters per day occur, showing inequalities in height and time.

3.2 Mean sea level variation

As it can be seen from Table1, continuos water level measurements for a period of one-year (March 1997 to February 1998) is available only at Mina Zayed. The monthly averaged mean sea level at Mina Zayed was obtained from the measurements. This could give insight about the possible change in the mean sea level along the year. Figure 3 shows the variation in mean sea level at Mina Zayed during the year based on the Admiralty Chart Datum (ACD). From this figure it can be seen that the mean sea level at sea, which is represented by Mina Zayed station averaged over 1 year is about 1.3 m ACD. The figure shows also that the mean sea level increases gradually from April to reach its maximum value in July (148 cm ACD), then it decreases to reach its minimum value in January (118 cm ACD).

The gradient in the mean sea level in the surrounding waters around Abu Dhabi Island can be obtained by determining the mean sea level at different locations in the tidal system at the same specific period of time. Because of the lack of the Bench marks of the measuring stations which are used to relate the vertical levels (i.e. water levels) to one level for all stations, it is difficult to get the gradient in mean sea level from the measurements accurately. The calibrated Umm Al Nar 2-D flow model (UEM) is capable of producing the gradient in the mean sea level accurate enough based on a good reproduction of the hydraulic phenomena in the model. The model was run to simulate the water levels and flow pattern for one-month period in May 1997, where the measurements were carried out. Table 3 presents the mean sea level at different stations in the tidal system.

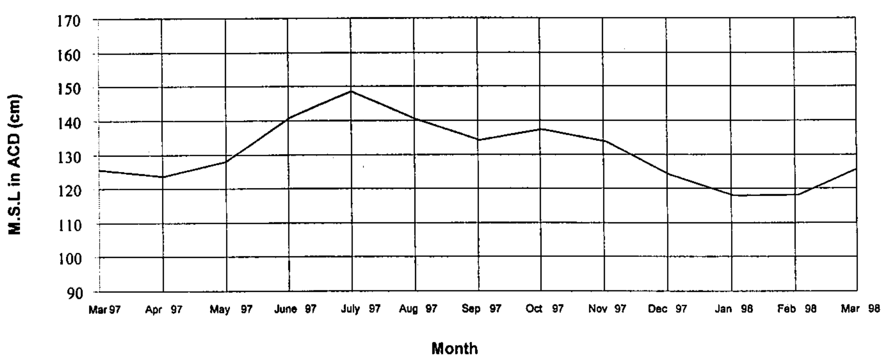


Figure 3 : Mean sea level variation at Mina

Table 3: Mean sea level at the measuring stations as obtained from the flow model

Location	Station name	Mean sea level ACD (cm)	Average mean sea level (cm)
Open sea	Mina Zayed	130.0	130.0
	Khalidia	130.0	
	Bahrany	130.0	
North lagoon	Umm Al Nar channel	133.0	134.0
	Airport channel	135.0	
South lagoon	South Peripheral channel	135.0	136.0
	Futaisy	137.0	
	Mussfah	136.0	

It can be seen from Table 3 that the average mean sea level in open sea is same everywhere and is about 130 cm ACD. The mean sea level in the North and South lagoons are relatively higher than that of the open sea by about 4 and 6 cm, respectively. The gradient in mean sea level between the open sea and both lagoons depends on the geometry of these lagoons and how they are connected to the sea, see Figure 1.

3.3 Tidal propagation around Abu Dhabi Island

The tidal system around Abu Dhabi Island can be divided into three main parts, which are Open sea, North and South lagoons, see Figure 1. The propagation of tidal waves was studied based on the spring and neap tides of May 1997, where they were used in the calibration and verification of the Umm Al Nar 2-D numerical model (UEM) and the availability of the water level data in most stations during these periods. In the paper the extremes of spring and neap tides, which occure on 9 and 16 May 1997, respectively are presented. The meteorological effect was excluded from the measurements and only the astronomic tide based on the obtained tidal constituents from the tidal analysis is presented. Figures 4 and 5 show the tidal propagation at open sea for spring and neap tides, respectively. The propagation of tidal wave along Mina Zayed, Khalidia and Bahrany represents the propagation of tidal wave at open sea. It can be seen from the figures that minor difference in amplitudes and phases at the open sea stations is observed. Generally, it can be seen that the tidal wave propagtes first to Mina Zayed, a few minutes later it propagates into Khalidia, then to

Bahrany. It can be concluded that the tidal waves propagate from North-east to South-west with almost same tidal characteristics at open sea in front of Abu Dhabi Island and North and South lagoons.

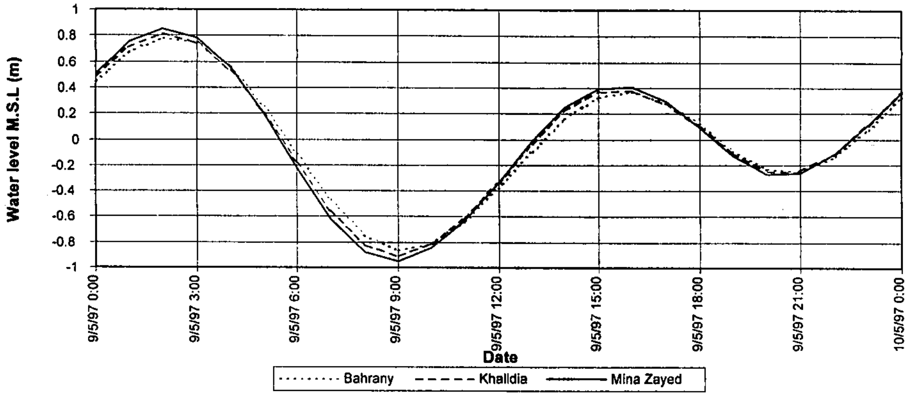


Figure 4: Tidal propagation at open sea (spring tide)

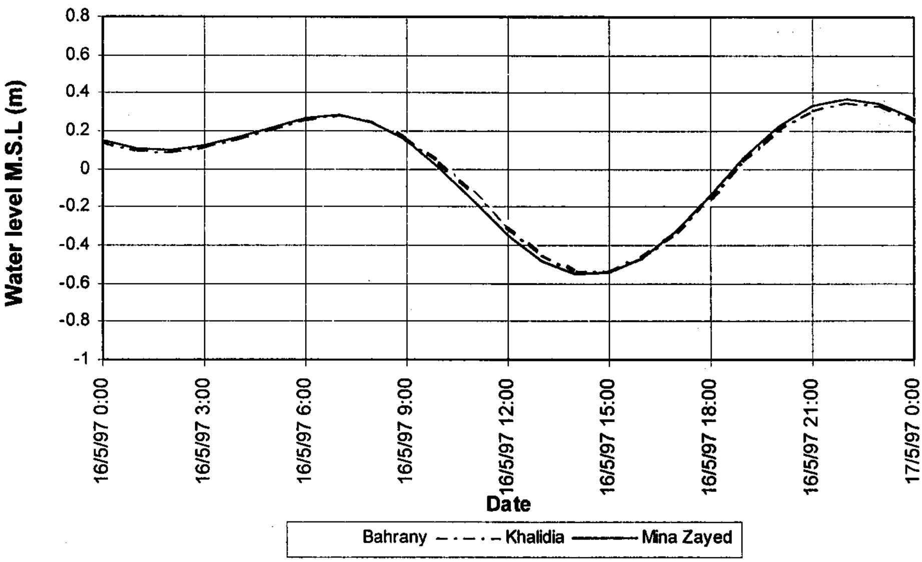


Figure 5: Tidal propagation at open sea (neap tide)

Figures 6 and 7 show the tidal propagation in the North lagoon in spring and neap tides, respectively. It can be seen from the figures that tidal wave approaches from Mina Zayed to Umm Al Nar channel, then to Umm Al Nar basin, which is represented by UAN intake.

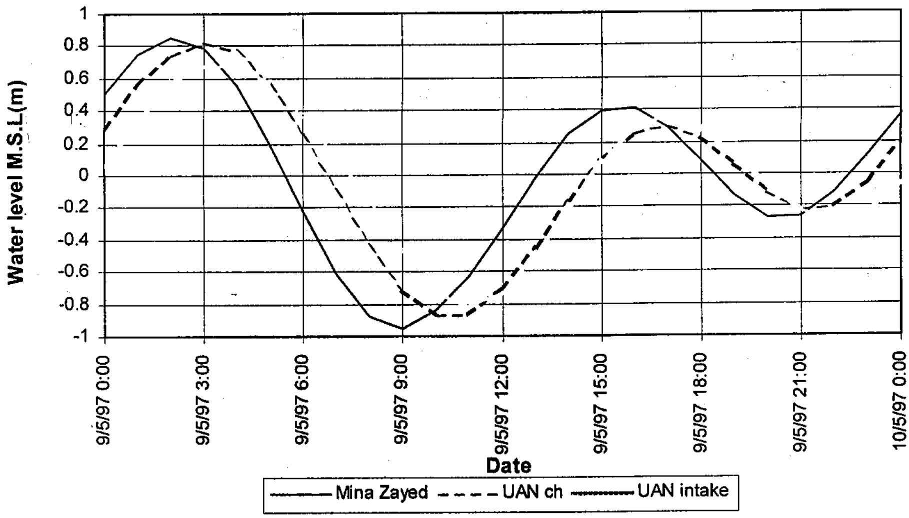


Figure 6: Tidal propagation in the North lagoon (Spring tide)

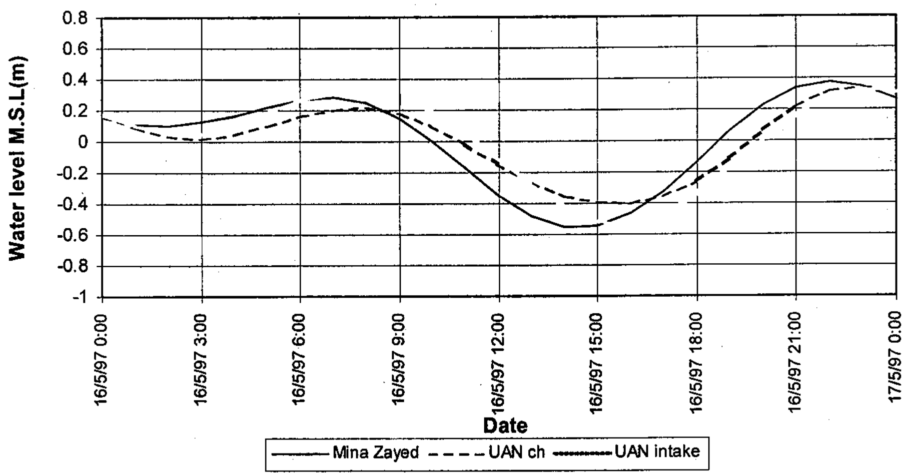


Figure 7: Tidal propagation in the North lagoon (Neap tide)

The figures show that the maximum time lag for the wave propagation from the open sea to Umm Al Nar basin is about 2 hours. Generally, wave damping occur when tidal wave propagates into the North lagoon, which depends on the bed roughness, geometry of the lagoon and main channel and the existing tidal flats. The reduction in the wave height is about 15 cm (on average) in the spring tide and about 5 cm (on average) in the neap tide.

The characteristics of tidal propagation in the South lagoon for spring and neap tides can be seen in Figures 8 and 9, respectively. It can be seen from the figure that tidal wave propagates from the open sea at Khalidia to

South Perepheral channel, then to Mussafah. The maximum time lag of the wave propagation from Khalidia to Mussafah is about 2 hours, which is more or less the same time needed for tidal wave to propagate from open sea to reach the Umm Al Nar basin in the North lagoon. Generally, wave damping occur when tidal wave propagates into the South lagoon. The reduction in the wave height is about 17 cm (on average) in the spring tide and about 3 cm (on average) in the neap tide.

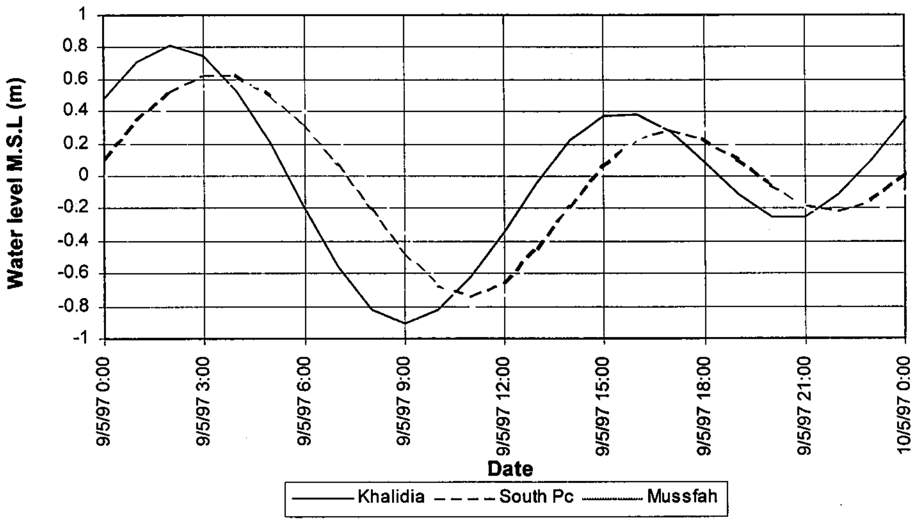


Figure 8: Tidal propagation in the South lagoon (Spring tide)

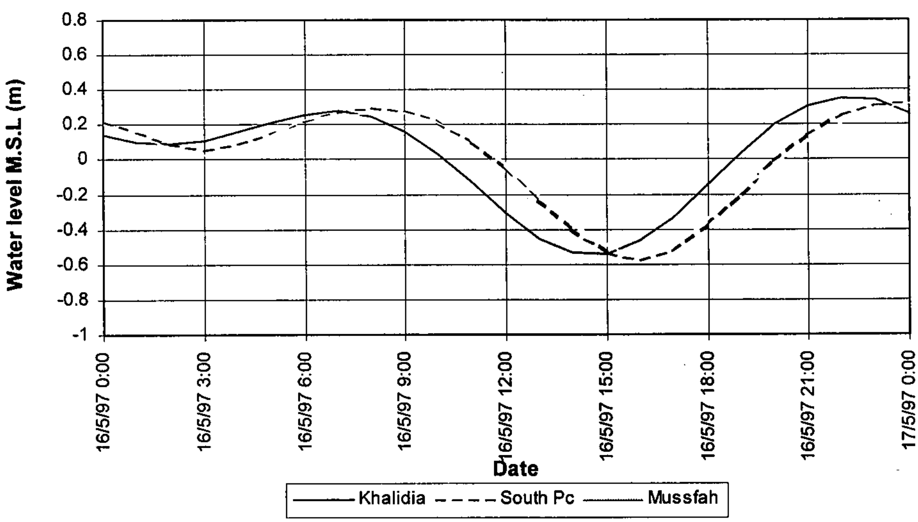
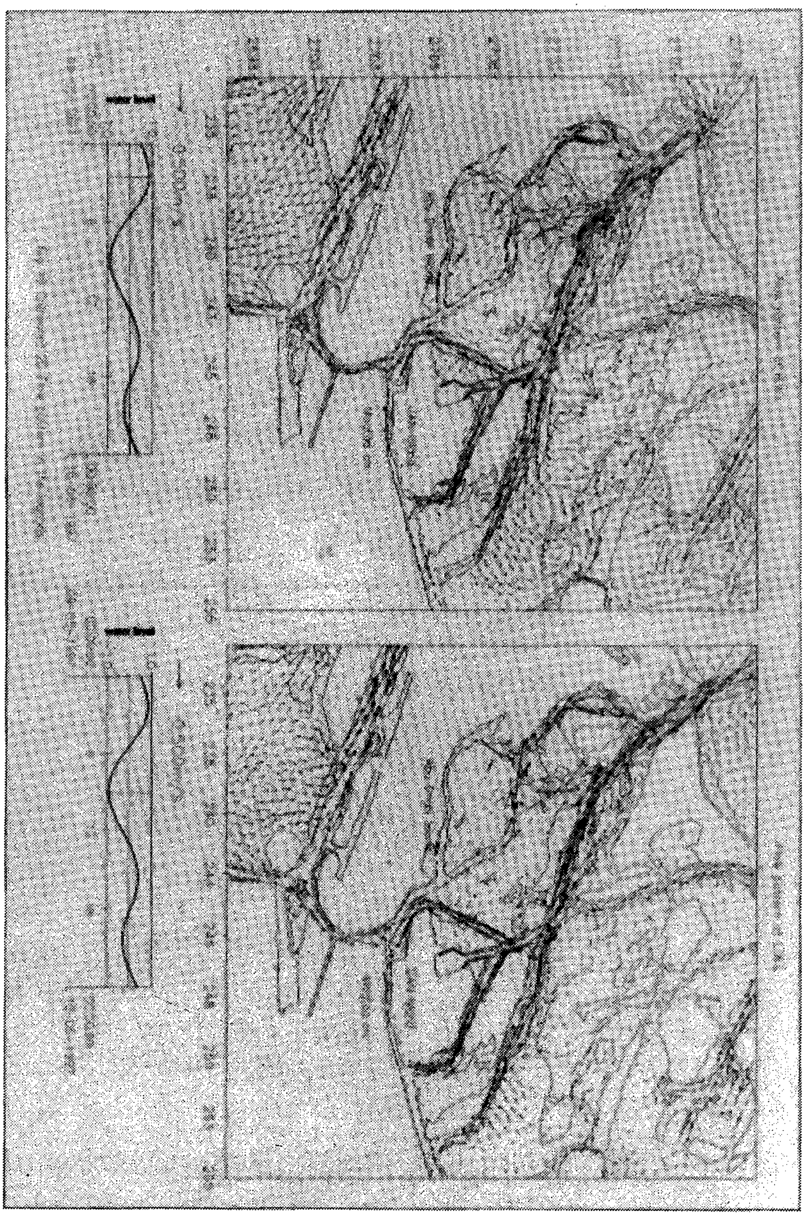


Figure 9 : Tidal propagation in the South Lagoon (neap tide)

From the study of the tidal propagation in the North and South laggons it can be concluded that the way that tidal waves propagate from open sea to each of the lagoons is more or less the same.

4. FLOW PATTERN AROUND ABU DHABI ISLAND

The calibrated Umm Al Nar numerical 2-D flow model is capable of producing the flow pattern in the surrounding waters around Abu Dhabi Island. Figure 10 shows the flow pattern in the tidal system in the spring tide at high and low waters on 9 May 1997, respectively.



From this figure it can be seen that the North and South lagoons are fed from the Arabian Gulf by a system of main channels where the flow has a big concentration. The water exchange between the lagoons and Open Sea is done mainly through these channels, which are Umm Al Nar channel in the North lagoon and South Peripheral channel in the south lagoon. It can be seen also that most of the area of the North lagoon is shallow with lower velocities, especially during the low tide. The figure shows that the velocities in Maqta channel which connects the North and South lagoons is relatively small compared to the velocities in the main channels of North and South lagoons.

Figure 11 shows the water discharge in the main channels, where most of the water exchange between the lagoons and open sea occur. These channels are Umm Al Nar channel in the North lagoon and South Peripheral channel in the South lagoon. The figure shows also the flow rate in the Maqta channel, which connects the North and South lagoons. It can be seen that the discharge in Umm Al Nar channel is bigger than the discharge in South Perepheral channel. The figure shows that the flow rate in Maqta is relatively small compared to the flow rate passing in the main channels of the two lagoons. This is because the flow rate passing from the North lagoon to Maqta channel is compensated by the flow rate passing from the South lagoon to this channel, which also can be seen in Figure 10. It is also expected that the meeting point of the tidal waves from North and South lagoons is located somewhere close to Maqta channel. This meeting point is very much influenced by the flow condition in the lagoons. This means that any change in the geometry in the topography in one or both lagoons and channel system will have a significant influence on the flow pattern and tidal propagation in Maqta channel and the location of the meeting point of the tides.

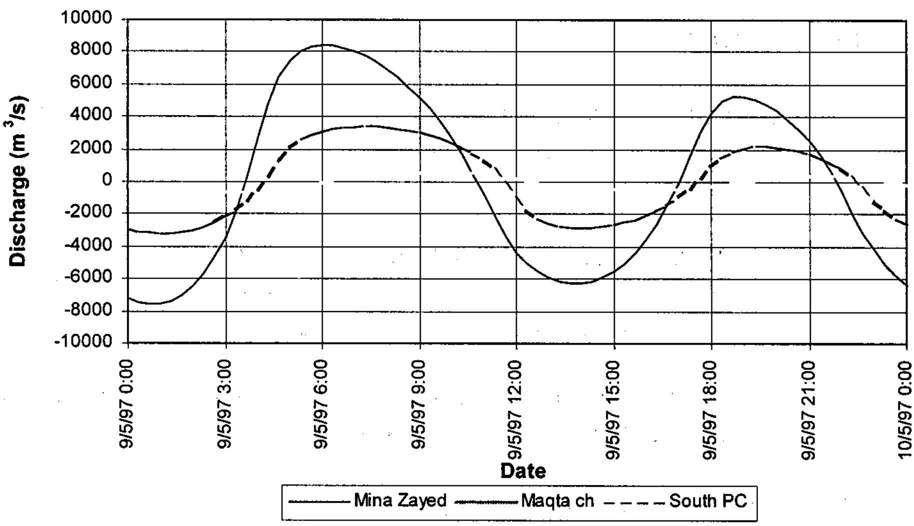


Figure 11: The flow rate in the main channels and in Maqta channel

5. CONCLUSIONS AND FUTURE APPLICATIONS

Tidal propagation and flow pattern in the surrounding waters of Abu Dhabi Island is a complicated phenomenon. Field measurements of water levels, which were carried out at different stations in the tidal system enables a better understanding of tidal propagation in the system. Based on the tidal analysis, it is found that the tide in the hydraulic system around Abu Dhabi Island is of mixed type with mainly semi-diurnal tide and a significant diurnal component. The mean sea level is varied during the year as a result of the climatic and the air pressure variation. From the analysis of the water level measurements and the results from the 2-D flow model it is found that the average mean sea level in open sea is relatively lower than that in the North and South lagoons by about 4 to 6 cm. From the tidal analysis of the wave propagation it can be concluded that the behavior of tidal propagation in the North and South lagoons are more or less the same. Tidal wave in open sea propagates to North lagoon mainly through the Umm Al Nar channel and to South lagoon through South Peripheral channel. The flow pattern in the system shows that the flow exchange between the open sea and the North and South lagoons is done mainly through the main channels passing through the lagoons, where they have a relatively big flow concentration. The flow in Maqta channel, which connects the two lagoons, is relatively small and the meeting point of the tidal wave coming from North and South lagoons is located somewhere close to Maqta channel. It is recommended to continue with the water level measurements at the mentioned stations to verify the mean sea level variation and to have more quantitative characteristics of tidal propagation in the tidal system.

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Tidal Flow Simulation by Conjunction use of Scale and Numerical Models, Case Study: Abu Dhabi Lagoon System

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TIDAL FLOW SIMULATION BY CONJUNCTIVE USE OF SCALE AND NUMERICAL MODELS, CASE STUDY: ABU DHABI LAGOON SYSTEM

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ABSTRACT

Accurate simulation of tide (horizontal and vertical), is a pre-requisite to proper planning, design and management of water resources projects in the Arabian Gulf, as well as for a better management of the marine environment.

The tidal hydraulics around Abu Dhabi Island has been simulated by both a scale model and a numerical hydrodynamic model(s). The objective being to study the tidal flow phenomena around the island, and to provide needed information for the design and operation of the water related projects. Typical examples in the area are: re-circulation studies relevant to power and desalination plants, oil spill studies, intake/outfall configuration, etc. The models also provide basic input data for environmental impact assessments.

First, the numerical model, covering wider area, provides boundary condition for the scale model. The latter is then calibrated and verified based on filed observations. Additional data from the scale model can be derived to enhance calibration of the 2D numerical model. The outcome is two (interactive) simulation tools, a scale model and a numerical model which guarantee maximum benefit from each modelling approach, had they been used independently.

The paper concludes by presenting model results at selected locations of numerical model, scale model against observations from the prototype, and comments on the future prospective of the modeling tools at the Umm Al Nar hydraulic laboratory.

Key words: Tidal scale model, numerical model, hybrid modelling

INTRODUCTION

The use of scale models for investigations of coastal engineering problems and tidal hydraulics is dated back to the 19th century (Hughes, 1993). Since then, the operation of hydraulic modelling facilities has flourished at various places in the world. Recently (the last 20 years), and due to the advancement in computer technology and mathematical theory, the numerical modeling of tidal flow, appears also as a powerful simulation tool. At present, 1D, 2D and even 3D numerical models of coastal areas, has become an indispensable tool for the hydraulic institutions.

Being part of the scientific world, although started very late, the Umm Al Nar Hydraulic laboratory, has developed more or less in the same way. During early eighties a scale model for the Abu Dhabi lagoon system, was built to advice in the design and operation of the power and desalination plants. In the mid nineties, the lab could quire a numerical simulation package (TRISULA and later the DELFT3D version) developed by W|L Delft Hydraulics. Subsequently several numerical hydrodynamic models were developed and applied to study the tidal flow characteristics in the area. In particular studies related to re-circulation of heat and salt in the intake/outfall systems, of the Abu Dhabi Water and Electricity Authority (ADWEA).

This paper highlights the conjunctive use (hybrid modelling) of the scale model and numerical models(s) as applied to the estuary system around Abu Dhabi Island. A brief description of the prototype is given as an initial step to the simulation process. Then a brief overview of the scale model is given and the corresponding results attained so far. The numerical model also reviewed and sample results are presented. The advantages and disadvantage of both modelling approaches were reviewed to define conclusions on the future prospective on their application for the anticipated studies of ADWEA.

THE ABU DHABI LAGOON SYSTEM

The tidal hydraulics around Abu Dhabi is characterized by the two lagoons, North and South, and their junction at the Maqta channel, see Fig. 1. To improve flow exchange with the open sea, deep channels were dredged. The Umm Al Nar channel in the north lagoon, and the South Peripheral channel in the south lagoon, respectively. The Maqta channel, meeting point of the tide, was also dredged to a moderate depth.

The main deriving force of flow in the system is the astronomical tide, and intermittently the wind forces. The tide in this part of the Arabian Gulf is mixed semi-diurnal type, with a significant diurnal component. The tidal range is about 1.5 to 2m. Current velocities vary from 0.7 m/s in the dredged channels, to about 0.1 m/s on the tidal flats. Propagation of the tidal wave in the channel system is governed by channel geometry (width, depth and bottom roughness). Therefore, any significant change of channel geometry will affect the propagation of the tidal wave in both lagoons, in addition to their mutual effect at the confluence point through the Maqta channel.

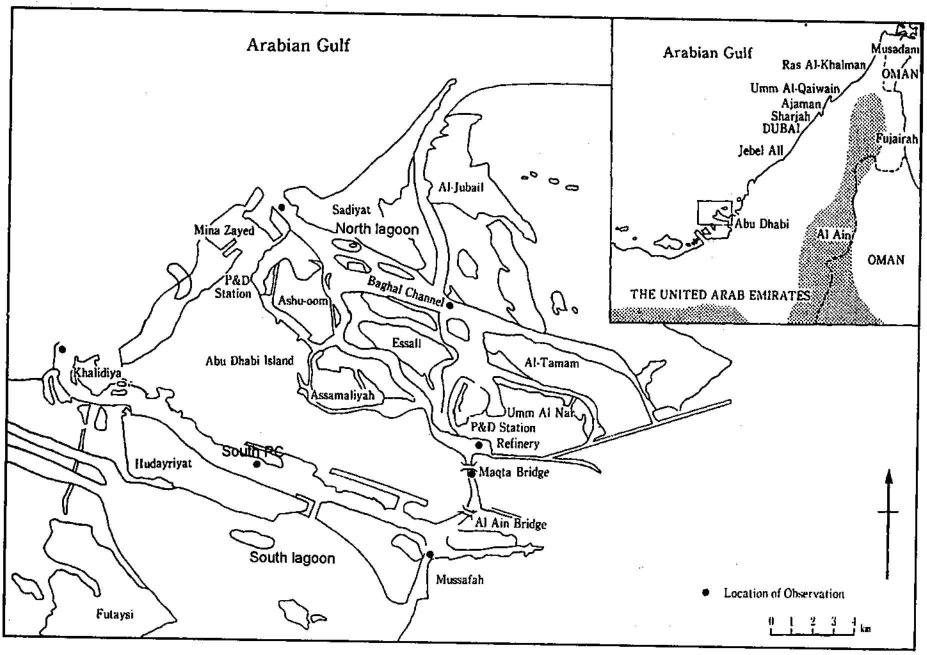


Fig.1 The Abu Dhabi lagoon system

Necessity for accurate studying of the tidal hydraulics in the area arises due to the increasing demands for power and desalinated water for the Abu Dhabi city, as well as for studying the impact of these plants on the marine environment. Design and optimal operation of cooling water systems is a typical hydraulic problem for ADWEA. Oil spill trajectory prediction to plant intakes is also very relevant in the Arabian Gulf, a busy oil traffic part of the world. However, the developed tools can equally be instrumental for studies related to tidal waters for the other departments in Abu Dhabi.

UMM AL NAR SCALE MODEL

The scale model simulating Abu Dhabi lagoon system at Umm Al Nar Hydraulic laboratory was first completed in 1987. Thereafter, it had been used for related studies until early 90's. During the period 1994/98, the model topography has been updated to the then present bathymetry (1997 condition), as well, it has been modernized to the state of the art technology, regarding model control system, tide generation and data processing techniques. By now, (end of year 2000), the model is in its final stages of calibration.

The scale model covers an area of 560 km^2 of the prototype, with a horizontal scale of 375, and a vertical scale of 60, resulting in a distortion factor of 6.25. Optimal locations of model boundaries were defined by the Umm Al Nar 2D numerical model, which covers larger area of the lagoon system (Thabet et al, 1996). Discharge boundary conditions were defined at 5 locations, UAN North, UAN West1, UAN West2, UAN South2 and UAN South3. Water level boundary is defined on the southern part of the sea at UAN South1 as shown by Fig. 2 below.

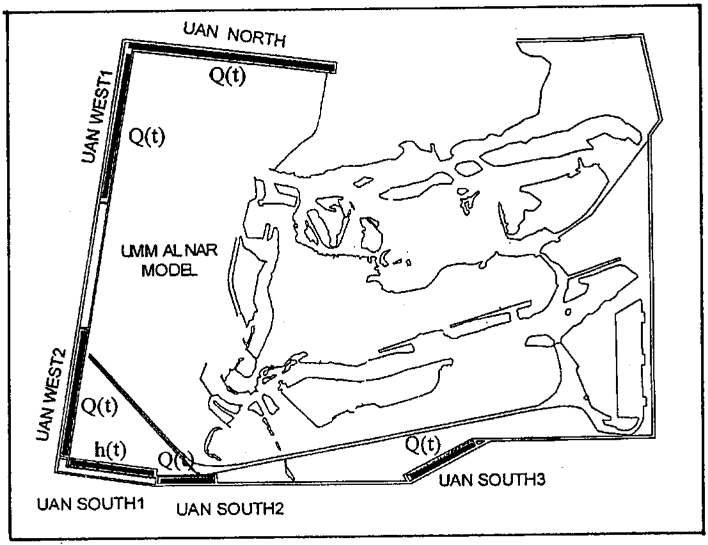


Fig.2 Umm Al Nar scale model and location of boundaries

The use of hybrid modelling approach in defining the boundary condition for the scale model has the advantages over direct measurements, not only as being cost effective, but also its flexibility to investigate numerous options and select the correct set up of boundary conditions and directly the associated values.

Intensive field survey campaign was conducted during the summer of 1997 to collect the necessary data for model calibration. Water level, current magnitude and direction, and discharges were measured during a spring-neap tidal cycle for the locations given in Fig 1. The data have been thoroughly validated, and missing gaps were completed. The spring cycle data of May 1997 is (currently) being used for calibration, while for model verification the data of the neap cycle will be utilized.

After reviewing the first results from the scale model versus the measured data, model calibration started by adjustment of the boundary conditions. It was found that the water level boundary UAN South-1, is the main controlling factor for flow condition in the open sea part of the model. Therefore, in order first to attain acceptable results for the open sea stations, the amplitude of the main tidal constituents (O1, K1, P1, N2, M2, and S2), of the South-1 boundary condition, has been adjusted by 10%, and the phase by +5°. The water level results are very much improved as depicted by Fig. 3.

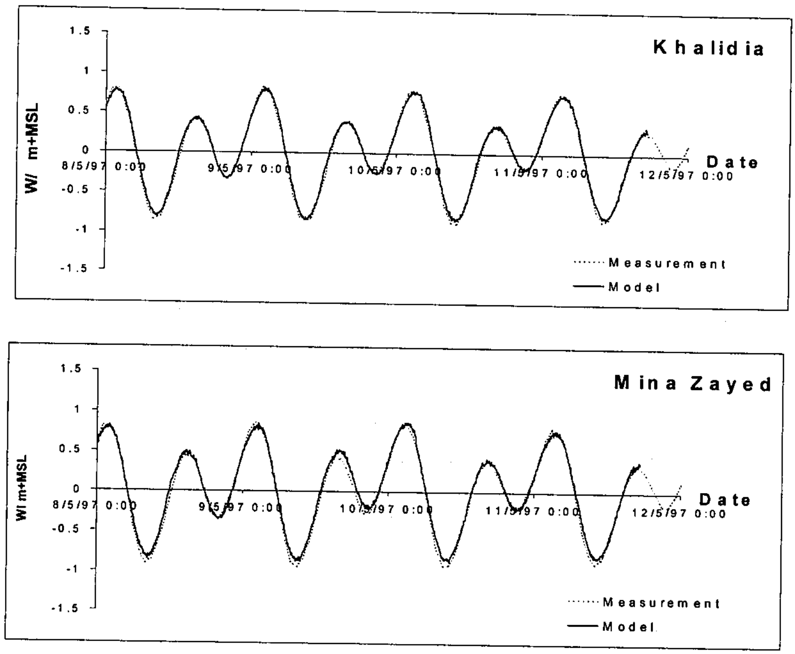
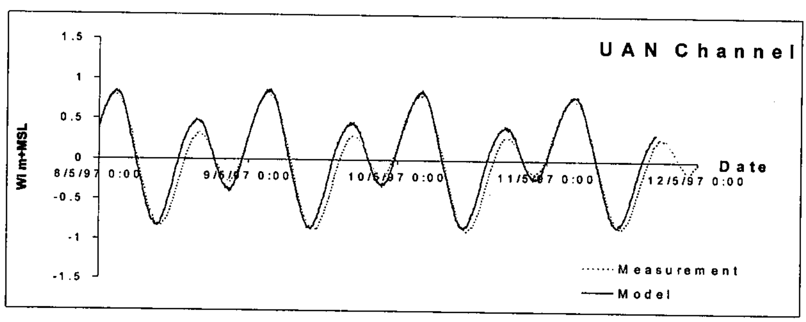


Fig.3 Calibration results at the Sea Stations; Khalidia and Mina Zayed

The next step would be to improve the calibration results of the inshore stations. Sample results of the present model situation, for those stations; Umm Al Nar channel (mid way) and Maqta channel (meeting point) are shown in Fig .4.



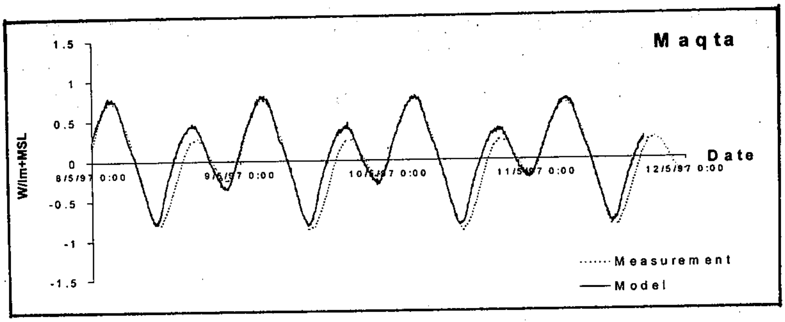


Fig.4 First calibration results at the inshore stations; UAN channel and Maqta

As can be seen, although the high high water is accurately simulated, still the low high water of the model results not yet resembling the measurement. An effective procedure, to facilitate calibration of the scale model is by testing a given calibration scenario first in a numerical model covering exactly the same area of the scale model, and has the same boundary conditions. Then to apply only the effective modification to the scale model. This save a lot of time and effort needed for rebuilding of the scale model. Presently, the calibration of the scale model is undergoing at this stage.

Although numerical modelling is being used intensively at the hydraulic lab of Umm Al Nar, as will be reviewed in the next section, it is anticipated to make use of the scale model to improve the calibration of the numerical model at some specific locations. Also, still there are some applications of interest to ADWEA, that can not yet be modelled by numerical models, e.g. configuration of intake/outfall systems in order to study detailed aspects of their hydraulic performance, these will be modelled by scale models. Generally, the present tendency is toward hybrid models that couple the far field numerical hydrodynamic model results to a smaller physical model covering specific problem areas that can't be handeled adequately by numerical modeling.

It is of mention that the scale model is a very useful tool for public relation purposes at the hydraulic lab.

UMM AL NAR NUMERICAL MODEL

The huge advancement in computer software and hardware and the progress in the solution of numerical methods, make computer simulation of tidal systems quite competitive compared to physical models. Experience of numerical modelling to solve wide scope of coastal engineering problems, has proven success in many places of the world. The early numerical models of esturine systems was attempted by 1 dimensional mathematical approximations, soon after, the 2D simulations finds vast applications, and very recently many real life problems are studied using 3D computations. It is clear that, the fast development in computer graphics, and animation facilities also make the use of computer models no longer an ambiguous job. Model topography and the water motion can easily be visualized in 3 dimensional view, which really facilitates model development and checking of results.

The UEM model, a numerical hydrodynamic 2D model developed for studying the tidal hydraulics of the Abu Dhabi lagoon system. It covers much wider area than the scale model. The two main objectives of the model is; to provide boundary conditions for the Umm Al Nar scale model, and secondly, to be used as a stand alone hydrodynamic model for tidal flow simulation within the lagoon system. The model is based on the DELFT3D modeling package of W|L Delft Hydraulics. The topography of the area has been simulated by an orthogonal curvilinear grid (173*228 computational grid cells), see Fig.5. The grid is denser along the channel system 70*70 m^2, and about 1500*1500 m^2 at the model boundaries.

To reduce errors in the cross advective terms, orthogonality of the grid has been carefully checked, so that angle between crossing grid lines is always approximately equal to 90°.

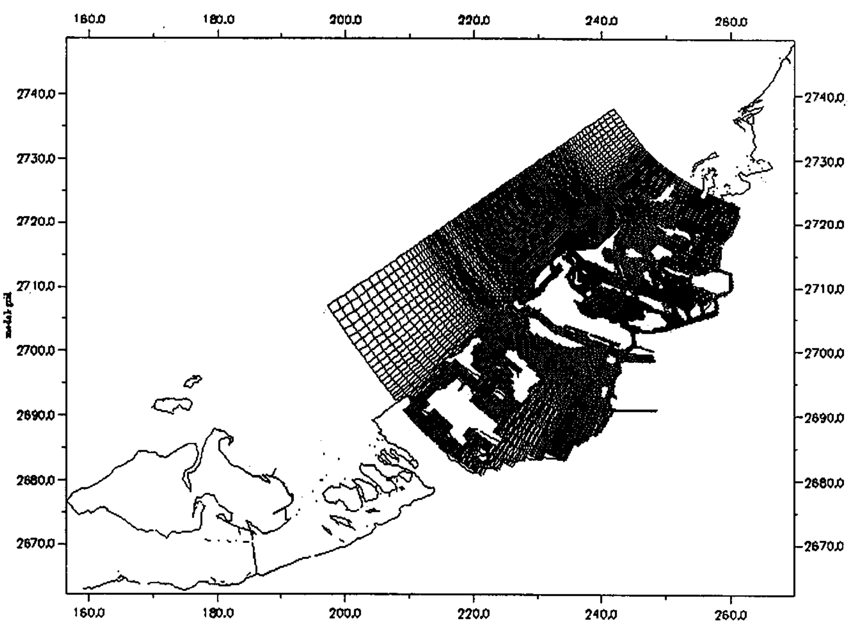


Fig.5 Umm Al Nar model grid

Bottom topography of the area for 1997 condition have been compiled from various sources; admiralty charts, ADCO bathemetric maps, and additional surveys conducted specifically for the model development. The model gets its boundary condition on the seaside, as astronomical tidal constituents, from a larger 2D numerical model "REG" covering the southern part of the Arabian Gulf, from Dubai to Dawha southward. The "REG" model also nested in the complete Arabian Gulf 2D model, which covers the whole of the Arabian Gulf. In this way of nesting (hybrid modelling), consistent set of boundary conditions could be derived to the smallest detailed model of Umm Al Nar.

The UEM model has been calibrated and verified based on field observations of tide level, current magnitude and direction and discharges that measured during 1997 and 1999 for more than 10 stations in the area. Calibration procedure started by modifying (to a limited extend) the boundary condition, then by modifying bed bathymetry in some places. Bed roughness is altered to a physically acceptable ranges, vary between 0.02 to 0.04 (Manning coefficient). Sample of the calibration results are given for 3 stations, one at the sea side, mid way, and at the end of the lagoon system as depicted by Fig. 6 below.

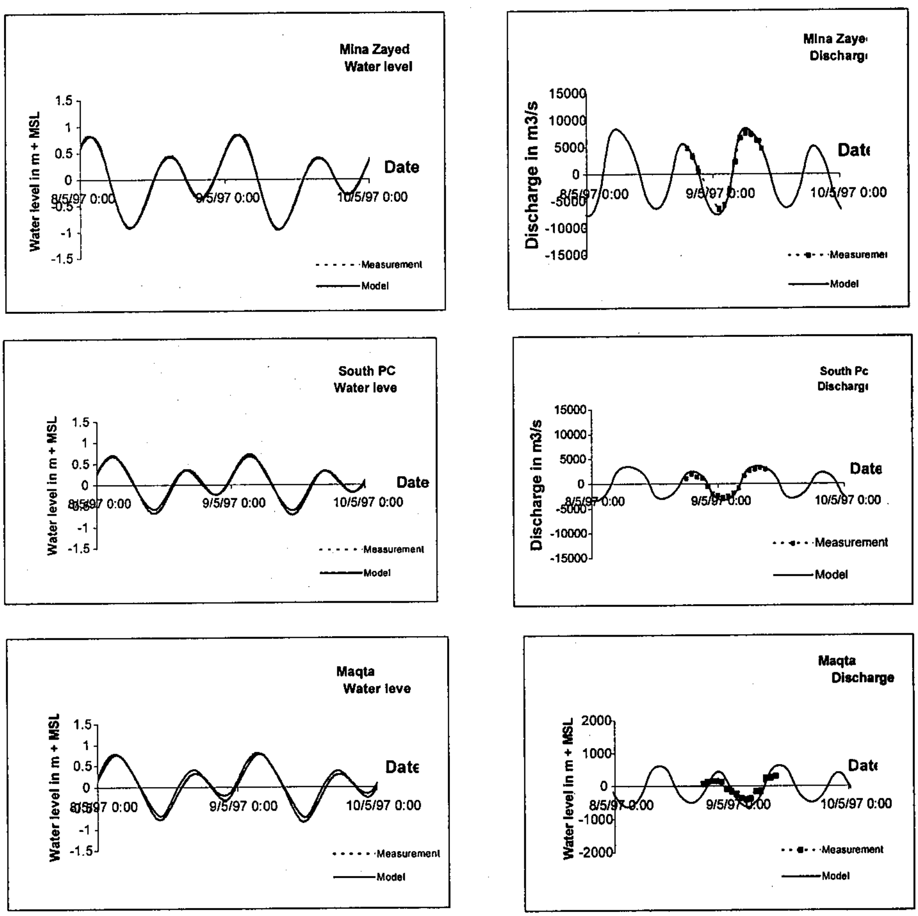


Fig. 6 Example of calibration results at sea, midway and at the inshore station

Secondly, model results were further verified by data available during 1999. An overall verification to the model for all stations is obtained by comparing tidal constituents derived from model results for one month computation, with those constituents derived from field observations. The differences in amplitude and phase of the main components O_1, K_1, M_2 and S_2 are given in table 1 below. This comparison has the advantages that, only tidal forces are compared, meteorological and other disturbances are excluded from the analysis.

Table 1 Model performance at 10 water level stations

H_c = computed amplitude **H_o = observed amplitude** **G_c = computed phase** **G_o = observed phase**	**O_1**	**K_1**	**M_2**	**S_2**
Mean H_c-H_o	-0.003	-0.017	-0.017	0.000
Abs. Mean $\|H_c$-$H_o\|$	0.011	0.017	0.026	0.010
RMS err. H_c-H_o	0.016	0.021	0.034	0.013
Mean H_c/H_o	0.988	1.062	1.048	1.003
Mean G_c-G_o	-0.7	-3.4	-0.6	-2.7
Abs. Mean $\|G_c$-$G_o\|$	4.5	4.5	2.6	3.0
RMS err. G_c-G_o	6.1	5.3	3.8	4.3

The absolute mean amplitude differences of the four constituents are 0.03m or less, and the absolute mean phase differences are lower than 5°, i.e. it is about 20 minutes for the diurnal component and 10 minutes for the semi-diurnal components. Thereafter, it is concluded that the UEM model is capable in reproducing the main features of the tidal motion sufficiently accurate. Subsequently, the UEM numerical model has been used as a core tool for the study of the Umm Al Nar Power and desalination plant extension. The re-circulation of the hot and saline plume has been investigated for different extension scenarios and the optimal one is selected based on the recirculated heat and salt at the plant intake and the impact to the echo system within the lagoon area.

Reviewing the relative merits of numerical models against scale model, it is anticipated that, the future applications of the Umm Al Nar Hydraulic laboratory will concentrate more on numerical modelling than scale modelling. The cost of scale models and their slowness in giving answers make them less competitive compared with the numerical models. An additional relevant advantage of numerical models is that certain aspects, which can not be studied by the scale model, such as wind set up, can easily be investigated by a numerical model.

Therefore, for the most common application of ADWEA, that is re-circulation studies, water quality simulation to investigate distribution of effluent constituents in the sea water and the associated environmental impact, the numerical models have been used and can be used successfully in the future. However, the scale model is not futile and will continue to be used as supportive tool to the numerical models.

CONCLUSION

The paper presents the experience of Umm Al Nar hydraulic laboratory on application of scale model and numerical models to answer design and operation questions related to the activities of ADEWA, power and desalination plants.

Model results are shown, and comparative evaluations are given. The numerical model has been used as a core tool for the re-circulation studies of the power and desalination plants in the area, as well as for the associated environmental impact assessment studies.

The use of numerical model get the advantages of being cost effective, flexible, fast in giving answer, and accurate enough for the most common application of ADWEA. Therefore, it is anticipated that to enhance the use of numerical models in the future. The scale model can be utilized to improve the calibration of the numerical model at particular areas, i.e. the two models will continue to be used interactively, with more emphasis on numerical modelling.

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Study of Seasonal Variation in Physio – Chemical Parameters Along the Coastal Area of Ras Laffan Industrial City

Muhammad R. Tayab, Arul A. Bernandaz and Roberto L. Olivenza

STUDY OF SEASONAL VARIATION IN PHYSICO - CHEMICAL PARAMETERS ALONG THE COASTAL AREAS OF RAS LAFFAN INDUSTRIAL CITY

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ABSTRACT

The newly established Ras Laffan Industrial City is situated on the Northeast of Qatar along the Arabian Gulf. It has total area of 106 km^2 with prominent establishments like port, loading berths, LNG plant, LNG and Condensate storage tanks, gas pipeline etc. In order to establish a baseline data and to assess impacts of the use of coastal waters by means of port operations, cooling water intake and discharge, seawater fire fighting system and other activities on the receiving environment, a seawater quality monitoring program was initiated. The monitoring involved determination of the air temperature, sea surface temperature, pH, salinity, dissolved oxygen, and ammonia. The study area was divided into two parts; inside and out side the port, 15 monitoring stations in and around the port were established. A station where the change in seawater quality was presumed negligible was considered as reference station. The results obtained for the monitoring period from March 1997 to February 2000 and inter parameter comparison are presented in this paper.

Key words: Ras Laffan, Arabian Gulf, Salinity, and Coastal waters

INTRODUCTION

The Industrial City of Ras Laffan (Figure 1) is situated on the northeastern coast of Qatar along the Arabian Gulf. The Simsima Limestone is known to be up to 30 m in thickness in the Ras Laffan area and of Middle Eocene age. The limestone is light brown to whitish gray, moderately weathered slightly fractured with occasional cavities infield with gypsum of clay [1]. The City occupies a total area of 106 km^2 has prominent industrial establishments like port, sulfur berth, heavy cargo berth, RO-RO berth, tug and supply vessel berth. The City also has two LNG plants, LNG and Condensate storage and loading facility, two seawater intake and outlet systems, gas pipeline etc.

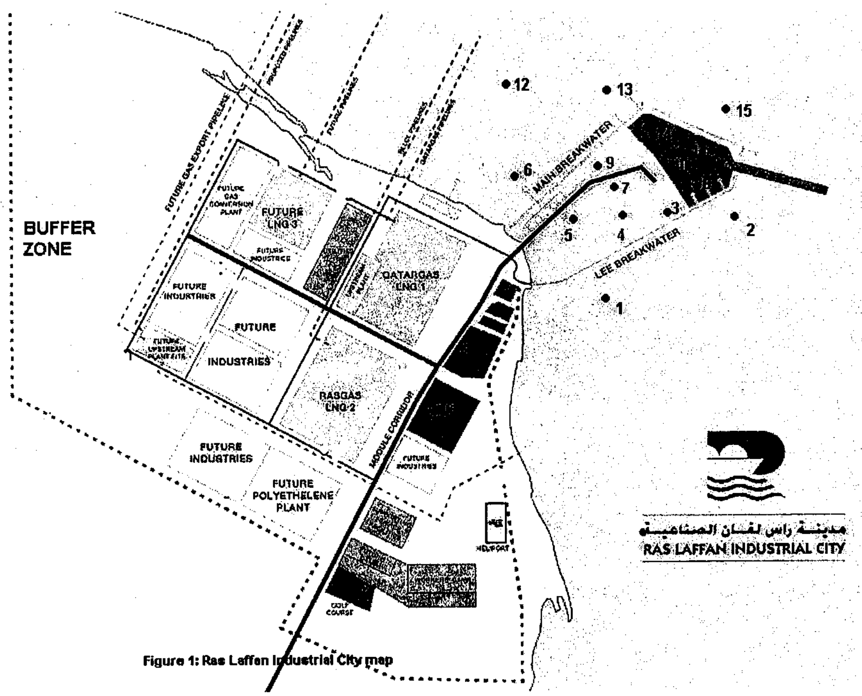


Figure 1: Ras Laffan Industrial City map

The use of seawaters for port activities, cooling water intake and discharge, fire fighting system etc has potential impacts on the quality of the receiving water. The monitoring helps in detecting changes in the environmental quality and taking preventive measures immediately.

To achieve these objectives, a seawater quality-monitoring program was carried on monthly basis. The monitoring involved determination of sea surface temperature, pH, turbidity, salinity, dissolved oxygen, and ammonia. The results obtained for the period from March 1997 to February 2000 and inter parameter comparison are presented in this paper.

OBJECTIVES OF THE STUDY

The objective of this study was to monitor and assess change in seawater quality on monthly basis, to protect and take preventive measures for the long-term resources of marine life and water supply in the Industrial City. Samples were collected inside and outside the port and analysed so that deviations if any in the water quality may be assessed at an early stage.

SCOPE OF THE STUDY

The scope of the study/monitoring was to assess the impact of port activities, plant operations, and debalasting on the marine environment of RLC by :

1. Collecting baseline information and data for further assessment and reporting

2. Collating information collected into one comprehensive environmental document/report

3. Developing an environmental management and monitoring program to identify mitigation strategies targeted towards avoidance, minimisation and impacts

4. Supplying data for decision making, employing good methodologies and techniques, provide information for future assessment.

DESCRIPTION OF THE STUDY AREA

The Ras Laffan port has an area of 8.5 km^2 in the adjoining coastal water is an artificial semi-enclosed bay (Figure 1) with a narrow opening of 400 m towards open sea. It has an entrance channel of 5.5 km long and 280 m wide, a turning circle of 13.5-m deep and 750-m diameter for manoeuvring and berthing of ships and two-anchorage area. There are two LNG berths for loading 1.35×10^5 m^3 LNG vessels, one condensate berth for loading 3×10^5 DWT vessels and berth for handles dry cargo/container on 2×10^4 DWT vessels. The newly built port is mainly handles export of LNG, Condensate and sulphur, which is the largest LNG port in the world.

Water movements in the study area are predominantly tidally influenced due to the shallow water depth in the region. Previous studies in the proximity of Ras Laffan Port indicated that currents near Ras Laffan display predominantly semi-diurnal patterns along the coast and predominantly diurnal current patterns at offshore sites. In addition, physical structures such as the port result in variable current conditions. To the north of the Ras Laffan Port residual currents are south-easterly to northerly, whilst to the south of Ras Laffan Port, residual currents are predominantly north-westerly, east to south-easterly and northerly [1]. The coastal water is shallow with depth between 1 and 14.5 m the seabed is mainly sandy but corals dead and alive also observed at various locations.

APPROACH STRATEGY

Among the several physico–chemical parameters of the water quality, only strategically important parameter viz., sea surface temperature, air temperature, pH, salinity, dissolved oxygen, and ammonia were monitored. The Ras Laffan port was divided into two areas; inside and out side the port and 15 monitoring stations in and around the port were established. The monitoring frequency was set to once a month.

METHODOLOGY

A portable Water Quality Data Logger (pHOX-900) consists of temperature, pH, salinity, conductivity, dissolved oxygen, turbidity and Ammonia probes was used for *in-situ* monitoring. All the probes were calibrated prior to the monitoring and the data was downloaded to a PC.

PROGRAM UNDERTAKEN

The onboard *in-situ* monitoring was carried out at 15 strategically selected positions (Figure 1) in and around Ras Laffan port using a mooring boat. The monitoring was conducted on a monthly basis from March 1997 to February 2000 and the data obtained were processed in the onshore laboratory.

RESULT AND DISCUSSION

The degree of stratification or mixing through water column is depended on tides, wind, atmospheric warming or cooling, evaporation and fresh water inflow [2]. In Ras Laffan there is no fresh water input to the sea therefore the marine environment has a complex phenomenon. The finding of each parameter of interest from March 97 to February 2000 is discussed below.

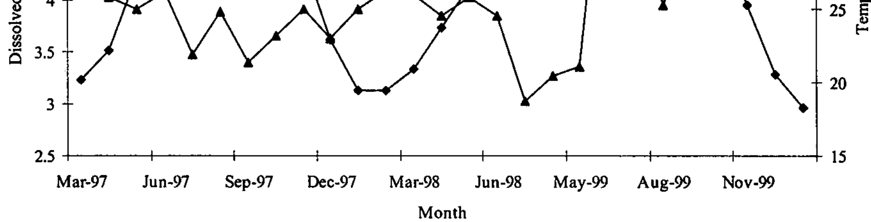
i) Sea Surface Temperature

The sea surface temperature inside the port area ranged between 18.3 and 35.6 °C where as outside port temperature ranged between 18.4 and 35.1 °C. It is natural that open sea has lesser temperature than enclosed area. The data compared with 1976 winter cruise of Atlantis, where surface temperature varied between 19.5 and 20 °C [3]. The relation between temperatures on land and sea is, the temperature is at maximum in July on air and in August on sea. At the same time, the air temperature is at minimum in January and in February on sea [4]. The air temperature in the study area ranged between 20 and 38.4 °C.

ii) Dissolved Oxygen (DO)

Higher DO concentrations in surface water are by dissolution from the atmosphere and oxygen evolution from photosynthetic activity in the photic zone [3]. Temperature, salinity, biological activity, current and mixing process controls DO distribution [5]. The dissolved oxygen inside the port area ranged between 2.93 and 5.49 mg/l where as the outside port area it was between 3.23 and 6.17 mg/l.

Naturally, the open area has higher dissolution than closed area. While analyzing the data a relation between Dissolved Oxygen and Temperature was studied, at higher temperatures the DO concentration was low and vice versa. A comparison of DO values to that of Sea surface temperature is given in Figure 2. DO is an index of biological activity in water, the general character of distribution of oxygen in water is helpful in current and mixing process studies [5].



Temperature

iii) pH

The pH of seawater in all the oceans varies between 8.1 and 8.5 depending upon temperature, salinity and partial pressure of CO_2 [5]. The pH of the study area is stratified with no notable variations between inside and outside the port values. The pH ranged between 8.17 and 9.23 inside the port area and 8.14 and 9.23 outside the port area.

iv) Salinity

In the Gulf, high salinity coupled with low oxygen is common [6]. The salinity at RLC ranged between 36.71 and 52.87 ppt inside the port and between 35.97 and 52.75 ppt outside the port area. Most of the monitoring period lower salinity was recorded in summer where as higher temperature was recorded. This may be due to the influence of hypersaline water from the Salwa bay by the currents. No study has been done on this aspects this is an assumption only. A comparison of salinity with Sea surface Temperature is given in Figure 3. The surface salinity in all oceans varies with latitude it reaches it's maximum at 20 ^{0}N and 20 ^{0}S and again decreases toward high latitude [5]. Evaporation rate is higher than the fresh water inflow causes higher salinity and in Ras Laffan study area there is no fresh water input to the sea.

Salinity in the study area is attributed to the formation of a stratified water column, thermocline and so displays a seasonal and geographical variation, paralleling that of temperature [1]. CH2M Hill reported a variation in salinity of 40-44.5‰ in coastal waters and 39-40.2‰ in offshore waters of the study area.

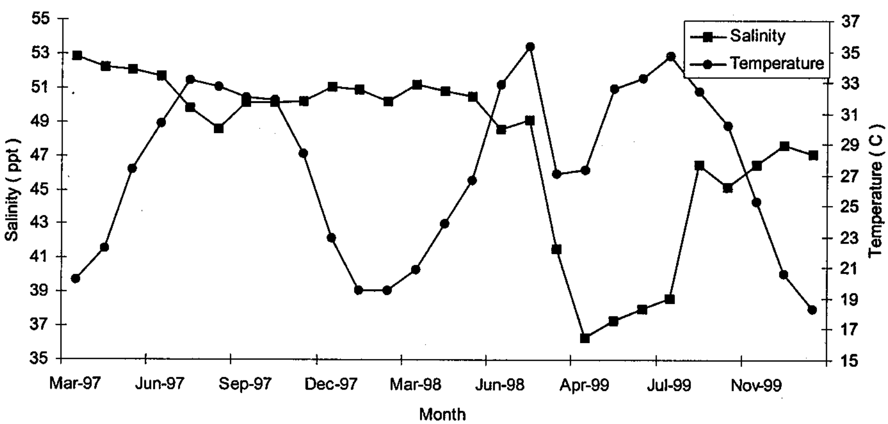


Figure 3: Comparison of Salinity with Sea surface temperature

v) Ammonia

Atmospheric deposition, sediments and deep waters are the source of ammonia for the upper water column. Particulate – N is converted to NO_3 through intermediate stages NH_3 and NO_2. Biological process could lead to net utilization of nutrients and if the rate of utilization exceeds rate of supply, the concentration decreases but during the winter the reverse occurs [5]. The ammonia ranged between 0.01 and 0.82 inside the port and between 0.01 and 0.94 mg/l outside the port.

CONCLUSION AND RECOMMENDATION

The seasonal variations have influenced much on water quality particularly on dissolved oxygen and temperature. The change in these two parameters also influenced on salinity. After comparing the data, it was found that there is no much difference in the water quality of inside and outside the Ras Laffan port area. Therefore, it can be concluded that the water in both study areas was well mixed.

It is recommended that for better analysis and assessment of water quality in Ras Laffan, the water sampling should be carried out at various depths in all monitoring stations along with oceanographic parameters. This will help to develop a physico-chemical model.

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Desalination Seawater Intakes and Feed Treatment Technology

Mohammad A.K. Al-Sofi

DESALINATION SEAWATER INTAKES AND FEED TREATMENT TECHNOLOGY

Mohammad AK. Al-Sofi

ABSTRACT

This paper will address various types of desalination seawater intakes. Special emphasis will be placed on various water capacity, type of desalination process, nature of shoreline topography, sea basin depth and its nature. Another area of interest would be disinfection requirements and its relation to above aspects. The paper will also address filtration requirements based on general knowledge of sea-born undissolved solids also known regional and seasonal undissolved solids loadings. A brief review of salinity and seasonal temperature variation will also be included. In regard to filtration and pumping an inventive concept will also be put forward.

Key Words Seawater desalination processes, intake design configuration, suspended and dissolved solids, also filtration and pumping requirements.

INTRODUCTION

Seawater desalination in the Gulf Cooperation Council (GCC) States over the past five decades has taken stepwise growth in a number of respects. These were increasing and improving design unit capacities, adoption of a number of processes, variation in plant Sites and capacities as well as integration of desalinated seawater with power production, in addition to conceptual development of process hybridization.

In the contrary to the above wide spectrum of development, seawater intake design did not receive the same level of attention in some cases, particularly where reverse osmosis was introduced and/or stretched into handling of seawater as a feed-tock. It became obvious with time that seawater reverse osmosis (SWRO) developers and more specifically promoters have thrown the process into the sea with no life jacket. Over-ealous and premature SWRO indulgence into seawater handling had a number of negative impacts among which seawater intake design, and clear identification of treatment requirements, stand up lofty as a major drawback.

CLAFFIFICATION OF SEAWATER INTAKES & FUTURE DEVELOPMENT

Intakes can be classified into three main categories. These are shore-lines, extended or sub-basin installations. Within either category, one or two (see figures 1 through 4), there are surface or sub-surface draw type, while the third category (see figures 5 through 8) can be either of above or below shore line beach well installation. Moreover, the extended category can be described as either open or contained culvert under sea basin. On the other hand, beach wells could either be horizontally or vertically built. For many years the idea of coming up with a new concept for seawater intake design had been contemplated. In this concept, a mouthless, i.e., closed lagoon or an intake channel is proposed especially for larger plants and more particularly for SWRO and/or nanofiltration plants (see figures 9 and 10). Moreover, it is felt that concrete volute pump (CVP) design renders itself quite well to this conceptual design. The adoptability of CVP is primarily due to couple of features in its design, e.g., height as the most vital of which is its limited depth requirement. This feature could very much allow to maintain water level in the closed and embanked intake basin couple of meters below low tide sea level. The purpose of operating the embanked intake basin below the tidal variation of sea level is to enhance ingress of water into the enclosed intake through sea floor under the influence of drop in liquid head inside the lagoon, i.e., established level differential. This proposed design concept could be viewed as being a hybridization of all three above categories of intake structures.

INTAKE COMPONENTS

Seawater intake components can be divided into two main groups based on the process which is either thermal, e.g., distillation or non-thermal, e.g., SWRO which is the most common non-thermal process and of a permeation nature. However, as a rule of thumb, any intake seawater requires chemical treatment in addition to debris filtration.

THERMAL PLANTS INTAKE

Thermal process intakes are quite similar regardless of the process or whether it is for single purpose desalination, dual-purpose distillate and power production by boiler, turbine & generator or even the latter by itself, i.e., BTG. One should also understand that dual purpose GTG-DIST and CC-DIST are practically the same as the first and the second in the same order as they are mentioned in this paragraph. Moreover, the use of the term DIST has been used in this context to denote all distillation processes. These in the order of their capacities and importance are Multistage Flash (MSF), Multi Effect Distillation (MED) and Vapor Compression Distillation (VCD).

In general, thermal plants intake are either of class one or two. That is to say, the extravagance of going to class three is not required at all. More importantly, the required seawater intake capacities for thermal plants makes class three quite prohibitive from the construction cost point of analysis.

Figure 1 shows a typical construction of class one. This construction can also be applied to class two in certain configuration. Shoreline intake, i.e., class one could either be of one or sometimes of two stage pumping. The main features of class one as shown in Figure 1, are (1) intake chamber (2) bar screens, (3) first (a single stage) pumping devices (i.e. combined pump and its driver) (4) finer (or mesh) traveling screens and if (required), (5) second stage pump system. In the case of a single stage pumping system the fine (or mesh) traveling screens are placed ahead of the pump. There are new designs where pumps are lowered down from a platform. In such design it is still felt that debris filtration could become a problem if not carried out ahead of pump suction bell mouth.

NON-THERMAL PLANT INTAKES

Historically, both classes: one and two intake seawater designs were used for seawater reverse osmosis (SWRO). Malta for more than one reason can be considered as the largest successful experimentation country for the adoption, if not the birth, of class three intake configuration of inland seawater beach wells.

GROSS INGREDIENT RANGE OF SEAWATER

The gross ingredient of seawater can be categorized into a number of classes. These are (1) water, (2) dissolved substance,(3) suspended matter, (4) pollutants, e.g., petroleum hydrocarbons. The last has been classified separately for more than one reason. These reasons are primarily (1) its cross over nature into class(es) two and/or three and more importantly (2) its adverse effect especially on membrane processes. However, some influential dissolved substances in seawater are metals and alloys. Likewise, two subclasses of suspended solids are also of detrimental effect. These are hard solids, suspended organic and biological contents of seawater.

CHEMICAL TREATMENT

Thermal plant intakes require four types of chemical treatment. Each one of these four is targeted towards a specific intention. In order to passivate seawater these four intentions are for the prevention of (1) biological growth, (2) corrosion (3) scaling and (4) foaming. There are ways and means, which are gaining popularity especially in recent years to do away or at least minimize chemical treatment for any and all of the above intentions. These non-chemical, i.e., mostly mechanical treatments of varying extensiveness are gaining popularity for their cost effectiveness and more importantly as environmentally friendly processes.

SWRO FEED TREATMENT

SWRO process has historically implemented a combination of chemical and mechanical feed treatment approach. Feed treatment of membrane processes especially SWRO has been very detailed and in most cases (with the exception of Malta) quite costly. Treatment costs for SWRO generally constitute twenty to thirty (20-30) percent of total plant installation project price.

For SWRO plants (of class one or two intakes) few chemicals are essential. These are coagulants and their aids, sodium hexametaphosphates (SHMP) and/or acid with or without disinfection chemical, e.g., chlorine, hypochloride, ferric chloride, ozone or (in some cases) copper ion containing chemicals.

In addition to the above chemical treatment, mechanical treatment could also be quite extensive for SWRO feed. Two to four stage filtration steps are generally used. These, in addition to bar screens, are: (1) traveling (mesh) screens, (2 & 3) one or even two stage media, then (4) cartridge and in some cases microfilters.

As stressed upon earlier, the introduction of class three intake concept came about to alleviate and more importantly reduce both chemical and more so the mechanical SWRO pretreatment requirements and hence the cost of seawater desalination by membrane processes of which SWRO is the most dominant.

Moreover, and in recent years two additional or should one say alternative, steps are being introduced into seawater desalination as technology transfers from hard brackish and/or waste water treatment industries.

Ultra and then Nano-filtration (UF &NF) are in their way to adoption as two membrane treatment steps in seawater desalination. Indeed, they are being introduced into DESALINATION PROCESSES as they have quite great potentials for all desalination processes. That is to say, it is foreseen as pilot plant experimentations are being undertaken for the adoption of UF & NF into thermal as well as membrane feed treatment schemes. Figure 11 shows the said pilot plant schematic process flow regime, where hybrids of NF-SWRO and NF-MSF as well as tri-hybrid of NF-$SWRO_{reject}$-MSF are tested, while Figure 12 shows some NF-MSF hybridized pilot plant results.

MERTIS OF VARIOUS DESIGNS

First category renders itself along deep and rocky basin shores. Surface shoreline seawater intake is obviously simpler and cheaper to build. Next to which comes subsurface shoreline intake design. Extended intake costs start to escalate depending on the nature of the site. The cheapest design is where a short dredged channel could satisfy the requirements. In such cases the topography and the nature of the basin play vital roles. The cost of contained channels, lagoons and naturally enhanced flow closed piping or culvert could vary drastically depend on site specifics. Yet, placing seawater pumps at the mouth of closed pipe or culvert is almost always more expensive than the other types of this category where pumps are installed in a pit along the shoreline.

The third category, however, is always of higher costs. Nevertheless, for membrane process, e.g., SWRO it could be very attractive as the seabed is utilized as a natural sand filter, thus overall cost effectiveness can be achieved.

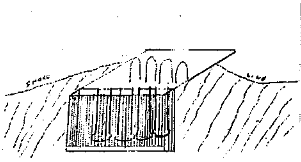


FIGURE 1. Shore line Seawater Intake

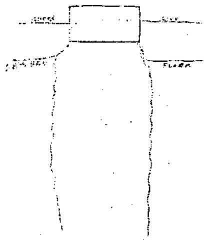


FIGURE 2, DREDGED CHANNEL INTAKE

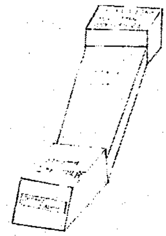


FIGURE 3, CLOSED CULVERT SEAWATER INTAKE

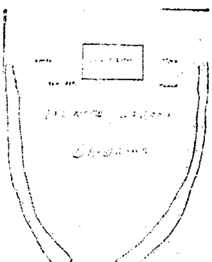


FIGURE 4, DREDGED LAGOON SEAWATER INTAKE

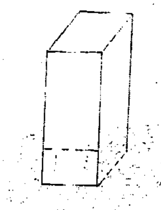


FIGURE 5, EXCAVATED BEACHWELL SEAWATER INTAKE

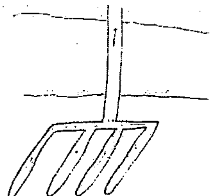


FIGURE 6, UNDER SEA BED HORIZONTAL BEACHWELL SEAWATER INTAKE

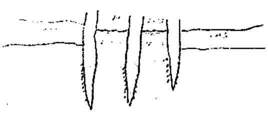


FIGURE 7, ON-SHORE SELF JET BEACHWELL SEAWATER INTAKE

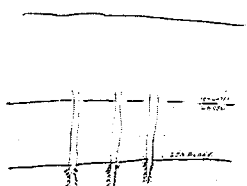


FIGURE 8, SEA BED FLOOR SELF JET BEACHWELL SEAWATER INTAKE

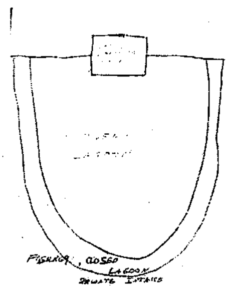
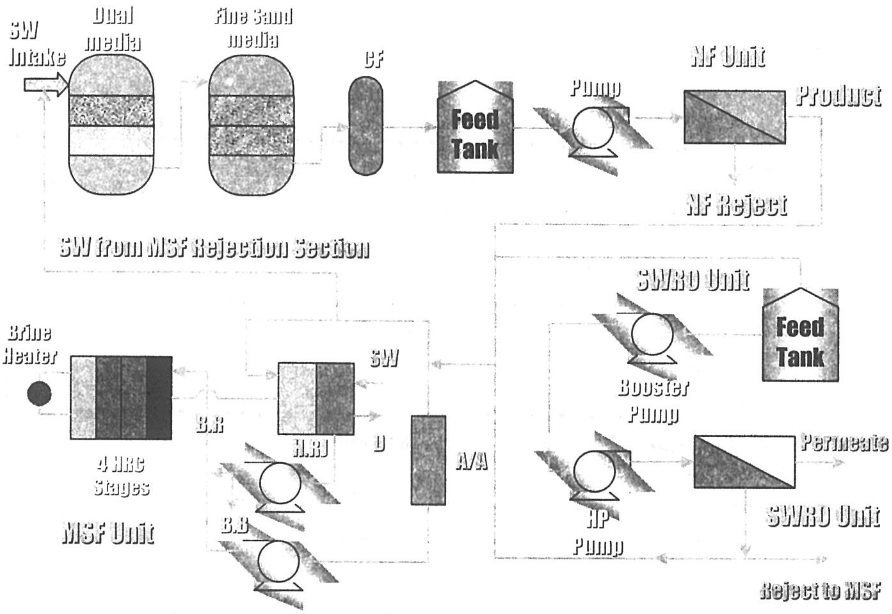


FIGURE 9, CLOSED LAGOON SEAWATER INTAKE

FIGURE 10 CLOSED SCAVENGER CHANNEL SEAWATER [illegible] INTAKE

Schematic Flow Diagram of NF, SWRO and MSF Pilot Plants



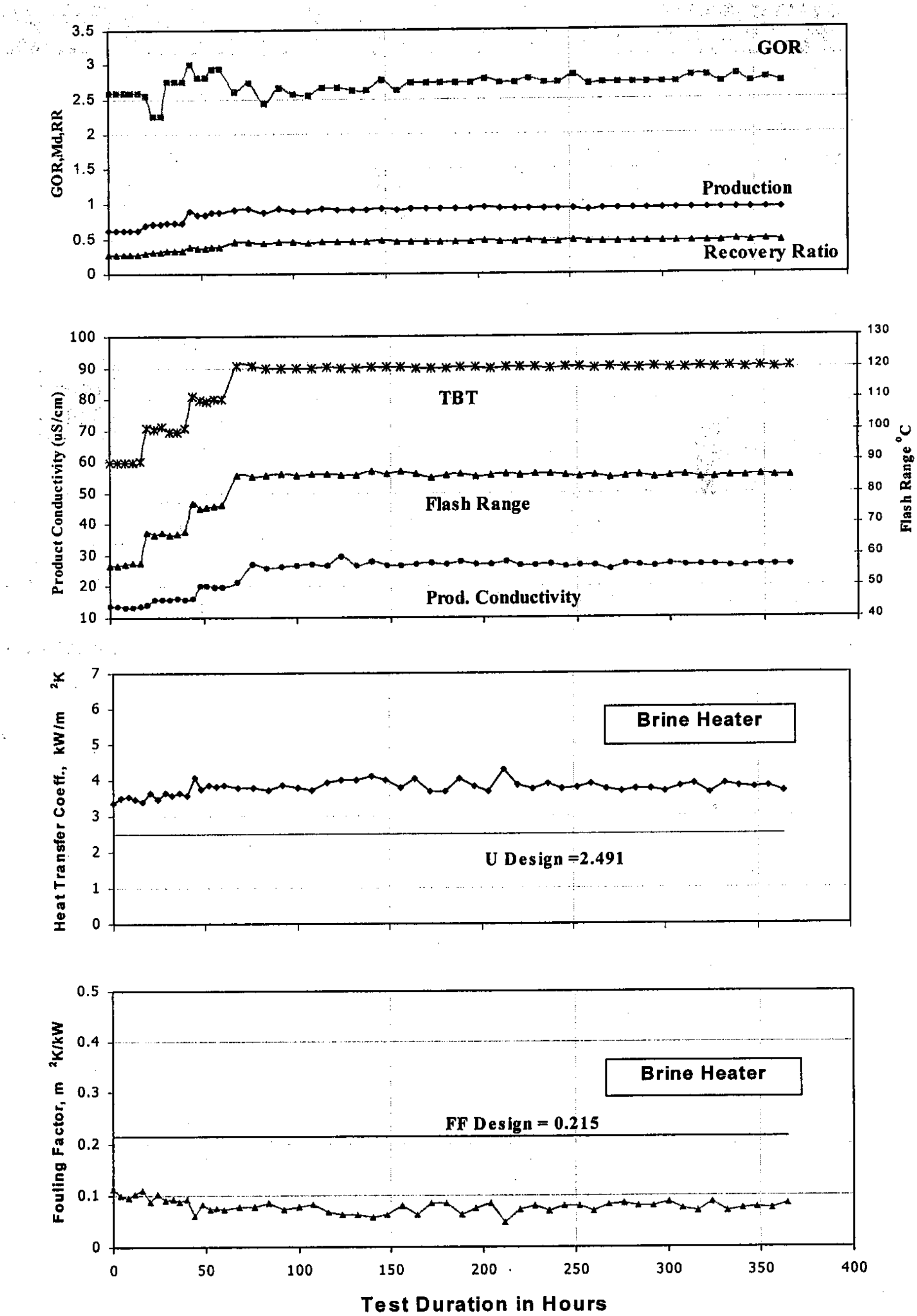


Figure: Performance of MSF Pilot Plant with nanofiltrate feed (TRISEP 80) at TBT 120°C, CR (1.53) and Makeup (flow 2.0 m^3/hr and pH 6.3-6.8) without antiscalant. ***(Based on Local Data)***

Irrigation Water

Guidelines for Design and Operation of Irrigation Systems Under Saline and Arid Conditions

M.A. Hashim and Ahmed A.H. Abdulmalik

GUIDELINES FOR DESIGN AND OPERATION OF IRRIGATION SYSTEMS UNDER SALINE AND ARID CONDITIONS

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ABSTRACT

Since the fresh water resources have already been depleted in arid zones, it is inevitable to use saline water for sustainable crop production. However, to achieve this objective great efforts are required to overcome the saline irrigation water related problems, mainly soil salinization and waterlogging. Many worldwide investigations were carried out on water quality classification and its suitability for irrigation. Water of electrical conductivity (ECw) greater than 3 dS/m is classified as having severe restriction on use for irrigation. However, no specific work has been done on farms layout and irrigation systems design associated with saline and arid conditions. This study attempts to pave the way for efficient use of saline water for crop production through improvement of farms layout and irrigation systems design and operation. It is based on field trials, design experience and observations on the use of saline water for irrigation in Qatar during the last two decades.

The study reveals that there are some design aspects of farms that need special considerations when irrigating with saline water. These include water quality, soil texture, salt tolerance and properties of crops, leaching and drainage, windbreaks, irrigation scheduling, windspeed and direction and number of operating hours. Impact sprinklers were used successfully for irrigation of field crops, forages and bulb vegetables with water quality of ECw up to 6.5 dS/m. The drip method, using irrigation water of ECw 4.5 dS/m, was more favorable to growing multi-fruit vegetables (tomato, squash and potato) than other irrigation methods. The bubbler method proved to be more suitable for irrigation of wild and date palm trees than the drip method with water salinity range between 4.3 and 9.0 dS/m. Application of these improved farms layout and irrigation methods in rehabilitation of existing farms and reclamation of abandoned farms will increase crop production and conserve land and water resources.

Key words : Saline water, arid conditions, farms layout, irrigation systems

INTRODUCTION

The fresh water resources available for agriculture have already been depleted in many parts of the arid lands. On the other hand, most of the arable soils have been over exploited. What is left for us to utilize for extra food production in these arid regions is the saline water.

There are a number of potentially undesirable impacts on the environment, as well as on the economic and social components of society caused by improper irrigation, which must be considered, if agricultural production is to be sustained, even more so if it is to be expanded by the use of saline waters (Rhoades et al., 1992). These undesirable impacts include mainly soil salinization and waterlogging. In Qatar the latest water survey indicated that the number of abandoned farms boosted to 288, which represents about 24% of the total number of farms (Hashim,1998). About 74% of the ground water used for irrigation in Qatar is classified as saline water with electrical conductivity (ECw) equal or greater than 3 dS/m (DAWR, 1997). According to Ayers and Westcot (1985) water of ECw greater than 3 dS/m is considered to have severe restriction on use for irrigation.

The factors causing salinization and waterlogging in the soils of arid regions have been investigated by Balba (1976), El Gabaly (1978) and Skogerboe and Walker, (1981). These factors include: (i) using poor water quality for irrigation; (ii) inefficient water use for irrigation; (iii) lack of adequate drainage; (iv) seepage from canals and reservoirs; (v) rising of water table due to inadequate water and soil management; (vi) uneven distribution of water; (vii) improper irrigation and drainage techniques at the farm level, from both over-irrigation and under-irrigation; (viii) presence of impermeable layers in the soil profile; and (ix) areas of low relief adjacent to or surrounded by areas of relatively higher relief which usually receive water from these surrounding areas.

It is worth mentioning that many worldwide investigations were carried out on the problems caused by irrigation with saline water and their management. On the other hand, no specific work has been done on farms layout and irrigation systems design associated with saline water and arid conditions. This study is based on large-scale field trials, design experience and observations on the use of saline water for irrigation in Qatar during the last two decades. In order to mitigate the effects of the factors causing salinization and waterlogging, modern techniques will be used in designing and operating irrigation systems. The emphasis here will be on proposing typical layout of farms; and pointing out guidelines for proper irrigation systems design, operation and better adaptability to saline waters, soils and crops.

MATERIALS AND METHODS

- Field investigations were carried out in selected farms where the performances of the irrigation systems were not satisfactory. Pertinent data were collected for analysis, problems were identified and viable solutions including re-design of the systems were proposed and implemented. The sprinklers overlapping for various wind velocities pointed out by Walker (1980) was used for evaluation and design of the sprinkler irrigation systems.

- The meteorological data were collected by Agro-meteorological Sub-section of the Department of Agricultural and Water Research (DAWR) from the three government stations located at north, center and south of Qatar.

- Use the uniformity coefficient (Cu) of Christiansen to estimate the uniformity of water application of the sprinkler irrigation systems.

- Water chemical analysis and soil physico-chemical analysis were performed by the Central Agricultural Laboratory of the DAWR.

- Penman-Monteith Method was used for estimation of evapotranspira tion and irrigation scheduling and calculations were aided by the computer program **CROPWAT,** which was developed by Martin Smith (FAO, 1992).

RESULTS AND DISCUSSION

Layout of Farms under Saline and Arid Conditions (Hashim, 1999):

The loop pipeline system

Evaluation of different farm layouts and irrigation systems and their operation indicates that a layout which is based on the loop pipeline system (circular main) is suitable for arid lands and saline conditions. First of all, it fits well in the irregular shapes of farms which are usually elliptical or approximately circular and thus, permits maximum use of arable lands. Secondly, the loop system can feed water to all cultivated fields, even with a partly defected pipeline or some broken pumps (Fig. 1). The loop system facilitates frequent irrigation, which is required when dealing with saline irrigation water.

The pumps, reservoirs and piping layout for different wells' water qualities, as depicted in Fig. 2, are proposed for proper practicing of mixed and alternate irrigation. Connection of all the wells by the loop pipeline helps in efficient reclamation of salt-affected fields by the good quality well water. Moreover, it makes easy the provision of better quality well water to plants which are sensitive to salts during specific stages of growth.

Close observation and experience in designing piping systems demonstrated that all plastic pipes: buried polyvinyl chloride (PVC) and polyethylene (PE) laid on the surface provide the best types of pipes for the on-farm irrigation systems under saline conditions.

Need for drains and protective embankments

Qatar, being a typical arid land is characterized by scanty rainfall of annual mean of 89.9 mm. The low intensity of rain and its erratic nature makes the provision of drain a minor factor in farm designing and its construction is rather an uneconomic item. However, the occasional heavy thunderstorms of high intensity, for example 137 mm fell in 5 hours on 12th March, 1995 (total annual 277 mm) lead to loss of yield and destructive effects in farm buildings. This event makes consideration of drains a necessity in design and layout of farms. The necessity arises from the fact that all farms in north and central Qatar are located in depressions.

Design of drains

This study 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. 3 protects the farm from the destructive effects of heavy thunderstorms and from salinization by salts carried by rain water from near-by lithosols (high rocky areas) and deposited on the fields. The design also makes provision for drainage of excess rain water which would otherwise remain on fields of heavy texture soils, causing waterlogging and destroying young plants. The drainage water can be directed to relatively low depressions and recharged artificially into the groundwater aquifer through specially designed recharge wells.

Some Important Aspects in Design and Operation of Irrigation Systems under Saline and Arid Conditions

In addition to the design procedure of irrigation systems which the hydraulic engineers usually consider, this study lays emphasis on some aspects related to saline and arid climate conditions that affect designing and operating of irrigation systems (Hashim, 1999).

Pumping test

The basic concept of irrigation under saline conditions in arid areas is to provide enough water to satisfy crop evapotranspiration and leaching

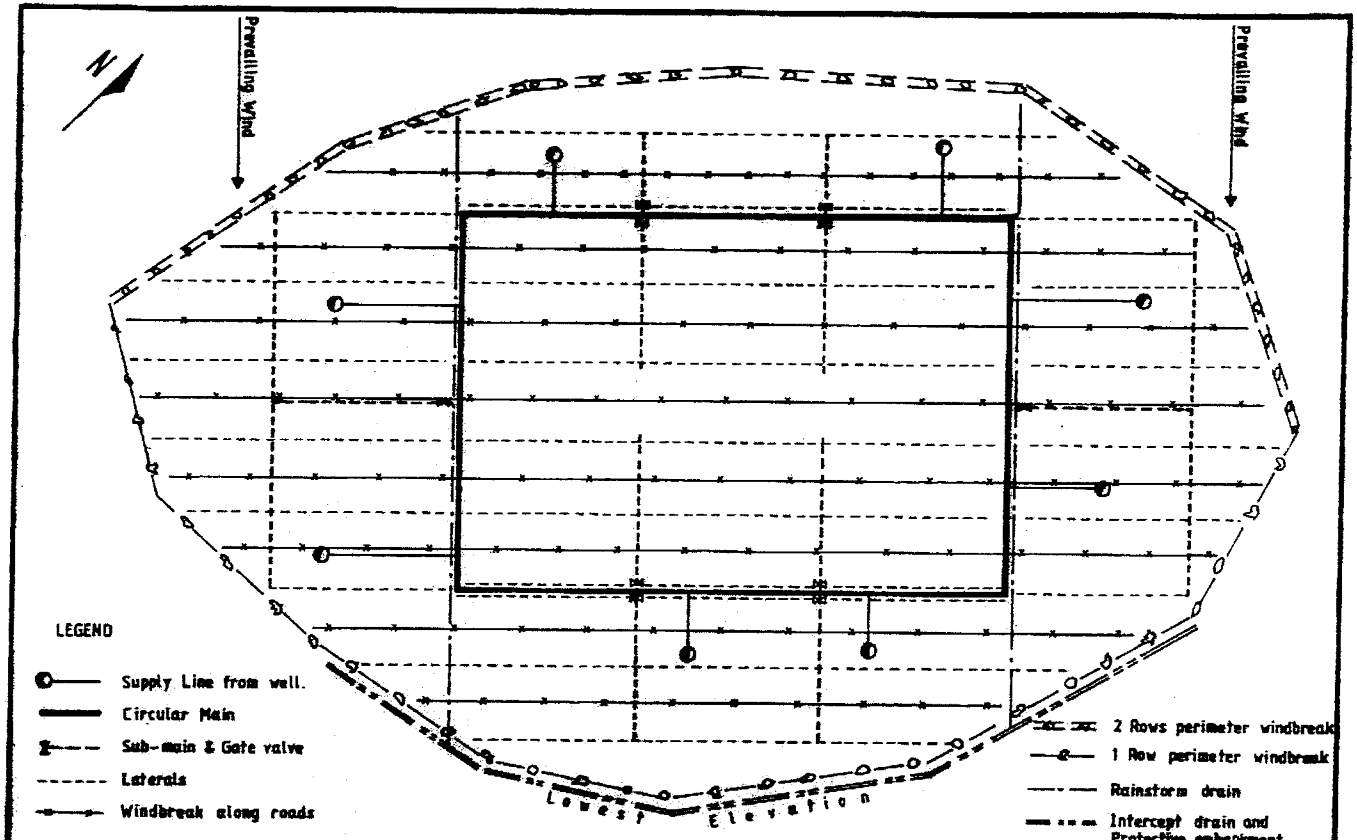

FIG.1 TYPICAL LAYOUT OF FARMS UNDER SALINE AND ARID CONDITIONS (NTS)

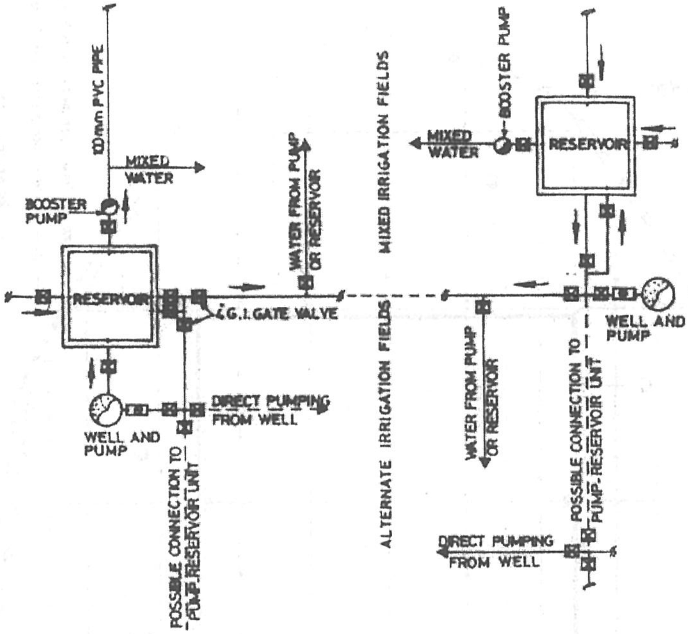

FIG-2: PUMPS-RESERVOIRS CONNECTIONS AND PIPING LAYOUT FOR DIFFERENT WELLS WATER QUALITIES (NTS)

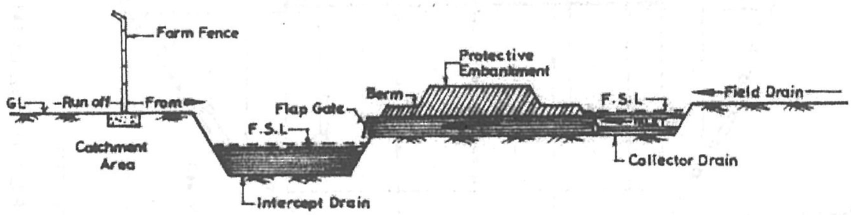

FIG-3: PROPOSED DRAINS AND PROTECTIVE EMBANKMENT FOR FARMS (NTS) requirements. In order to achieve this objective, pumping test (discharge versus drawdown) proved to be necessary. Investigations revealed that the main reason for the low pump discharge from wells was the wrong selection of pumps. Accurate pumping test is exceptionally necessary for adequate design of static head to avoid low pump discharge or no discharge. Most plants, under saline conditions can not tolerate drought periods because the excessive salt concentration of soil water retards or even ceases growth. On the other hand, over pumping at a rate exceeding 40 m^3/h in the Qatari farms, caused excessive drawdown which lead to sea water intrusion; the evidence being the deterioration of wells' water qualities and abandonment of farms near the coast.

Oversizing the main line

Because of the possibility of salt accumulation on soils with time and the need for reclamation, oversizing the main line and increasing the pump capacity is an advantage. Flexibility in the main line is also useful to offset the extra pressure loss from the rough inner surface of metal pipes caused by the corrosive effect of saline water.

Reasons for unsatisfactory crops growth

An overview of the unsatisfactory crops growth and yields under some irrigation systems revealed that their designs were hydraulically balanced; but no considerations were made regarding water quality, soil type and salt tolerance of crops. Chemical water analysis (including biological analysis in case of treated sewage effluent) is very important. Physico-chemical analysis of soil before planting and after harvest is required for proper integrated management of saline water and salt affected soils. To design and operate irrigation systems successfully we should consider the salt tolerance, rooting depth, leaves' types and orientation of the crops we plan to grow.

Operating hours of irrigation systems

The arid climate, characterized by high temperature, huge evaporation and strong desiccating winds, limits the operation hours of irrigation systems. The best flexible irrigation design should be based on 12 hours daily operation.

Establishing proper windbreaks (trees or date fronds) creates a micro-climate that retards wind speed, increases relative humidity and decreases evaporation and evapotranspiration. In such a case the daily operation of irrigation systems can be increased to 16 hours.

Irrigation scheduling

An example of the irrigation scheduling for squash and comparison between scheduled water requirements and actual consumption is shown in Table 1.

The result shows that there is a good agreement between the theoretical irrigation scheduling and the actual consumption of the crop. It is worth noting that irrigation schedule was sometimes subjected to modification due to rainfall, cloud cover, abnormal wind speed, temperature and humidity.

Wind speed and sprinklers spacing

Wind speed and direction is the most important climatic factor affecting the uniformity of distribution of the sprinkled water and the timing of irrigation in arid lands. The results of the uniformity coefficient (CU = 0.75-0.85) and observations of overlapping of water jets for wind speed up to 18 Km/h (5 m/s) imply that sprinkler spacing proposed by Walker (1980) and used in this study are applicable under saline and arid conditions.

Table 1: Irrigation scheduling based on evapotranspiration compared to actual crop water consumption

Farm: Rodhat Al Faras Government Farm
Crop: Squash **Variety:** Clarita
Date of sowing: 13-11-96 **Date of harvest:** 10-02-97
Method of irrigation: Drip
Season: 1996/97
Water quality: ECw = 4.5 dS/m LF* = 0.15 Le* = 85%
Soil Texture: Sandy clay loam

Month	Reference Evapo-transpi-ration ETo (mm)	Reduc-tion Factor	Crop Factor (Kc)	Crop Evapo-transpi-ration ETc (mm)	ETc x 1 / 1-LF Le (mm)	Actual Total Water Consumption (mm)	
						Irri-gation	Rain-fall
Nov.96							
13-20	24.6	0.30	0.40	3.0	4.2	8.5	
21-30	27.3	0.30	0.40	3.0	4.6	8.9	0.6
Dec.96							
1-10	23.3	0.50	0.50	5.8	8.1	8.8	
11-20	22.6	0.70	0.70	11.1	15.4	16.0	
21-31	22.8	0.80	0.80	14.6	20.2	19.7	
Jan.97							
1-10	20.0	0.90	0.90	16.2	22.4	16.6	16.0
11-20	21.8	1.00	0.90	19.6	27.2	27.6	
21-31	25.1	1.00	0.90	22.6	31.3	31.0	
Feb.97							
1-10	26.3	1.00	0.80	21.0	29.1	26.3	
TOTAL					162.5	163.4	16.6

*LF = Leaching fraction *Le = Leaching efficiency

The uniformity of application was greatly improved at Wadi Al Araig Experimental Station where both windbreak trees and laterals were re-arranged in the field perpendicular to the prevailing wind direction. However, when the field's shape does not permit such layout of laterals, installation of square spacing of sprinklers gives satisfactory uniformity coefficient.

In order to avoid the scorching effect of sprinkling water on leaves of windbreak or fruit trees located downwind, half circle sprinkler were used along the boundaries of the irrigated fields.

Clogging problem consideration in design of drippers

The main problem which faces large-scale application of drippers with saline water is the great potential for clogging. In addition to soil particles from well water and algae from open reservoir, chemical precipitation of iron and calcium carbonate add to the clogging problem. This problem, which was common in many farms, affected the uniformity of water application. Experience in design and operation of the drip irrigation system indicates that the following considerations are useful in mitigation of the clogging problem:

- Make direct connection to the well pump (Fig. 2) without need for storage and booster pump. This is possible because the operating pressure of dripper does not exceed 1.4 bars (20 psi) to give a discharge of 9.5 l/h. In such a case there will be no algae growth, a common problem in open reservoirs, and filtration cost will be reduced.

- An orifice dripper equipped with a self pressure compensating element and self flushing proved to be effective in reducing clogging problems.

- A complete water analysis, which is usually a pre-requisite for irrigation systems design under saline conditions, should include manganese, iron and quantity and diameter of suspended solids in case of drippers.

- The type and capacity of filters shall be based on the system flow, diameter of the suspended solids and type of drippers to be used.

• **Leveling consideration in design of surface irrigation system**

The most important step to consider in designing surface irrigation system under saline conditions is to prepare a complete topographic map, supplemented with contour line. This is necessary to make possible proper leveling of basins and grading of furrows. Practicing surface irrigation in unleveled land, leads to poor water distribution over the field, uneven plant growth, bare spots and eventually low yield.

- **Adaptability of irrigation systems to saline irrigation water**

Sprinklers

The adaptability of sprinklers to irrigation of crops is limited mainly by the level of water salinity (ECw) and concentration of B, Na and Cl in the irrigation water in addition to tolerance of crops to salts. Impact sprinklers were used successfully for irrigation of field crops, forages and bulb vegetables with water quality up to 6.5 dS/m. Sprayers attached to the center pivot lateral are used for irrigation of forages at Er Rakiyah Farm (ECw = 2.14-3.31 dS/m). The yields of onion (conventionally considered sensitive to salts) and potato (moderately sensitive to salts) were exceptionally high under sprinkler irrigation (ECw 5.3 dS/m, Na = 33.6 me/l & Cl = 41.0 me/l) and coarse sandy soil of Wadi Al Araig. On the other hand, squash (moderately tolerant to salts), cucumber and tomato (both moderately sensitive to salts) did not withstand the toxic effect of Na and Cl as shown by drying of leaves and subsequent defoliation.

Drippers

The result of field trials indicate that drip irrigation system, attaining maximum water use efficiency, is more favorable to growing multi fruit vegetables (squash, potato, cucumber and tomato) than other irrigation systems.

It was observed that the wild trees in the six afforestation blocks of the north, date palm trees and fruit trees in orchards did not respond well to drip irrigation system. The mortality percentage of the wild trees was more than 50% within few years.

Bubblers

The present healthy growth of 36000 different trees (acacia, zizyphus and date palm) after replacing drippers by bubblers in the afforestation blocks mentioned above together with the safe level of soil salinity proves that the bubbler irrigation system is more suitable for trees than the drip irrigation system.

The bubbler systems are very successful in irrigation of date palm trees at Rodhat Al Faras Farm and Al Mashabiyah Project, where the water salinity ranges between 4.3 and 9 dS/m.

CONCLUSIONS

The following conclusions are drawn from the results and findings of large-scale field trials, surveys, designs and observations on the use of saline water in Qatar (Hashim, 1999):

(i) Some important aspects that need special considerations in the design of irrigation systems under saline and arid conditions include topography, water quality, irrigation scheduling, leaching requirements, soil properties, salt tolerance of crops, wind speed and direction, number of daily operating hours and clogging of drippers.

(ii) Using the loop PVC piping system connecting wells of different water qualities, oversizing the main pipeline, increasing the pump capacity, establishment of windbreaks, provision of rainstorm drains and single embankment intercept drains improve layout of farms and design of irrigation systems and lead to better saline water management.

(iii) Using evapotranspiration as a guide for adequate irrigation scheduling under saline conditions, results in saving water and reducing the hazards of salinization and waterlogging. Penman-Monteith method attains reasonable level of accuracy and the scheduled irrigation data compare well with the actual water consumption of the tested crops. Such data represent a good reference for design and operation of irrigation systems.

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Bioaccumulation of Pollutants Produced from Chemical Fertilizer Industry by Yeast Hybrids and Bacterial Transconjugants

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BIOACCUMULATION OF POLLUTANTS PRODUCED FROM CHEMICAL FERTILIZER INDUSTRY BY YEAST HYBRIDS AND BACTERIAL TRANSCONJUGANTS

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ABSTRACT

Nile river water in Egypt is concerned very much with the pollution of factory effluents. This is a major global health hazard. The potential of pollutants recovery from these effluents was examined using microbial biomass of genetically constructed strains. The results have shown that the hybrid yeast cells can successfully recover higher percent of total pollutants in the presence of sugarcane refuse other than that absorbed without it. Slightly more cobalt than cadmium was accumulated by the hybrid yeast cells. In the parental strains of bacteria, using sugarcane refuse, extremely high arsenic and cobalt - absorbing ability was found in *Micrococcus halobius* B and *Bacillus cereus*, respectively. Bacterial transconjugants were found to accumulate more abundantly of arsenic and total pollutants in the absence of sugarcane refuse. Parental strains of bacteria used in this fundamental research shows more tolerant to higher temperature - industrial effluent stress. The relative increase in nitrite uptake by transconjugants was greater than nitrate, although, the relative of arsenic removal was greater than that in cadmium and cobalt. The results showed variability in protein banding patterns among bacterial strains and their transconjugants according to genetic background. This paper deals with the genetic construction of microbial strains to maximize the process of pollutants adsorption - desorption from the effluents by biological material.

Key words: Auxotrophs - electrophoretic patterns - prototrophs - relative increase - temperature tolerance - uptake of pollutants.

INTRODUCTION

Modern society is facing an increasing number of environmental problems. In the past decade, environmental biotechnology has developed as an offshoot from civil engineering via sanitary engineering. Environmental biotechnology is defined as a discipline which studies the application of biotechnology to solve environmental problems for potable water production, wastewater purification, solid waste treatment, soil and sediment clean-up, and air and off-gas treatment. The emphasis is on the "bio" as much as on the "technology".

The removal of radionuclides, metal or metalloid species, compounds and particulates from solution by biological meterial, particularly by non-directed physico-chemical interactions, is now frequently termed "biosorption" (Gadd, 1990a). Although virtually all biological material has biosorptive properties (Macaskie and Dean 1990), most work 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 (Macaskie, 1991). Furthermore, appropriate treatment of loaded biomass can enable recovery of valuable elements for recycling or further containment (Brierley, 1990; Winkelmann and Wing, 1994).

Cadmium, for example, is a nonessential heavy metal (Miettinen, 1975) and a powerful enzymatic inhibitor (Lockwood, 1976), it has been considered as an extremely significant pollutant due to its high toxicity and great solubility in water which determines a wide distribution in aquatic ecosystems. Cadmium causes point mutations in *Bacillus subtilis*, *E. coli*, *Salmonella thyphimurium*, DNA Single-strand breaks in *E. coli* and DNA polymerase activity inhibition in *E. coli* and in humans (Rosas *et al.*, 1984). Fore toxicity of heavy metals, it has been shown in the past that carcinogenic and toxic chemicals relate to each other in a straight proportional relationship, the highly carcinogenic chemicals also being highly toxic (Suess, 1995; Ashicla, 1965; Ehrlich, 1981 and Hashem, 1995). The present investigation emphasizes the practical evidence for microbial biotechnology as a feasible alternative to existing treatment methods for the removal and/or recovery of nitrite, nitrate and heavy metals from liquid industrial wastes.

MATERIALS AND METHODS

Genetic material

The strains employed in all experiments in this work were constructed and described previously by Kosba and Zaied (1997). These strains were listed in Tables A, B, C and D.

Culture

Media for growing microorganisms and other culture conditions have been described previously by Horikoshi *et al.* (1981). Precultured cells were used for following uptake experiments.

Table (A): *Saccharomyces cerevisiae* strains.

Strain	Genotype	Designation
YNN282	α *trp 1-Δ his3 - Δ 200 ura3 - 52 lys 2 - 801 a ade 2 - 10 gal mal CUP*^{r}	108
IL 126 – 7A	a ura1 $P^{+}W^{-}$ $C321^{R}$ $E221^{R}$	116
2780 – 96	a *ade1 leu1 ura2 rad1 can1*-100 E^{R}	120

Table (B): Registry number of heterozygous diploids from yeast.

Hybrid No.	Derivation and (or) reference	Designation
5	108 x 116	H_5
6	108 x 116	H_6
7	108 x 116	H_7
8	108 x 116	H_8
9	108 x 116	H_9
10	108 x 116	H_{10}
17	108 x 120	H_{17}
18	108 x 120	H_{18}

Table (C): Bacterial strains isolated and used in this study.

Strain	Source of isolates	Designation
Micrococcus halobius(A)	Infected banana fruits	1
Micrococcus halobius (B)	Infected banana fruits	2
Bacillus cereus	Chicken feathers	3
Micrococcus luteus	Chicken feathers	4
Micrococcus lylae	Wastewater from Talkha Fertilizer Factory	5
Bacillus subtilis	Infected banana fruits	6
Bacillus licheniformis	Chicken feathers	7

Table (D): Bacterial transconjugant strains.

Matings	Selectable markers in transconjugants	Designation
1 (*hico*^{s} *kan*^{r}) x 6 (*hico*^{r} *kan*^{s})	*hico*^{r} *kan*^{r}	10
2 (*myco*^{s} *kan*^{r} *hico*^{s}) x 3 (*myco*^{r} *kan*^{s} *hico*^{r})	*myco*^{r} *kan*^{r} hico^r	11
2 (*myco*^{s} *kan*^{r}) x 4 (*myco*^{r} *kan*^{s})	*myco*^{r} *kan*^{r}	12
1 (*kan*^{r}) x 7 (*kan*^{s})	*kan*^{r}	13
1 (*kan*^{r} *hico*^{s}) x 3 (*kan*^{s} *hico*^{r})	*kan*^{s} *hico*^{r}	14
5 (*myco*^{s} *eryth*^{r}) x 7 (*myco*^{r} *eryth*^{s})	*myco*^{r} *eryth*^{r}	15
2 (*myco*^{s} *kan*^{r}) x 7 (*myco*^{r} *Kan*^{s})	*myco*^{r} *kan*^{r}	16
1 (*kan*^{r}) x 4 (*Kan*^{s})	*kan*^{r}	17
4 (*myco*^{r} *eryth*^{s}) x 5 (*myco*^{s} *eryth*^{r})	*myco*^{r} *eryth*^{r}	18
3 (*myco*^{r} *hico*^{s} *eryth*^{r}) x 5 (*myco*^{s} *hico*^{s} *eryth*^{r})	*myco*^{r} *hico*^{r} *eryth*^{r}	19
2 (*myco*^{s} *kan*^{r} *hico*^{s}) x 6 (*myco*^{r} *kan*^{s} *hico*^{r})	*myco*^{r} *kan*^{r} *hico*^{r}	20
5 (*myco*^{s} *hico*^{s} *eryth*^{r}) x 6 (*myco*^{r} *hico*^{r} *eryth*^{s})	*myco*^{r} *hico*^{r} *eryth*^{r}	21

r = Resistance   s = Sensitive.
Markers: kan = Kanamycin   myco = Nystatin
hico = Amoxicillin trihydrate
eryth = Erythromycin

Industrial effluents

The present work was undertaken with the finishing industrial wastes resulted from the unit of ammonia at Talkha Fertilizer Factory (TFF), Dakhlia Governorate. Polluted water was collected from the main pipe of the factory before mixed with the effluents of water in the river. Industrial wastes was collected through January and June from 1995 and 1996.

Uptake experiments

In the heavy metals uptake test, precultured cells were suspended in 250 ml conical flasks containing 100 ml minimal medium of yeast (Eckardt and Haynes, 1977) and bacteria (Vogel and Bonner, 1956) supplemented with factory effluents instead of distilled water and incubated under a static conditions at 30°C for six days. Thereafter, the cells were collected by filtration on membrane filter (pore size 0.45 mm) and the other by centrifugation. Amounts of metals taken up by the cells were determined by measuring metal contents in the filtrate using Atomic Absorption Spectrometer at Chemistry Dept., Faculty of Science, Mansoura University according to Nakajima and Sakaguchi (1986).

Nitrite and nitrate assays

In this procedure, each water sample was divided into two equal aliquots. The first aliquot was analyzed for No^{-}_{2} by a modification of the Griss II sovay method, which was described by Bremmer (1965). In this analysis, the aliquot was treated with diazotizing reagent (sulfanilamide) in Hcl solution to convert the No^{-}_{2} to a diazonium salt, and it is subsequently treated with a coupling reagent (N - [1-naphthyl ethylene diamine di hydrochloric acid) to convert the diazonium salt to an azo compound. The intensity of the reddish purple color that develops as a result of these treatments is then measured. The second aliquot was analyzed for No^{-}_{2} + No^{-}_{3} by reducing the No^{-}_{3} to No^{-}_{2} using a Zn metal powder according to Heans (1975). Then it was determined by the previous method for the No^{-}_{2}. The concentration of No^{-}_{3} was calculated by difference.

Temperature tolerance

Cell concentrations of strains, grown with distilled water and (or) with industrial effluents in a growing medium, were determined from turbidity measurements by absorbance at 450 nm according to Trumbly and Bradley (1983) and Novotny and Lavin (1971) at temperatures of 30, 33, 36, 39 and 42°C.

Relative increase

The relative increase by different transconjugants in absorbance of nitrite, nitrate and heavy metals was expressed as a marked increase in ratio percent of transconjugant to the mid parents over one hundred percent according to Singleton *et al.* (1982).

Genetic stability assays

Bacterial strains were grown aerobically for thirty days without shaking at 30°C in nutrient broth containing factory effluents instead of distilled water. At the late of incubation period, appropriate dilutions were plated on nutrient agar plates and incubated at 30°C for 3 days. Colonies were directly picked up and replicated on minimal and complete medium, and auxotrophs were scored that only growing on complete medium but not on minimal medium. These were identified as auxotrophic mutants according to the method fully described previously by Mergeay and Cerits (1978).

Electrophoretic analysis

(a) Extraction of soluble proteins

Extracts were prepared for electrophoretic analysis according to Laemmli (1970).

(b) Gel electrophoresis

A 10% polyacrylamide slobe gel was prepared by mixing 13.3 ml of 30% acrylamide containing 0.8% bisacrylamide, 10 ml of 1.5 M Tris Hcl buffer (pH 8.8), and 15.9 ml of deionized water. After degassing, 0.4 ml of ammonium persulfate (100 mg ml^{-1}), and 20 µlN, N, N, N, tetramethylethylenediamine (TEMED) were added. Stocking gel was prepared as above except that 1.7 ml of acrylamide, 8.0 ml distilled water and 0.5 M tris Hcl buffer (pH 6.8) were used. The running buffer (pH 8.3) consisted of 14.4 gm of glycine, 3.0 gm tris, and deionized water to make one liter. 20 µl of buffer was added to each protein sample, this buffer containing 0.5 M tris Hcl (pH 6.8), glycerol 10%, B-mercaptoethanol, 5% bromophenol blue 2.5% and distilled water and applied to the gel. A voltage of 150 V at 48 mA was applied for five hours. After electrophoretic separation, the gel was stained for 30 min at room temperature with coomassie brilliant blue (CBB) R250 (CBB 0.125 gm, $CuSo_{4y}$ 0.125 gm in 31 ml methanol, 25 ml glacial acetic acid and 69 ml water) and destined for 24 h with solution containing 20 ml methanol, 20 ml glacial acetic acid and 160 ml water. The standard proteins in parental strains were used as references.

RESULTS AND DISCUSSION

Accumulation of pollutants by hybrid cells of yeast

The cells of eight different hybrids of *Saccharomyces cerevisiae* resulted from two different crosses were suspended in minimal medium containing industrial effluents with (B) and without (A) sugarcane refuse. The results obtained with representative hybrids for nitrite, nitrate and heavy metals uptake are illustrated in Figure (1). From the results it can be seen that the total amounts of pollutants absorbed by the hybrid yeast cells in the presence of sugarcane refuse (sucrose) as a sole carbon source instead of glucose were greater than without it in a treatment solution. A similar range in absorption ability was observed for nitrite, cadmium and arsenic in the presence of sugarcane refuse than the uptake observed without it. Extremely high total pollutants - absorbing ability was observed in the presence of sugarcane refuse. It was shown in the earlier papers (Skowronski, 1984a and b) that *Stichococcus bacillaris* takes up cadmium by adsorption (a fast phase) and energy-dependent transport (a slow phase). Cadmium transport into cells is completely inhibited at 4°C and increases with temperature increase up to 35°C (Skowronski, 1986). As shown herein from Fig. (1) slightly more cobalt than cadmium was accumulated. The present results are in accordance with those reported by Brady *et al.* (1994), who found that cadmium was accumulated to a greater extent than either cobalt by yeast. Their results indicated that protein is a heavy metal accumulating component. Their data also indicated that the outer mannan-protein layer of the yeast cell wall is more important than the inner glucan-chitin layer in heavy metal cation accumulation. This also are in agreement with those reported by Dedyukhina and Eroshin (1991), who demonstrated that there is linear relationship between the amount of certain heavy metals accumulated within a cell and the cell protein content.

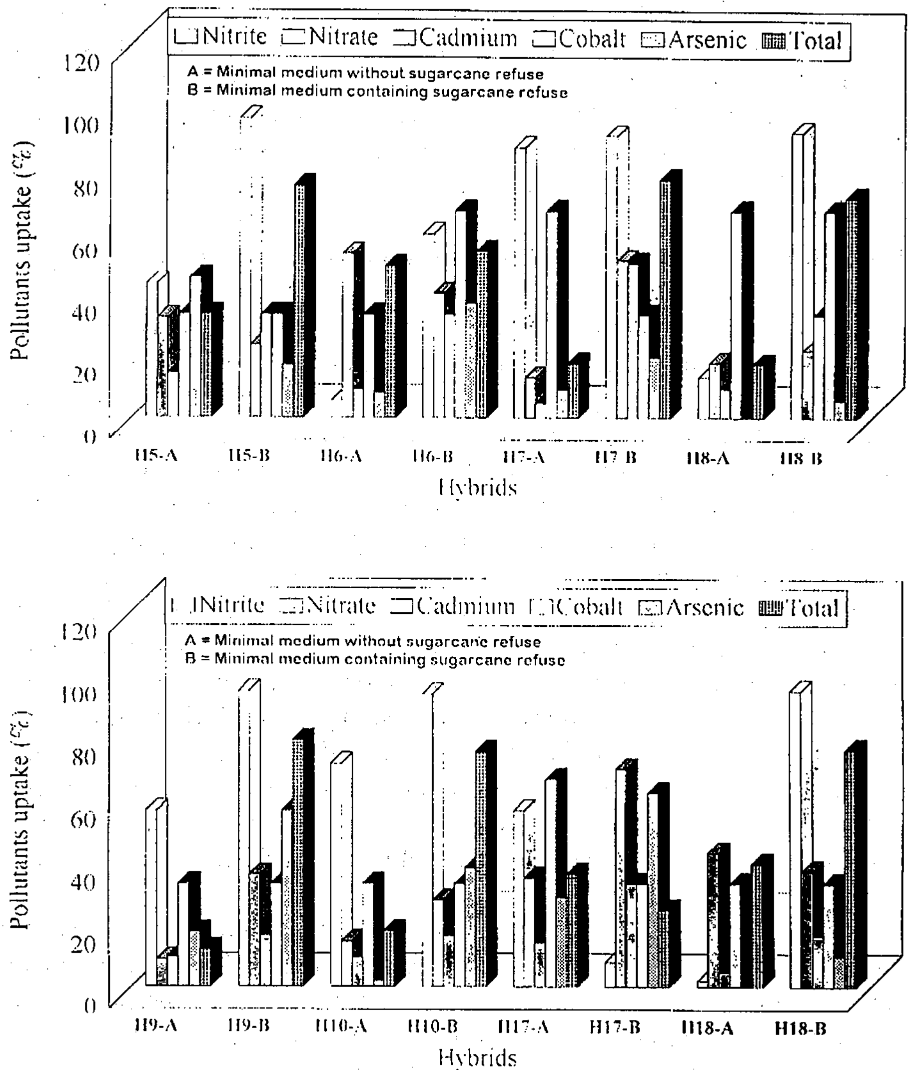


Fig. 1 Absorption capacity of pollutants from factory effluents by heterozygous diploids of Saccharomyces cerevisiae.

The results indicated that all of hybrid yeast cells using in this study revealed positive hybrid vigor for their tolerance to industrial effluents and uptake of heavy metals in their content than either their parental strains which failed in their tolerance to these effluents and unable to grow in the medium containing it. It is interesting to note that in the present study the heterozygotes are, on the whole, better buffered against environmental change than are homozygotes (Dawson, 1966). This is because the heterozygote has two alleles at many loci, and thus is able to maintain more homeostatic development when the environment fluctuates (Wills and Nichols, 1969). Recently, Sakai *et al.* (1988) reported that the yeast *Saccharomyces cerevisiae* is a good host for heterologous gene expression and protein secretion. It has been suggested that metallothioneins (MT), which are small cysteine-rich polypeptides that can bind essential metals, e.g. Cu and Zn as well as non-essential metals like Cd, have been recorded in all microbial groups such as bacteria and yeasts (Winge *et al.*, 1989) may be of potential in metal recovery since it can bind other metals besides Cu, e.g. Cd, Zn, Ag, Co and Au, although these metals do not generally induce MT synthesis (Butt and Ecker, 1987).

Biosorption of pollutants by parental and bacterial transconjugants

The results diagrammatic with representative parental strains of bacteria are shown in Figure (2). From this Figure it can be seen that the amounts of pollutants absorbed by the parental cells differs markedly in different strains of bacteria. Of special interest to this discussion is the wide range in the effectiveness with which different species and genera of bacteria absorb pollutants. Amongst these strains there are many species with a high ability for arsenic and total pollutants uptake, extremely high arsenic and cobalt-absorbing ability was observed in *Micrococcus halobius* B and *Bacillus cereus*, respectively, in the presence of sugarcane refuse. The present results are in agreement with those obtained by Tsezos *et al.* (1989), who found that the immobilized microbial biomass can successfully recover all of the uranium from dilute (less than 300 mg U/L) solutions. It has been shown in the literature that the immobilization of the microbial biomass into particles of a desirable size, mechanical strength and biosorptive characteristics is the best way to apply the process of biosorption for metal value recovery from process or waste solutions (Kiff and Little, 1986).

As shown in Figure (3) a similar range in absorption ability was observed in transconjugants. Of the transconjugants tested extremely high total pollutants uptake and arsenic-absorbing ability was observed without the presence of sugarcane refuse in solutions. All transconjugants tested accumulated different amounts of metal ions from a solution containing the effluents. This suggests that the selective accumulation of heavy metal ions by transconjugants is determined by interionic competition (Nakajima and Sakaguchi, 1986). The

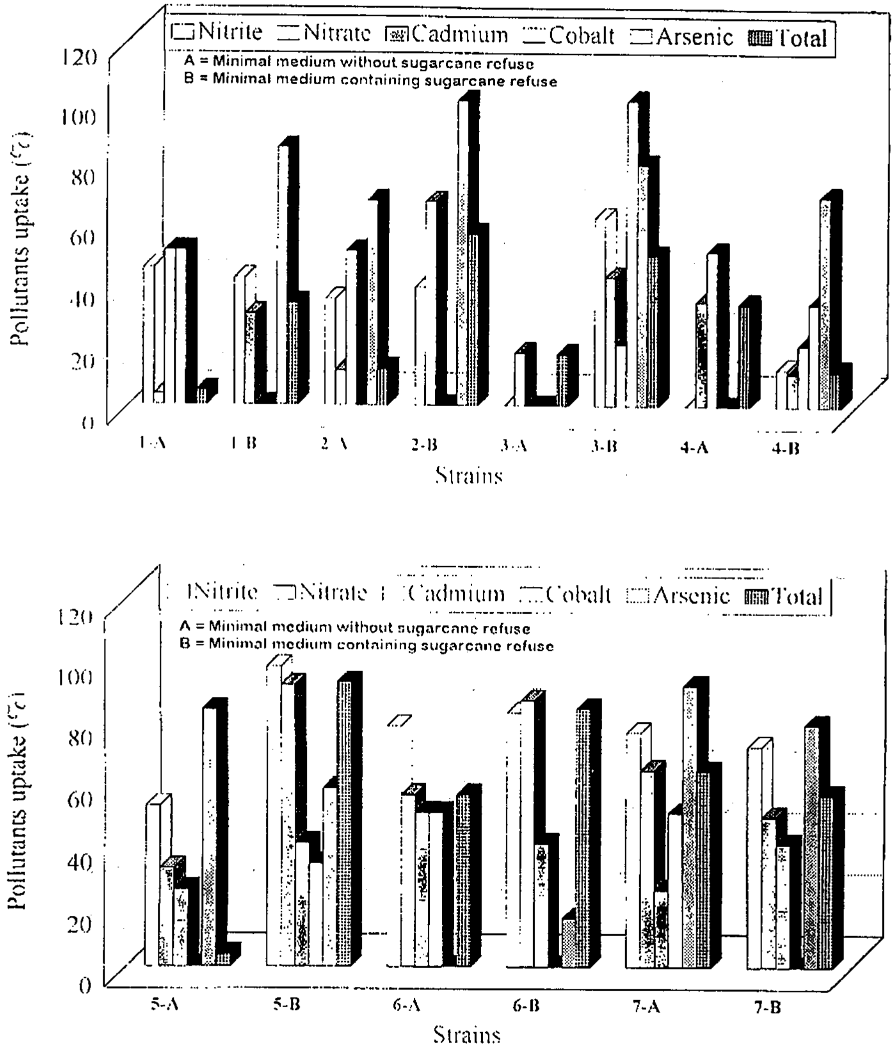
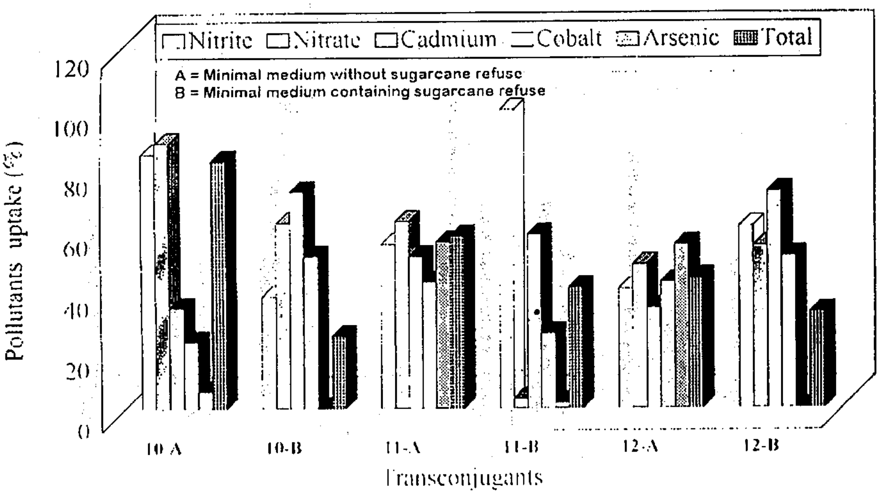


Fig. 2 Absorption capacity of pollutants from factory effluents by parental strains of bacteria.

present results are in accordance with those obtained by Nakajima and Sakaguchi (1986), who found that the relationship between the uptake of uranium and absorption of mercury is not the same in all groups of microorganisms and the total quantity of metal ions absorbed by microbial cells differed greatly from species to species. They also found that in bacteria and yeasts many species were found to accumulate mercury more abundantly than uranium. Our results showed that transconjugant cells had better and excellent adsorbing characteristics. Successful biotechnological exploitation of microbial metal accumulation may depend on the ease of metal recovery and biosorbent



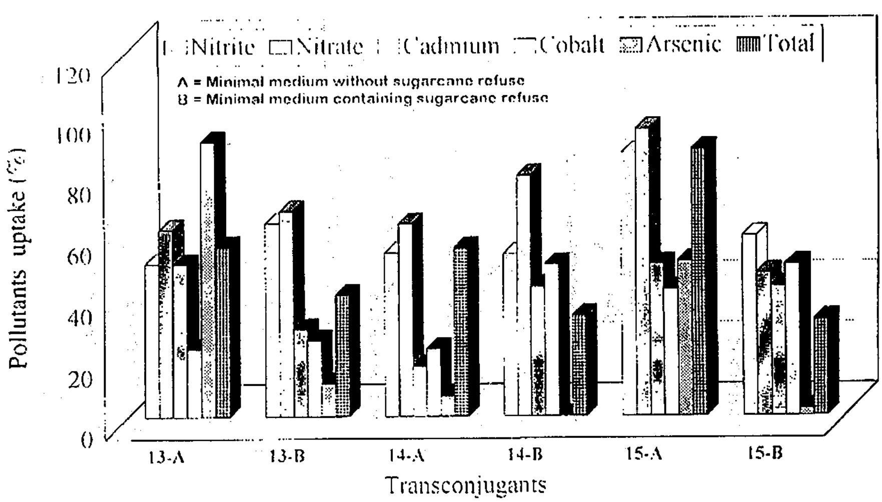
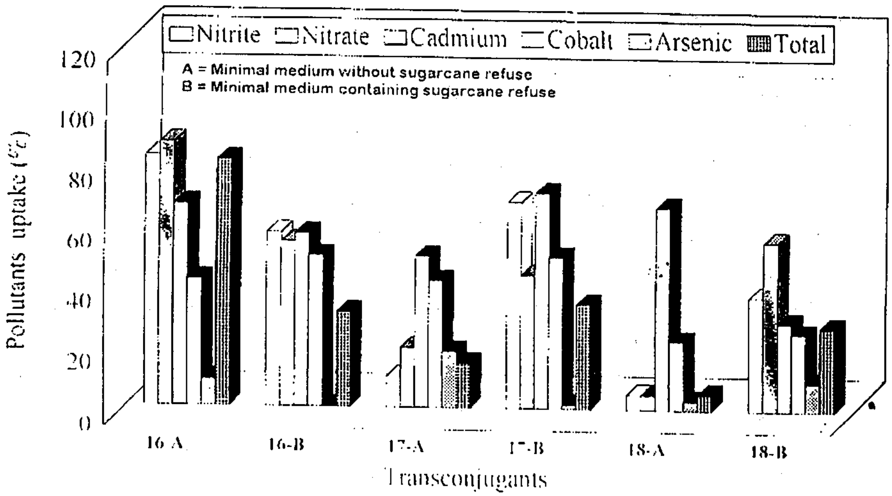


Fig. 3 absorption activity of pollutants from factory effluents by bacterial transconjugants.

regeneration for use in multiple biosorption-desorption cycles (Volesky, 1990). The mechanisms used for metal recovery from loaded biomass depends on the ease of removal from the biomass and this can depend on the element involved and the mechanism of accumulation. Metabolism-independent biosorption is frequently reversible by nondestructive methods and may often be considered analogous to an ion exchange process. Metabolism-dependent accumulation and intracellular compartmentation or sequestration within organelles or by binding to induced proteins etc., is often irreversible requiring destructive recovery. If a cheap and plentiful supply of waste biomass is used to recover valuable metals then the economics of destruction may be satisfactory. This has been demonstrated that it may be possible to apply selective



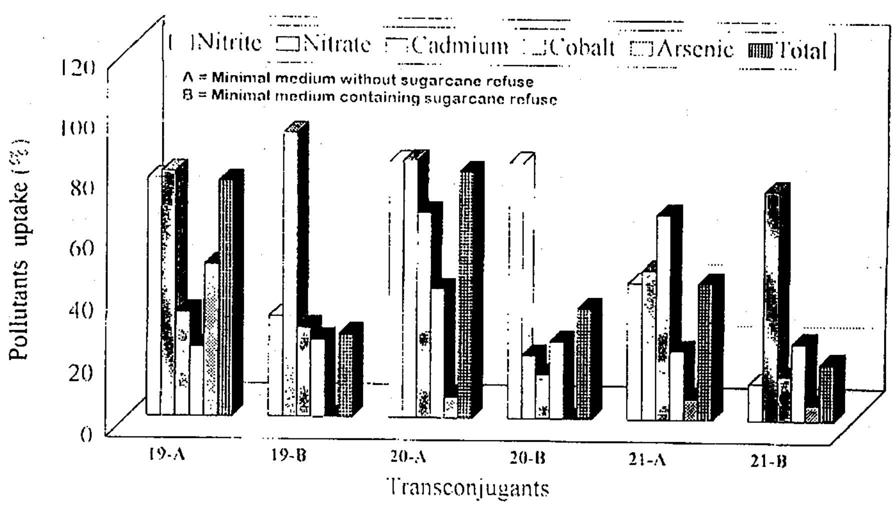


Fig. 3 Continued.

desorption of chosen elements from a biosorbent loaded with a number of different elements with an appropriate choice of eluant. Industrial application of biosorption depends on such factors as loading capacities, efficiencies and selectivity, ease of metal recovery and an equivalence at least to traditional physical and chemical treatments in performance (Brierley, 1990). In comparison with existing treatment methods, several biosorptive processes after advantages including high efficiency at low metal/radionuclide concentrations (Volesky, 1990). Furthermore, microbial biomass may be supplied as a fermentation by-product or specifically grown using cheap substrates. Many biosorbents appear competitive in cost with ion exchange resins while frequently exhibiting higher efficiencies (Volesky, 1990).

Effect of temperature on growth

The addition of factory effluents to nutrient broth did not inhibit the growth of different strains used in biosorption of pollutants (Tables 1, 2, 3 and 4). *Micrococcus lylae* and *Bacillus cereus* were not consistently similar tolerant to higher temperature stress (42°C) than were other isolates in the presence of industrial effluents. All selected four strains from the growth rate experiment, for temperature-industrial effluents-tolerant growth, were able to survive the initial exposure to factory effluents with an temperatures. The viability of the tolerant strains was maintained in all of the treatment solutions for the duration of the experiment. Some of strains grew slightly when exposed to high extreme temperature. The temperature tolerance of each strain may differ. It is practical, therefore, to examine the tolerance of effluent in relation to the tolerance of temperature. Slow-growing strains were not more tolerant to effluent-temperature than were fast-growing strains. *Micrococcus lylae* isolated from industrial effluent were not consistently more tolerant to temperature-effluent than were isolates from non-effluent conditions. Our results agree with those of Singleton *et al.* (1982), who found that isolates from saline environments were not consistently more tolerant to salt than were isolates from non-saline soils. The results of the experiments described above emphasize that these strains belonging to genera *Micrococcus* and *Bacillus* not only can withstand the industrial effluents but they even grow at high temperatures. The present results are in accordance with those reported by Singleton *et al.* (1982), who found that four strains of *Rhizobium* selected from the growth rate study could survive in extremely high concentrations of salt, and the salt-tolerant strains survived for more than 65 days with no reduction in viable

Table (1): Growth rate of *Micrococcus halobius* as a function of temperature tolerance with two different water samples.

Time	Temperature (^0C)									
	Distilled water					Industrial effluents				
	30	33	36	39	42	30	33	36	39	42
24	0.54	0.50	0.53	0.54	0.27	1.18	1.10	1.13	1.13	0.47
48	0.93	0.79	0.99	0.82	0.41	1.24	1.22	1.22	1.21	0.75
72	1.008	1.02	1.25	1.02	0.50	1.32	1.34	1.32	1.29	0.97
96	1.28	1.24	1.30	1.03	0.60	1.38	1.42	1.41	1.42	1.11
120	1.55	1.35	1.43	1.18	0.67	1.51	1.53	1.60	1.46	1.25
164	1.71	1.68	1.47	1.29	0.61	1.60	1.36	1.74	1.47	1.34
188	1.51	1.41	1.12	1.22	0.59	1.59	1.35	1.76	1.47	1.35
F-test	**	**	NS	**	NS	**	**	**	**	**
L.S.D. 0.05	0.21	0.16	0.63	0.34	0.44	0.07	0.09	0.17	0.15	0.20
L.S.D. 0.01	0.30	0.23	0.89	0.48	0.62	0.09	0.12	0.23	0.22	0.29

NS = Not significant ** = Significant at 0.01 probability level, based on F-test.

Table (2): Growth rate of *Bacillus cereus* as a function of temperature tolerance with two different water samples.

Time	Temperature (^0C)									
	Distilled water					Industrial effluents				
	30	33	36	39	42	30	33	36	39	42
24	0.40	0.47	0.75	0.42	0.15	0.04	0.02	0.83	0.33	0.10
48	0.59	0.88	0.81	0.49	0.55	0.56	0.46	1.03	0.87	0.23
72	1.02	1.01	1.14	0.97	0.41	1.10	0.86	1.08	1.02	0.31
96	1.47	1.38	1.25	1.13	0.43	1.50	1.30	1.10	1.29	0.57
120	1.42	1.39	1.29	1.36	0.39	1.57	1.35	1.13	1.18	0.71
164	1.41	1.32	1.28	1.29	0.55	1.64	1.52	1.19	1.18	0.59
188	1.29	1.25	0.93	1.27	0.54	1.45	1.40	1.35	1.16	0.49
F-test	**	**	**	**	**	**	**	*	NS	**
L.S.D. 0.05	0.17	0.23	0.26	0.54	0.10	0.29	0.18	0.25	0.78	0.17
L.S.D. 0.01	0.24	0.32	0.37	0.76	0.14	0.41	0.26	0.36	1.09	0.24

NS = Not significant
*, ** = Significant at 0.05 and 0.01 probability levels, respectively, based on F-test.

Table (3): Growth rate of *Micrococcus leteus* as a function of temperature tolerance with two different water samples.

Time	Temperature (^0C)									
	Distilled water					Industrial effluents				
	30	33	36	39	42	30	33	36	39	42
24	0.58	0.58	1.41	0.95	0.29	1.30	1.09	1.09	1.18	0.66
48	1.02	0.87	1.46	1.14	0.30	1.41	1.23	1.20	1.32	0.92
72	1.23	1.09	1.46	1.36	0.31	1.49	1.38	1.32	1.41	1.14
96	1.33	1.35	1.40	1.31	0.40	1.53	1.53	1.44	1.50	1.27
120	1.40	1.57	1.43	1.47	0.46	1.55	1.51	1.50	1.55	1.31
164	1.66	1.54	1.31	1.36	0.55	1.66	1.44	1.52	1.58	1.32
188	1.46	1.33	0.98	1.29	0.56	1.62	1.43	1.53	1.58	1.24
F-test	**	**	**	**	**	**	**	**	**	**
L.S.D. 0.05	0.20	0.22	0.21	0.12	0.06	0.10	0.10	0.09	0.10	0.19
L.S.D. 0.01	0.29	0.31	0.29	0.17	0.08	0.14	0.14	0.13	0.14	0.26

** = Significant at 0.01 probability level, based on F-test.

Table (4): Growth rate of *Micococcus lylae* as a function of temperature tolerance with two different water samples.

Time	Temperature (^0C)									
	Distilled water					Industrial effluents				
	30	33	36	39	42	30	33	36	39	42
24	0.44	0.13	0.43	0.55	0.04	0.29	0.29	0.31	0.11	0.06
48	0.91	0.38	1.02	0.29	0.52	0.30	0.73	0.87	0.29	0.18
72	1.20	1.44	1.28	0.60	0.44	0.91	1.07	1.43	0.29	0.31
96	1.65	1.23	1.48	0.88	0.81	1.11	1.43	1.52	0.48	0.56
120	1.70	1.20	1.49	1.01	0.90	1.22	1.41	1.33	0.66	0.68
164	1.78	0.96	1.68	1.09	1.07	1.57	1.41	1.32	0.93	0.68
188	1.60	0.87	1.48	1.15	1.16	1.27	1.29	1.19	0.94	0.63
F-test	**	**	**	*	**	**	*	**	NS	**
L.S.D. 0.05	0.14	0.16	0.39	0.50	0.50	0.41	0.72	0.49	0.24	0.35
L.S.D. 0.01	0.19	0.23	0.55	0.71	0.70	0.57	1.01	0.69	0.34	0.50

*, ** = Significant at 0.05 and 0.01 probability levels, respectively, based on F-test.

counts. The results of the experiments indicate that the strains selected for biosorption can grow and survive at high temperatures in the presence of industrial effluents which are inhibitory to many of microorganisms.

Relative increase in pollutants uptake

The correspondence between the relative increase of pollutants uptake and plasmid rescue transformation noted in Tables 5 and 6. In the absence of sugarcane refuse, the relative increase in nitrite uptake by bacterial transconjugant was greater than were nitrate, although, the removal of arsenic was greater than that in cadmium and cobalt. results indicated that most of transconjugants revealed a higher relative increase in pollutants uptake greater than 80%. The results obtained with representative transconjugants shows that the relative increase in pollutants uptake by bacterial cells differs markedly in different transconjugants. This is in agreement with the results of Porro *et al.* (1992), who found that fermentation studies with transformed yeast strains showed that the release of b-galactosidase allowed an efficient growth on buffered media containing lactose as carbon source as well as on whey-based media, the transformed strains utilized up to 95% of the lactose and a high growth yield was obtained in rich media. The process of pollutants removal depend on the ability of transconjugants to effect chemical transformations of heavy metals and their compounds by, e.g. oxidation, reduction, methylation and demethylation (Lovley *et al.*, 1991). Once inside cells, metal ions may be compartmentalized and/or converted to less toxic forms by, e.g. precipitation or sequestration by metal-binding proteins (Gadd, 1990b).

A similar range in absorption ability was observed in the presence of sugarcane refuse in solutions. When examined which of heavy metal ions can be most readily absorbed by transconjugants from a solution containing industrial effluents, the results indicated that extremely high relative increase was observed in nitrite than were in nitrate.The relative increase in arsenic uptake was greater than were in cadmium and cobalt.This is in accordance with the results obtained by Nakajima and Sakaguchi (1986),who found that uranyl, mercury, lead and copper ions were more readily accumulated by cell of bacteria, yeasts, fungi and actinomycetes than the other ions 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. On the other hand, Nakajima and Sakaguchi (1986) also found that cobalt accumulation from a mixed solution containing 9 different heavy metals was very poor, but all species tested by them accumulated large amounts of cobalt ion from a solution containing cobalt as the only metal ion. The results suggests that the selective accumulation of heavy metal ions by microorganisms is determined by interionic competition. This results are also in agreement with those reported by

Brierley (1990), who found that the granulated *Bacillus* preparation is non-selective and can remove many heavy metals from solution independent of differing initial concentrations, e.g. Cd, Cr, Cu, Hg, Ni, Pb, U and Zn, but does not bind Ca, Na, K or Mg. Single or mixed metals are generally loaded to > 10% of the dry weight giving a removal efficiency of > 99% and effluents with total metal concentrations around 10 - 50 ppb. Some of bacterial transconjugants in the absence of sugarcane refuse as well as of the present could oxidise As^{3+} to As^{5+}, this can improve certain arsenic removal methods since arsenate As^{5+}, is more easily precipitated from wastewater by Fe^{3+} than is arsenite, As^{3+} (Williams and Silver, 1984). Microbial transformations of arsenic and chromium species are also associated with a decrease in toxicity and may have relevance to wastewater treatment (Williams and Silver, 1984). On the other hand, microorganisms can transform heavy metal and metalloid species by reduction (Lovley *et al.*, 1991). Hansen *et al.* (1984) found that a continuous cultures of Hg^{2+} - resistant bacteria, can reduce Hg^{2+} to Hg^{o} with mercuric reductase, volatilized Hg^{o} from contaminated sewage at a rate of 2.5 mg L^{-1} h^{-1} (98% removal).

Industrial effluent mutagenesis

The data with respect of bacterial strains sensitivity to factory effluents are presented in Table (7). The results suggested that there was a lower percent of auxotrophic mutants induced. It must be noted that the sensitivity of *Micrococcus lylae* to these factory effluents was equal zero. This indicated that this strain have a higher genetic stability to industrial effluent stress than were other strains. The consistently more tolerant to factory effluent stress by this strain are due to the source of isolation from these effluents. The basis of adaptation by cells or organisms to mutagens or heavy metals is little understood. Metals such as cadmium, mercury, nickel and zinc are known to induce genotoxic adaptation through conditioning exposure against subsequent challenge exposure to mutagens, triethylenemelamine and maleic hydrazide in plant cells *in v* (Subhadra *et al.*, 1993). Furthermore, it has been experimentally shown that metal adaptation could be induced in one or two generations by growing normal plant populations on metal-contaminated soil (Walley *et al.* 1974). The results of the present study also indicated that strain of *Micrococcus lylae* was more resistant to metal exposure than other used strains. This is in accordance with those found by Subhadra *et al.* (1993), who found that seeds of *Hordeum vulgare* L. with residual mercury were more resistant to mutagen and metal exposure, than normal seeds.

Table (5): Relative increase in pollutants uptake from industrial effluents in the absence of sugarcane refuse.

Transcon-jugants	Items	Nitrite	Nitrate	Cadmium	Cobalt	Arsenic	Total
10	MP	0.098	3.6525	0.04	0.06	+0.0095	3.841
	TC	13.30	10.048	0.02	0.08	0.06	23.508
	RI	+13471.43	+175.10	-50.0	+33.33		+512.03
11	MP	+0.03	1.765	0.02	0.00	0.002	1.75825
	TC	8.543	7.0535	0.03	0.15	0.57	16.3465
	RI		+299.63	+50.0		+28400.0	+829.70
12	MP	+0.058	2.79	0.04	0.00	+0.0015	2.7705
	TC	6.25	5.431	0.02	0.15	0.56	12.411
	RI		+94.66	-50.00			+347.97
13	MP	0.0965	4.155	0.03	0.06	0.0005	4.3421
	TC	7.95	7.038	0.03	0.08	0.94	16.038
	RI	+8138.34	+69.39	0.00	+33.33	+187900.0	+269.36
14	MP	0.02275	1.275	0.02	0.03	0.007	1.29725
	TC	8.527	7.256	0.01	0.08	0.07	15.943
	RI	+37381.3	+469.10	-50.0	+166.67	+900.0	+1128.98
15	MP	0.1026	5.90	0.02	0.03	0.0105	6.0632
	TC	13.706	10.829	0.03	0.15	0.53	25.245
	RI	+13258.7	+83.54	+50.0	+400.0	+4947.62	+316.36
16	MP	0.0885	4.645	0.03	0.03	0.0095	4.8031
	TC	13.129	9.971	0.04	0.15	0.09	23.38
	RI	+14735.03	+114.66	+33.33	+400.0	+847.37	+386.77
17	MP	0.05	2.30	0.04	0.03	0.0105	2.3095
	TC	1.571	2.234	0.03	0.15	0.19	4.175
	RI	+3042.0	-2.87	-25.0	+400.0	+1709.52	+80.78
18	MP	+0.0439	4.045	0.03	0.00	0.001	4.0306
	TC	0.749	0.534	0.04	0.08	0.03	1.433
	RI		-86.80	+33.33		+2900.0	-64.45
19	MP	+0.01665	3.02	0.01	0.00	0.003	3.01835
	TC	12.255	9.128	0.02	0.08	0.51	21.993
	RI		+202.25	+100.0		+16900.0	+628.64
20	MP	0.09	4.1425	0.04	0.03	+0.001	4.302
	TC	13.222	9.627	0.04	0.15	0.07	23.109
	RI	+14591.11	+132.40	0.00	+400.0		+437.17
21	MP	0.1235	7.3775	0.03	0.06	0.001	5.5621
	TC	6.999	5.541	0.04	0.08	0.07	12.73
	RI	+5567.21	-24.89	+33.33	+33.33	+6900.0	+128.87

MP = Mid parents TC = Transconjugant RI = Relative increase (%).
+ (MP or TC) = Increase in concentration.

Table (6): Relative increase in pollutants uptake from industrial effluents in the presence of sugarcane refuse.

Transcon-jugants	Items	Nitrite	Nitrate	Cadmium	Cobalt	Arsenic	Total
10	MP TC RI	3.158 5.7565 +82.28	5.692 3.6478 -35.91	0.02 0.05 +15.00	0.00 0.14	0.0095 0.00 0.00	8.8795 9.5943 +8.05
11	MP TC RI	2.5345 15.309 +504.02	5.2905 0.1996 -96.23	0.01 0.04 +300.0	0.09 0.07 -22.22	0.017 0.33 +1841.18	7.942 15.9486 +100.81
12	MP TC RI	1.287 9.291 +621.91	3.788 3.2009 -15.50	0.01 0.05 +400.0	0.03 0.14 +366.67	0.016 +0.87	5.131 11.8119 +130.21
13	MP TC RI	2.885 9.831 +240.76	3.84 4.031 +4.97	0.02 0.02 0.00	0.00 0.07	0.0155 1.97 +12609.68	6.7605 15.922 +135.52
14	MP TC RI	2.612 8.293 +217.50	3.488 4.7334 +35.71	0.01 0.03 +200.0	0.09 0.14 +55.56	0.0155 0.17	6.2155 13.0264 +109.58
15	MP TC RI	4.32495 9.206 +112.86	6.875 2.8177 -59.02	0.04 0.03 -25.0	0.03 0.14 +366.67	0.013 0.43 +3207.69	11.28295 12.6237 +11.88
16	MP TC RI	2.8075 8.916 +217.58	5.6425 3.2647 -42.14	0.02 0.04 +100.0	0.00 0.14	0.017 +0.67	8.487 11.6907 +37.75
17	MP TC RI	1.3645 10.579 +675.30	1.9855 2.6262 +32.27	0.01 0.05 +400.0	0.03 0.14 +366.67	0.0145 0.23 +1486.21	3.4045 13.6252 +300.21
18	MP TC RI	2.80445 5.7565 +105.26	5.0205 3.3285 -33.70	0.03 0.02 -33.33	0.06 0.07 +16.67	0.012 1.63 +13483.33	7.927 10.805 +36.31
19	MP TC RI	4.05195 5.0095 +23.63	6.523 5.5635 -14.71	0.03 0.02 -33.33	0.12 0.07 -41.67	0.013 +0.87	10.73795 9.793 -8.80
20	MP TC RI	3.0805 12.948 +320.32	7.4945 1.2213 -83.70	0.02 0.01 -50.0	0.00 0.07	0.011 +1.07	10.606 13.1793 +24.26
21	MP TC RI	4.59795 1.8505 -59.75	8.727 4.478 -48.69	0.04 0.01 -75.0	0.03 0.07 +133.33	0.007 0.93 +13185.71	13.40195 7.3385 -45.24

MP = Mid parents TC = Transconjugant RI = Relative increase (%).
+ (MP or TC) = Increase in concentration.

Table (7): Genetic effects of industrial effluents on different bacterial strains used in biological therapy.

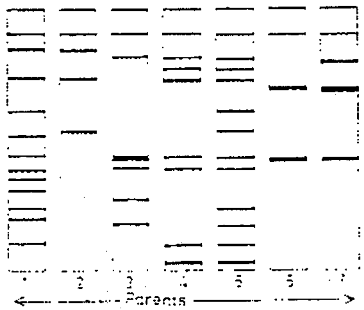
Strains	Prototrophs † (%)	Auxotrophs induced (%)
Micrococcus halobius A	94.12	5.88
Micrococcus halobius B	96.30	3.70
Bacillus cereus	96.30	3.70
Micrococcus luteus	90.63	9.37
Micrococcus lylae	100.00	0.00
Bacillus polymyxa	88.24	11.76
Bacillus licheniformis	87.10	12.90

† = 100 colonies from each strain were examined for industrial effluent mutagenesis.

Patterns of soluble protein

Soluble protein patterns of parental bacterial strains and their transconjugants are shown in Figure 4. These results demonstrate certain differences and similarities among parental strains and their transconjugants with respect to their soluble proteins. Since proteins are under genetic control (Scandalios, 1979) the protein patterns shown here may establish a causal relationship between genetic background and protein. Ainsworth and Sussman (1965) reported that protein contents of fungal cells are known to vary with the species and age and composition of culture media. Many studies reported changes in electrophoretic protein pattern during differentiation of fungi (Hilbert and Botton, 1986). This results are in accordance with those obtained by Ragab *et al.* (1989), who found differences in protein banding patterns, amylase and esterase profiles between the wild type and four mutants in *Aspergillus flavus* according to growth stage and genetic background.

In conclusion, the present work is part of a wider effort to develop to full scale this novel hazard metals value recovery technology using genetic constructed microbial biomass in order to maximize the efficiency of the biomass use for the industrial application of biosorption for heavy metals recovery. Our results suggest that engineering of microbial strains with constitutive expression of Metallothioneins (MT) genes which may then accumulate elevated amounts of metals and the production of different MT specific for different metals (Butt and Ecker, 1987) could be very useful biosorption tool, it should be of interest in the removal and recovery of pollutants from industrial effluents.



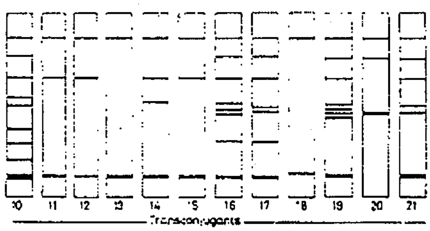


Fig. (4): Electrophoretic soluble protein patterns on polyacrylamide gel of parental bacterial strains and their transconjugants.

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Wastewater Reuse

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WASTEWATER REUSE

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ABSTRACT

Reutilization of water has been occurring for many centuries along the major rivers of the world where human settlements and large industries as well as agricultural activities take water from rivers and return the wastewater to them for subsequent use by downstream entities. Rapid population growth coupled with industrial and agricultural, the demand for water especially in the arid and semi-arid regions is continually on the increase, causing alarming draw down in the deep aquifers, the traditional source of water. Wastewater could be a valuable source to augment this dwindling water supply; at least the irrigation supplies and should not be wasted. As this paper discussed, there are two extremes of use of untreated wastewater for irrigation as practiced in some countries (for example China) and very stringent standards that necessitate an expensive tertiary treatment (for example California State, USA), there exist a middle-of-road approach. This intermediate solution should combine flexibility with a permit system on a case-by-case basis to make the best use of this valuable source of water. Sociocultural aspects are considered favourable especially from Islamic Religion point of view. The economical justification of wastewater reuse schemes should take into consideration the overall treatment costs, the environmental and health allowances, i.e. the cost of treatment required even when reuse is not practiced, the benefits of water and the saving in conveyance and pumping. Wastewater is a valuable resource and when properly treated and managed can be safely used for many purposes. This paper has summarized the wastewater treatment techniques mostly used in the world at present, especially for the reuse for irrigation purposes.

Advances in wastewater treatment technology, including treatment reliability, allow the safe use of effluent for any required purpose when reasonable precautions are taken. From health aspect view the paper concluded that wastewater treatment processes that effectively remove all or most of the pathogens in wastewater provide a major or total reduction in the negative health effects caused by raw wastewater reuse [7].

Key words : Wastewater Reuse

INTRODUCTION

Mankind has used wastewater effluent long time ago. In 1531 perhaps was the first recorded use of wastewater for irrigation in Germany. In the U.S.A. 1871, in France 1872, in India 1877, in Australia 1893 and in Mexico 1904. In most of the countries, at those times the aim was to protect the rivers from pollution rather than enhance crop production (Shuvel 1986). Unintentional, indirect reuses of wastewater effluent have been practised on the main rivers in the world. In the populated area of Rhine River there were more than six millions inhabitants dispose their wastewater into Rhine River after some degree of wastewater treatment. Downstream, other communities draw their needed water from the same river and dispose back to it. Enormous examples all round the world of the same case can be thought about. Following statement by UNESCO, draws attention to the mentioned facts "There is no new water on the face of the earth probably there is no previously unused water either" [1].

Within the past few decades the accelerating demand on natural resources to provide water for the vast use and the associated cost meeting that demand has led to an increase interest in the reuse of wastewater for domestic, industrial, agricultural and groundwater recharge purposes. The idea of "closing the water cycle" is supported by wastewater reuse that long cherished by environmentalist and ecologists. If it is done properly, it is a good environmental engineering practice with sound favourable, economical, political and public opinion merits [2]. It was suggested that the real solution of the main sources of environmental pollution should be based on the "Big Five R's": recycle, reuse, recovery reclamation and renovation. This is true for the coming century being the most critical factor by pollution and which on a world-wide scale is being the scare-fresh water [2]. The increase demands for water have forced reassessment of the sources of supply and has led to the consideration of new alternatives, among these is the use of renovated wastewater.

Wastewater from urban areas can pose a serious threat to the environment of developed and developing countries alike. Drinking water weather it is surface or groundwater sources may become contaminated by pathogenic microorganisms and toxic substances. Improper disposal of wastewater may pollute rivers, lakes, recreational and fishing areas and along coastal shores which result in odors and mosquito breading problems [3]. Arid and semiarid countries of the world, water is rather scarce, therefore each available drop of water must be carefully used and managed in a safe and economically feasible manner so that no higher quality, unless there is a surplus of it. Reuse of wastewater for domestic, industrial and agricultural purposes were practised all round the world. Several purposes will be accomplish if wastewater is treated safely and managed properly among those are:

minimizing or eliminating wastewater pollution, providing needed nutrients and fertilizer to soil and increasing agricultural output.

Today wastewater reuse is becoming widely accepted, but it is often based on very strict and unenforceable health regulations [(3)]. A survey carried out by the World bank-UNDP programme on the current wastewater reuse practice in developing countries has estimated that about 80% of the wastewater flow urban areas used for permanent or seasonal irrigation [(3)].

ENVIRONMENTAL IMPACT

Wastewater reuse has a positive and negative impact on the environment. It is important to identify these impacts and to invest the positive and to do what ever is possible to reduce or eliminate the negative. The positive effects of the reuse on the environment are elaborated below.

Reclaiming municipal wastewater for agricultural reuse is increasingly recognized as an essential management strategy in areas of the world where water is short supply. Wastewater improves the environment as it reduces the amount of waste (treated or untreated) discharged into water receiving bodies and it conserves water resources by decreasing the demand for fresh water[4].

Irrigation with appropriately treated wastewater has been used to reduce desertification, grow greenbelts which help to stabilize the desert around cities and control dust storms, and reforest barren areas and control soil erosion. Scarcity and increasing price of timber in the developing countries, used mainly for cooking and heating by low-income people, calls for renewed timber production, especially in the vicinity of urban centres [4].

The amount of organic and chemical contaminants entering surface water and groundwater are reduced through two natural processes. First, the substances are absorbed by the crops as nutrients and prevented from mixing with the runoff or groundwater. Second, pathogens organisms trace elements are filtered by the soil. This phenomenon is accomplished with minimum of technical input and with little or no harmful side effects [4].

Using wastewater for irrigation instead of groundwater in those areas where over-utilization of the later causing the mentioned problems can minimize the problem of salt-water intrusion in coastal areas.

The negative impacts of the reuse of wastewater on environment are elaborated below.

Due to the high cost and technology involved in the disaffection of wastewater by chlorination is uncommon in many countries (in many developing countries, even potable water is not disinfected). A negative aspect is the formation of trihalomathanes as a result of the chemical reaction between chlorine and humic compounds in wastewater. The most abundant of these trihalomethanes is chloroform, which is reported to be carcinogenic. Thus wastewater treatment followed by irrigation provides public health and environmental benefits that can not be achieved by treatment (including modern methods) and disinfection alone.

Ground contamination by nitrate accumulation is a serious problem in many countries which might result from the use of wastewater in irrigation and from the application of sewage sludge to land. The risk of such contamination of groundwater by wastewater irrigation depends on local conditions as well as the rate of application. Good understanding of the geological formation of the irrigated area and proper management of the irrigation will reduce the possibility of such contamination. On other hand, crops will take up nitrogen from the irrigation wastewater and there by reduce groundwater contamination.

There are some cases of environmental problems associated with the creation of habitat for disease vectors, such as mosquitoes or snails as a result of polluted water and standing pool. Application of standard vector control techniques should be applied wherever appropriate to prevent the transmission of vector borne disease.

SOCIOCULTURAL ASPECTS

The Sociocultural makeup of the involved influences the acceptance of wastewater reuse and the adoption of practice for safe implementation (That is, the values, beliefs and customs that are concerned with water supply, sanitation, hygiene and other activities related to water use). The idea of using wastewater in irrigation specially treated wastewater does not seem to create appreciable repugnance where it is implemented or proposed. In some areas, farmers have refused to exchange treated wastewater for available freshwater, other farmers with similar background in the same area have accepted wastewater irrigation. Thus this attitude seems to reflect an individual rather than cultural bias [4].

In some part of the world where farmers recognized the value of wastewater, have in some cases broken into sewer lines to take water even when reuse was not officially sanctioned. In Sana'a, Yemen partially treated wastewater is pumped to be used for illegal unstricted plants where fresh water is scarce. The population in developing countries, when made aware of imminent water-shortage problems, will be willing to accept reuse options especially if reuse is one component of an integrated water conservation effort.

The role of the religion on the feasibility of reuse in Islamic countries is often cited as an example of sociocultural that can limit the reuse of wastewater in these countries. The evidence shows that in most Islamic countries in the Middle East, water is scarce and wastewater is reused, mainly for irrigation[9, 10].

Social and behaviour factors are of fundamental importance to health considerations in human waste utilization. First human behaviour is a main factor in disease transmission from infected excreta. Second risk behaviours are controlled by deep-rooted cultural factors which differ from society to society and which are to be considered when planning for wastewater reuse. Third social acceptability of human waste utilization technologies may seriously influenced by their successful implementation.

The considerations of sociocultural factors in the development of wastewater reuse projects are of great importance. This may be done by social scientists whose familiar with the specific culture concerned. The feasibility and acceptability of wastewater reuse projects may be effected by important sociocultural and religious factors. For example in China, the use of raw excreta in agriculture is a deeply rooted and widely accepted cultural norm, so that pre-treatment may not be perceived as necessary. However, in some part of the world, religious and cultural constraints may forbid the contact with human faeces. Various religious authorities have permitted that the use of well purified, treated wastewater is an acceptable practice[5].

Islam divide water into three categories: tahour, tahir and mutanjjis. Tahour is pure natural water that could be used for religious and mundane purposes without any further treatment. Tahour water becomes tahir after religious washing and may be used again for religious purposes. Both tahour and tahir categories of water are liable to become mutanajjis, if they are defiled with pollution such as excreta or urine,etc., rendering them unfit both for the religious and mundane washing (this last category of water may still be used for irrigation, which involves no washing). However, if the basic properties of water color, taste and odor change owing to the mixing of water pollution, it will be considered unclean. The available technology, these days, can serve as a handmaid of religion by purify water of all contaminates. The reuse of wastewater effluent seems perfectly legitimate from the Islamic religious view point, and has therefore to be examined in each specific case from the latest concerns of health, cost, and public acceptance. It is worth to mention here that the Islamic Authorities in Saudi Arabia has unanimously expressed its approval of the reuse of wastewater effluent after their treatment for all purposes including religious purposes[11].

ECONOMICAL ASPECTS

Economical factors are governing water reuse in a community. With the progress of the technology for wastewater reclamation, economic consideration limits its use to special locations or particular purposes. As demands for water continue to increase, water reuse may gain more favour in certain uses and may release natural water sources for potable supplies. The controls of pollution are becoming stricter all around the world. Standards, guidelines and regulations require the contaminants to be kept out of systems wherever possible and the remaining contaminants to be treated in such a way to ensure that the environment is kept clean. To achieve this goal, the number of wastewater treatment plants are increasing to treat the wastewater to certain levels that will not cause pollution and will allow some direct use, or after additional treatment, almost any use. Where water is limited, the benefits obtained from it or the reclaimed water may help to pay for the cost of treatment.

Reclamation of water to satisfy the increase in demands depends upon the availability, reliability and cost of development of ground and surface sources of water, upon the costs of rainwater harvesting, upon the availability of brackish or sea water and the cost of desalination, and upon the practicability and costs of weather modification and other water augmentation techniques. When the cost of water of a certain quality can be applied by the other methods at costs less than that for water from advanced or secondary effluent, then advanced level treatment facilities will not be constructed for water reuse. As treated effluent is not to be used for drinking and its use is limited to certain purposes, the cost of treatment will vary according to the treatment processes required to meet quality demanded by the users.

Wastewater collection is usually the most expensive part of any city sewerage plant (sewers, pumping stations and treatment plants). Sewers are necessary, whether wastewater is to be reused or not and its cost should not be included in economical appraisals of reuse projects. This is applied to the cost of the wastewater treatment and disposal required to meet environmental protection requirements[5].

The charged cost for a reuse project is the cost of any extra treatment that would not have been required for pollution control purposes. Sometimes the quality of the effluent reuse in irrigation requires inferior quality as that needed for environmental protection, the reduction in treatment cost would then be a benefit of the reuse project. Effluent storage and distribution systems (ponds, tanks, canals,pipes,etc) are additional costs that included in the cost of irrigation project.

Additional economical benefits of the reuse included the value of agricultural crops, the job opportunities, the saving in the high cost of chemical fertilizers, wastewater can therefore partly or even fully replace chemical fertilizers and environmental pollution damage avoided.

In Peru benefit-cost studies showed that the irrigation components in reuse projects were feasible even if land and operation and maintenance for treatment were charged to farmers, but they were not economically viable if the full cost of investment in treatment facilities were charged against agricultural component[6].

The economical concept of reuse of wastewater is in principle should provide an equitable sharing of costs and benefits resulting from the multiple use of the water. If wastewater reuse saves money for the producer and user, the benefits should be shared. At the same time, if result is extra cost to one or other, then the burden also should be shared [4].

Economical evaluation of wastewater irrigation is difficult as the valuation of non financial aspects such as protection of environment or health. Less difficult problem is the allocation of treatment costs between wastewater producer and agricultural user. After solving these difficulties the analysis of wastewater irrigation can be carried out using standard technique of economic and social cost-benefit analysis[7].

HEALTH ASSESSMENT

The potential health hazards are in proportion to the degree of human contact with the water, the quality of the effluent and reliability of treatment processes for irrigation purposes. The contaminants in reclaimed water that are of health significance may be classified as chemical and biological pollutants. In general the biological contaminant is probably pose the greatest health risks, and quality control measures are directed at these pollutants. The chemical constituents (toxic heavy metals, pesticides and other organic contaminants) that may cause long-term health effects. These two contaminants, chemical and biological, will be discussed with some detail:

Chemical Pollutants

Wherever industrial wastewater is allowed to be discharged into the municipal sewerage system is likely the municipal wastewater to contain chemical pollutants. Those pollutants can be a variety of heavy metals and their compounds, pesticides, non degradable organic, etc. The chemical constituents are major concern when reclaimed water is used in the recharge of groundwater for indirect potable reuse, and could also be a concern when is used for crop irrigation. The mechanisms of food crop contamination and repeated application may lead to a build-up of contaminants on crop; uptake through the roots from the applied water on soil; and foliar uptake. The potential toxicity problem is the accumulation of heavy metals in plant parts that enter the food chain of human or animal. For example, cadmium (Cd) presence in municipal wastewater at levels that are not toxic to plants but that could built up inside the plants to levels harmful to humans and animals[4]. To prevent this type of built up, it is recommended to use the land limiting constituent (LLC) analysis in an all inclusive design method to control the accumulation of toxicant during wastewater reuse on land[12].

There is a shortage of information regarding the health significance of many of the known or suspected carcinogenic, mutagenic, or teratogenic organic constituents that may present in wastewater used in irrigation, some of these chemical constituents are known to accumulate in particular crops and thus may present health hazards to both grazing animals and humans.

Preventing industrial wastewater containing chemicals entering municipal sewerage is the best way of dealing with the problem, however, this is difficult to achieve where there are many small-scale industries, as in the developing countries.

Biological Pollutants

The principal infectious agents that may present in wastewater may classified into three broad groups: ***virus***, ***bacteria*** and ***parasites*** (***protozoa*** and ***helminths***).

There are approximately 30 excreted pathogens of public health importance and many of these are specific important in wastewater reuse projects. Reuse of wastewater in agriculture results in actual risk to public health if all of the following stages occur:(a) that either an infective dose of an excreted pathogen reaches the irrigation field or the pathogen multiplies in the field to reach an infective dose; (b) that this infective does reaches a human host; (c) that this host becomes infected; and (d) that this infection causes disease or further transmission.

If (d) does not occur then (a),(b) and (c) only pose potential risk to public health[1].

The transmission of virus disease is the least important and helminth infection is the most important health risk, with bacterial and protozoa diseases falling between those two extremes (1,p32).The people subject to potential risk from the agricultural use of wastewater are:(a) Workers of the agricultural fields; (b) crop handlers; (c) consumers of crops, meat and milk; (d) people living near by the fields concerned.

Workers in agricultural fields can be protected by wearing protective clothing (to prevent contact with pathogens), increased degree of hygiene (to prevent any pathogens present), avoidance of contract with pathogens by behaviour modification through public education and enhancement of public awareness, and possibly by immunization or chemotherapeutic control of selected infections. Agricultural fields workers' exposure to hookworm infection can be reduced by the continuous in field used of appropriate footwear, but this may be somehow difficult to be achieved in some developing countries.

Immunization is not effective against helminthic, infection, nor against most diarrhoeal diseases, but immunization of highly exposed groups against typhoid and hepatitis A may worth considering. Cooking and high standards of hygiene can reduce risks to consumer. Food hygiene is a theme to be included in health education campaigns. The efficiency of such campaigns may seem to be quite low in short span of time, however, in long term is the key to ever lasting protection of health measurements. People living near by fields which are irrigated by reused wastewater should be advised to avoid entering them and also prevent their children from doing so. Where sprinkler irrigation is practised residents should be 50 to 100 m away from such fields.

CROP RESTRICTION

Crops can be grouped into three categories according to the degree to which health protection measures are required[1]:

Category A

Treatment to WHO guidelines for "unrestricted" irrigation is essential: this covers fresh vegetables, spray-irrigated fruits, parks, lawns and golf courses.

Category B

Protection needed only for field workers: this includes industrial crops such as cotton, grains and forestry, as well as food crops for canning, fodder crops, pastures and trees.

Category C

Localized (drip or bubbler) irrigation preventing the exposure of workers and consumers: health protection measures are not required but pre-treatment is necessary to facilitate the operation of the irrigation installations.

One of the earliest and still most widely practised measures is to strict the type of crops which irrigated with raw wastewater or poorly treated effluent. Salad crops and other vegetables normally eaten raw are the main vehicle for the transmission of diseases associated with wastewater irrigation, forbidding the use of raw wastewater to irrigate such crops can be an effective method for protection of public health. In developed countries with long traditions of civic discipline and effective methods of law enforcement. Such regulations work well specially in centrally managed sewage farms or irrigated districts. However, in developing countries would be difficult to enforce laws and regulation among subsistence farmers near major urban canters irrigated with raw wastewater, the market demand for salad crops and fresh vegetables is great. It is difficult to enforce regulations that prevent farmers from obtaining the maximum benefit without some kind of compensating for the losses.

WASTEWATER TREATMENT

The objective of the wastewater treatment from all sources is the reduction of pollutants (solids, dissolved organic and inorganic compounds) and organisms (pathogenic agents such as bacteria, viruses and parasites). To fulfill this objective, verities of processes and technologies are available. The collected wastewater is transported to a treatment plant; the treated effluent is then returned to the receiving water body (either surface or groundwater) or to be used for irrigation or other purposes.

The selection of the proper process requires intensive consideration of various factors, including: (a) The quality of requirements for any particular reuse; (b) The need to reuse and conserve renovated water; (c) Economic; (d) Cultural and practical considerations; (e) Operation and maintenance capabilities; (f) Availability of chemicals and spare parts; (g) Process reliability; (h) Capital and revenue funding and land areas required.

Wastewater treatment plants have traditionally been designed to reduce pollution by removing organic solids and by stabilizing the effluent, and have excluded specific measures for pathogen removal. The choice of process is usually made by comparing latest developments with proven techniques to provide a scheme appropriate to the particular local situation. However, for developing countries with warm climates, or where sufficient land is available even in seasonally cold climates, the adoption of "natural" purification in stabilization lagoons has the attraction of low cost and simple operational needs[14].

In some projects in the Middle East the potential for effluent use has been recognized at an early stage and sewage treatment facilities developed accordingly. In all cases decision on the method of sewage treatment and the quality of effluent required have been made in the light of technical, economic, cultural and practical considerations. Particular emphasis has been placed on operation and maintenance capabilities, availability of chemicals and land areas required(14).

However, when there is need for reuse of wastewater for municipal, industrial, agricultural or any other purposes, then the selection of the wastewater treatment should be selected without any potential hazard to public health[14].

The sources of wastewater are municipalities, industry and agriculture. The total amount of wastewater available is dependent upon various factors. Wastewater for reuse is available largely from urban centres. The main constituents of the sewage are water(97-99.9%), organic matter and little amount of chemicals.

Industrial waste are characterized with toxic metals and chemicals may destroy the biological activity in municipal sewage treatment and excessive concentrations of organic matter may place a heavy burden on these plants unless they designed to deal with such excessive loads(15).

Agricultural wastewater (irrigation return flow) associated with advanced methods of irrigation such as drip, bubbler systems, etc with experienced management do not produce appreciable amount of return flow. Surface irrigation may introduce appreciable volumes of return flow that may contaminates surface or groundwater with varying amounts of fertilizers, pesticides and dissolved salts.

Continuous percolation of irrigation return flow without appropriate drainage may result in accumulation of salts and heavy metal and undesirable substances in underlying aquifer would also be of considerable concern.

Wastewater reuse is for municipal, industrial and agricultural purposes. The most potential reuse of the three uses of wastewater in the world is probably for irrigation. The emphasis on the treatment of wastewater for irrigation purposes will be given more attention in this paper.

The degree of removal of microbiological constituents of wastewater by treatment process can be expressed in terms of $\log_{10}$ units(e.g. a reduction of $4\log_{10}$ units $= 10^{-4} = 99.99\%$ removal(5).

The primarily aim of ***conventional wastewater treatment*** is to remove or reduce the suspended and dissolved organic fractions which decompose rapidly in natural bodies of water. The secondary goal of conventional wastewater treatment is to reduce or eliminate pathogenic micro-organisms in order to protect the quality of the sources of drinking water used by downstream communities. It has been found that conventional treatment systems are not particularly efficient in removing pathogens (13). Table 1 Shows the range of efficiency of pathogens removal for primary sedimentation, septic tanks, trickling filters and activated sludge (conventional wastewater treatment processes) as well as low cost waste-stabilization pond system. The most efficient conventional systems activated sludge which removes 90-99% of the virus, protozoa and helminths, and 90-99.9% of the bacteria. Higher percentage removal of pathogen can not be achieved by conventional processes without chemical disinfection, such as chlorination or further treatment such as filtration.

Waste stabilization ponds require minimal operational and maintenance skills and energy. When ponds are operated in series produce a high quality effluent with few settable solids, a safe level of pathogenic, bacteria, no helminths and high nutrients content[4]. Multicell pond systems with good design features and with a total retention time of 8-10 days might have removal efficiency 99.999-99.9999% (4-6$\log_{10}$) of the bacteria and helminths from raw wastewater, and with warm climatic conditions faecal coliform may be as low as 1,000 per 100 ml[4,5].

Disinfection of raw sewage by chlorine has never been completely successful in practice. The number of excreted bacteria in the effluent of well operated conventional treatment plant can be reduced substantially by chlorination. It is expensive practice and very difficult to maintain a high, uniform and predictable level of disinfecting efficiency. Chlorination will leave helminths eggs totally unaffected[5].

Tertiary treatment may be used to upgrade effluent from secondary biological treatment plants. In some countries rapid sand has been used mainly for the removal of further of suspended solid, nutrients and to reduce the BOD, with little decrease in micro-organisms. However the removal of helminth eggs in a well operated filter plant can expected to be effective. Intermittent slow sand filter has been used limited areas with acceptable results. The use of one or more ponds in series to a conventional treatment plant is a suitable means of upgrading an existing wastewater plant[5].

Sludge obtained from wastewater treatment contains a lot of undesirable chemical, biological and helminth eggs. To render a sludge contains helminth eggs safe for general use, it must be stored for 6-12 months in sunny dried weather[16].

CONCLUSIONS AND RECOMMENDATIONS

1. Wastewater is a valuable resource, specially in arid and semi-arid regions, which deserves to be reused in one way or another without adverse effect to public health.

2. Wastewater reuse reduces the environmental pollution, increases agricultural production, provides job opportunities, substitutes for chemical fertilizers, etc.

3. Public health can be protected by the following integrated measures: crop restriction; type and the degree of wastewater treatment; methods of application; human exposure and education and farm extension.

4. Selection of wastewater treatment technology for irrigation purposes should be based on maximum helminth removal; effective reduction of bacteria and virus and nuisance-free effluent.

5. In the development of a national master plan for water resources, attention must be given to include the reuse of wastewater as an essential water resource.

6. The formulation of national wastewater reuse regulations based on local environmental implication is an urgent need in most of the developing countries. Wastewater reclamation and regulation should be strict enough to permit irrigation without undue health risk, but not so strict that they would virtually rule out irrigation with sewage effluent.

7. Cultural and religion attitudes towards excreta disposal are influencing public acceptance of effluent irrigation. Islam has no objection against wastewater reuse if public health is preserved. Health promotion and education components may have to be built into reuse projects.

8. Wastewater treatments needs to be given priority to create a barrier in the potential transmission of disease through wastewater reuse in agriculture. Among all available technology in wastewater waste stabilization ponds, and in particular maturation ponds, can, if operated properly, provide an adequate solution to the disinfection of highly contaminated raw wastewater and make it suitable for direct reuse in agriculture.

9. Special consideration should be given to the health risk from toxic chemicals in wastewater. Effective and continuous monitoring specially when wastewater suspected to contain some qualities of industrial effluent. Groundwater and environment should be protected from contamination with toxic metals (heavy metals) and salt built up, by proper water resources management policies.

10. In general and based on economic benefit-cost analysis wastewater reuse is economically justified especially when associated factors such as environment benefits are considered.

11. Through training of personnel is essential to make wastewater reuse a successful practice by maintaining safe and economic operation. Training is recommended at different levels.

12. Research work on wastewater reuse in developing countries should be directed towards finding ways to reduce cost and to reduce largely virological hazards.

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Novel Approach to Soil Water Management Using a Hydrophobic S1Loxane Polymer

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NOVEL APPROACHES TO SOIL WATER MANAGEMENT USING A HYDROPHOBIC SILOXANE POLYMER

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ABSTRACT

The effectiveness of a non-toxic hydrophobic polymer (Guilspare) in irrigated agriculture to reduce evaporation, increase water-use efficiencies by a field grown crop, improve water distribution, and enhance salt leaching were studied under laboratory and field conditions. Guilspare, sprayed on soil as an aqueous solution, renders to the surface water repellency. In a soil column experiment conducted under ambient summer conditions, application of the hydrophobic polymer substantially reduced cumulative evaporation relative to untreated columns. The pattern of water loss from Guilspare treatments was approximately linear over time, while for controls the pattern of water loss was cyclic. Rates of water loss from control columns were significantly higher within 24 hours of water addition than treated columns. Guilspare treatment substantially reduced the amount of water loss during the constant rate stage (stage 1) of soil drying. The rates of stage 1 drying from control columns were higher than during falling rate stage (stage TI) of soil drying. The patterns of water and salt distribution with depth were substantially different after 36 days of watering with moderately saline soil. Guilspare application showed an increase in soil water contents in all soil depths in columns relative to controls. Guilspare treated soil also showed increased salt content at lower soil depths than controls. Yields of Okra *(Adelmoschus esculentus)* transplanted in April and October 1998 from treated and untreated control plots, at three rates of irrigation were higher on treated plots. Treated plots, with 25% less water (April-planting) and 50% less water (October-planting) gave similar yields to untreated plots. The treatment had a marked effect on suppressing weed growth. A wider perspective of the potential uses of the polymer in semi-arid and arid countries is proposed including; geomembrane type applications to assist in cultivating soils of low water holding capacity; maintaining salt free conditions in surface soils above saline subsurface horizons; reductions in water losses during conveyance, and run-off entrapment. This paper is

offered as a contribution to stimulating interest in using a relatively new technology to help achieve water security in Arabian Gulf Countries.

Key words : Evaporation, Salt Leaching, Guilspare, Okra *(Adelmoschusesculentus)*, Oman

1. INTRODUCTION

Proposing and evaluating alternative ways of performing farming operations are important goals for agricultural research. Should a new methodology under trial prove better than existing practice, its successful adoption is still, however, not guaranteed. Requiring farmers to perform additional tasks and to meet increased costs, usually result in low acceptance of new techniques. In dry areas, however, the need to increase irrigation water use efficiency and conserve water is widely recognized. Established methods to retain water in water soil through limiting evaporation include conservation tillage, mulching and the use of modern irrigation systems. Synthetic polymers [1] have been proposed as possible alternatives, but their use is still mostly experimental. Bouranis *et al.* [2] classified water insoluble polymers for use in soil as either hydrophilic or hydrophobic. The latter polymers include polysiloxanes, formed from alkylsiliconates in the presence of an acid [3]. Guilspare is a new water-based preparation that polymerizes in-situ on soil particles, coating them with a chemically bound alkylsiloxane polymer to render individual particles hydrophobic. All the potential uses of Guilspare have not yet been fully explored. This paper aims to review some of the experimental data gathered during the evaluation of Guilspare in Oman and propose a range of potential uses.

2. SOIL DRYING STUDY

The hypothesis was that the effect of applying the same amount of Guilspare polymer to soil surfaces on reducing evaporation would depend on the rate and frequency of irrigation water addition. The aim was to test, under ambient weather conditions of the Oman, the effect of two rates of watering at two intervals of applications on evaporation rates from a light textured soil in the presence and absence of a novel hydrophobic polymer.

2.1 Materials and Methods

A fine sandy loam (9.5% clay, 22.1% silt, 68.4% sand) soil was air-dried and passed through a 2 mm sieve. Soil pH was 7.56, and the $CaCO_3$ content 53%. Soil was packed into 16 cylindrical PVC columns, 600 cm in length and 10 cm in diameter. The soil surface was located 5 cm below the rim of the column to facilitate water application. The bulk density of the packed soil was 1.70 Mg m^{-3}.

The treatments imposed on the columns were as follows: water was applied at 2 rates (3 and 6 mm day^{-1} , denoted Rl and R2, respectively), at two intervals (4 and 8 days, denoted 11 and 12, respectively), either with or without Guilspare addition, denoted G and C, respectively). Moderately saline water (EC of 3.80 dSm^{-1}) was used in the experiment. Water salinity was adjusted for different R x I combinations to ensure that all columns received, by the end of the experiment, the same total amount of salt. Treatments were combined factorially and replicated.

Guilspare treated columns were prepared by pipetting aqueous solutions of Guilspare (23 mL of a 2% solution per column) evenly to dry soil surfaces and left to dry outdoors for 24 h.

After adding water, columns were weighed and set outdoors in sunlight at 09:00, then reweighed twice a day for 36 days. At the end of the experiment, columns were disassembled, and water and salinity contents in soil layers at regular depths were determined.

Meteorological data was collected during the experiment from an automatic weather station located 4 km from the experimental site.

The soil-filled containers acted as microlysimeters [4]. Water loss from soil was assumed to be entirely evaporative. The evaporation rate, expressed in mm of water, was calculated as ((initial weight - weight at time t) / soil surface). Cumulative evaporation data were analyzed by Analysis of Variance (ANOVA). Standard errors of the means (SEM) were calculated and Fischer's Least Significant Difference (LSD) values used to compare treatment means at each sampling time.

2.2 Results and Discussion

2.2.1 Weather conditions during the Experiment

Daily variations in maximum, mean and minimum air temperatures and relative humidities, and average wind speeds, and total and net solar radiation were recorded during the experimental period. Mean daily weather conditions were as follows: air temperature 35 °C, relative humidity 60%, wind speed 2.00 ms^{-1}, total and net solar radiation 559 and 279 cal cm^{-2}, respectively, and average potential evaporation of 17 mm day^{-1} prevailed during the period of study.

2.2.2 Effect of different rates of watering and frequency on cumulative evaporation.

Cumulative evaporation over 36 days for Guilspare treated and control columns watered at different rates and frequencies are shown in *Figures 1 and 2.* ANOVA revealed highly significant differences in cumulative evaporation between Guilspare treated and control columns. Guilspare treatment significantly reduced cumulative evaporation relative to controls within one day, and to continued to do so for the following 36 days. After 36 days, Guilspare treatment reduced average cumulative evaporation relative to controls by 53.5% for Rl and by 41.7% for R2 watering regimes. Reducing the interval between watering from 8 to 4 days significantly increased cumulative evaporation from control columns, but did not in Guilspare treated columns *(Table 1)*.

The two-stage classification of evaporation from bare soil under constant external conditions [5] helps to describe the patterns of cumulative water loss from treated and untreated soil. In the first stage of drying, atmospheric conditions determine the rate of loss, whereas in the subsequent second stage, drying is slower and limited by water diffusion within the soil [6]. In the R2I2 treatment combinations, mean cumulative evaporation *(Figure 3)* during the first 24 h of the drying cycle (defined as stage one drying) was higher from control soils than from soils treated with Guilspare (i.e. 6.2 and 1.4 mm d^{-1}, respectively). However, evaporation from during the last 24 h of the drying cycle (defined as stage two drying) was nearly identical between control and Guilspare treated columns (i.e. approximately 0.8 mm day^{-1}). Consequently, it appears that the effect of Guilspare was to reduce evaporation more in the period of the drying cycle when soils were relatively wet than in the period when evaporation from natural soils was dependent on the upwards transmission of water from lower soil layers to the surface.

2.3 Interpreting the Guilspare Effect

Following polymerization, Guilspare renders the treated layer water repellent, and hence dry compared with underlying wet soil. In an untreated soil, the hydraulic conductivity (K) is directly related to soil water content, through the equation of Kovacs [7] where K_s is the saturated conductivity, θ is the soil water content, θo is the soil water content when containing only strongly held water, n is porosity, and m is an empirical parameter between 3 and 4. Normally, the wetter a soil, the higher is its hydraulic conductivity. As the surface dries, conductivity falls and so does the evaporation rate. We suggest that Guilspare works because the constantly dry layer at the surface maintains a low hydraulic conductivity, thus slowing soil drying.

$$K = K_s ((\theta - \theta o) / (n - \theta o))^m \qquad (1)$$

Gardner and Hanks [8] and Fritton *et al.*. [9] have suggested that an evaporative zone about 1 cm thick develops in any soil, through which water from underlying wet soil must move before evaporating. In normal conditions, this zone may be near the soil surface in early stages of evaporation, but retreats downwards as the soil dries. Guilspare application rates need to be aimed at treating soil to a depth of at least 1 cm. This was probably only achieved in the loam textured soil of the present study with application rates of 3 $L.m^{-2}$ **or** more.

3. SOIL WATER AND SALT DISTRIBUTIONS UNDER GUILSPARE

3.1 Soil Water Distribution

Leaching soluble salts into lower soil horizons using fresh water offers an attractive method to treat salt affected soils. However, leaching in an arid environment is a use for water that might otherwise be used for raising a crop. Any improvements in the efficiency of the leaching process by reducing water loss through evaporation were investigated by determining the distributions of water and soluble salt concentrations in soils at different depths in the soil columns.

The distribution of water in soil at different depths for water application rates and intervals are shown in *Table 2.* Soils treated with Guilspare were wetter throughout the column than controls. It is of interest to note that increasing the frequency of water addition did not affect soil water contents in Guilspare treated soils, whereas control soils were significantly drier.

3.2 Soil Salinity Distribution

Salt tended to accumulate in surface layers of both Guilspare treated and control soils *(Table 3).* The extent, however, of salt accumulation was lower in Guilspare treated soils than controls, at least, in the R2 treatment combination. In subsurface soil layers, salinity tended to be inversely related to water content, i.e. wetter soils tended to be lower in salinity than drier soils. Hence, in Guilspare treated soils, salts accumulated in the lowest (and wettest) layer, whereas in control soils salts were relatively uniformly distributed under the R2 regime, but decreased significantly under Rl. The conclusion is that with the same amount of water added, salts were leached to greater depths in Guilspare treated soils than controls by reducing evaporative losses.

4. FIELD TRIAL ON YIELD RESPONSE TO GUILSPARE APPLICATION

The effect of Guilspare on improving yields of okra *{Adelmoschus esculentus}* was investigated in field trials [10] in Oman. The aim of these trials was to determine if Guilspare-treated soil would require lower irrigation rates than non-treated soils for similar crop yields, synonymous to an increase in yield per unit of irrigation.

4.1 Materials and Methods

Experimental Site and Layout of Plots: A 34x16-m area at the Agricultural Experiment Station of Sultan Qaboos University was used.

Soil class: coarse-loamy over sandy, mixed, hyperthermic, Typic Torriarents, (top 25 cm: 12% clay, 46% silt, 42% sand). Salinity: highly variable, from < 4 dSm^{-1} to 20 dSm^{-1} in saturated paste extracts. All crop production on site depends entirely on non-saline irrigation water.

Experimental plots were rotovated then hand-raked to remove stones prior to Guilspare application and planting okra. The same area was used for the successive autumn and spring crops. The experiment consisted of three Guilspare treatments and three irrigation rates, assigned at random to nine plots, arranged as a Latin Square. Each column had four 34 m lengths of 13 mm diameter pressure regulated driplines 1 m apart, and was divided into three 10 m long plots, surrounded by 1m wide walkways. Columns were separated by 2-m wide walkways. Water emitters along the driplines were 50 cm apart. Irrigation for each column was as indicated in Table 3, and was independently controlled and metered through an inlet valve Guilspare application: Aqueous solutions were applied to plots in each column at 20, 10 and 0 g L^{-1}, all at 4 L m^{-2} (G2, Gl and control) with a tractor-mounted pump and a 4 m long hand-held plastic spray-bar. The volume applied was adjusted by controlling the speed at which the spray bar passed over a unit area of soil. Solutions, applied to dry soil, were allowed to dry for 6 days before planting.

Cultivation and Irrigation Rates: 3 week-old seedlings of a heat tolerant okra variety were transplanted along all driplines on 12 April (spring planting) at a density of two plants per emitter. Fruits were picked by hand, in 8 harvests, on days 45, 55, 62, 69, 78, 85, 92 and 99 after transplanting. Plants were left to dry in the field, and then stubble and roots were removed by hand. The site was replanted on 6 October (autumn planting) with 3-week-old okra seedlings (Clemson Spineless) at two plants per emitter. Fruits were harvests on four occasions. All harvest data were calculated in Tonnes (green pod) per ha. No fertilizer was applied for the spring planted crop, but three weeks prior to the autumn planting fertilizer was injected through the irrigation system to all plots. Three rates of irrigation were applied for both spring and autumn planted experiments. Irrigation began daily, at 8.00, and amounts were adjusted so that columns received water in a fixed proportion to the amount received by the highest rate of application.

4.2 Meteorological and Irrigation Data

Spring: Air temperature: 46.5 - 24.2 °C. Relative humidity: 9.4 - 98.7%. Wind speed: 1.9 - 4.5 m s^{-1} . Cumulative reference evapotranspiration ETo: 827.8 mm over the growth period. Amounts of water applied to each irrigation treatment: 1000.3, 850.3, and 723.7 mm, i.e. in ratios of 1.21, 1.03 and 0.87 to cumulative ETo, respectively. Average daily irrigation rate for the 1.21 ETo plots was 10.3 mm d^{-1} .

Autumn: Air temperature: 38.0 - 14.6 °C. Relative humidity: 7.6 to 99.3%. Wind speed: 0.85 to 2.28 m s^{-1}. Cumulative reference evapotranspiration ETo: 390.04 mm over the growth period. Amounts of water applied to each irrigation treatment: 397.8, 259.9 and 192.2 mm, i.e. in ratios of 1.02, 0.66, and 0.49 to cumulative ETo, respectively.

4.3 Okra Yields in the Spring Growth Period

Total cumulative yields for the whole growth period are shown in *Table 4*. Averaged over the 3 irrigation rates, the Gl treatment gave a 40.3% increase in the average total yield compared to control plots. ANOVA showed yield differencbetween treatments and controls were not significant at $p < 0.05$. The highest total yield of 4.09 T ha^{-1} came with the Gl treatment at the highest irrigation rate.

In general, plants grown on treated soils yielded a relatively greater proportion of fruit earlier in the season: after 62 days, averaged over all irrigation rates, cumulative yields with the Gl treatment were 41.1% of the total yield, compared to only 33.1% on control plots.

4.4 Okra Yields in the Autumn Growth Period

The amounts of water applied by irrigation treatments, both absolute and relative, were lower in the autumn than in spring since ETo rates were lower. *Table 4* shows that total cumulative yields from G2-treated plots, averaged over all irrigation rates, were significantly higher than from either the Gl treatments or the controls. The highest cumulative yield of 6.25 T ha^{-1} , twice that of the controls, came from Gl treatments at the highest irrigation rate (1.02 ETo). At the lowest irrigation rate (0.49 ETo), the yield ratio from the G2 treatment to control fell to 1.50. This suggests that increased yields with Guilspare under a hot, arid climate may best be achieved by growing plants with relatively high irrigation rates and on smaller planted surfaces, rather than trying to reduce irrigation with the same surface area.

5. POTENTIAL AGRICULTURAL USES FOR HYDROPHOBIC POLYMERS

5.1 Soil Surface Sealant for Vegetable and Perennial Crops

The potential for treating vegetable crops with a surface sealant to reduce evaporation has been demonstrated to be feasible for experimental areas. However, serious cost considerations may be encountered if polymers are used for this application. In addition to the large volumes of soil that must be treated, i.e. areas of several ha to a depth of at least 1 cm, there remains the necessity to reapply after plowing. One advantage of apply polymers is the possibility of obtaining more than one crop per application without the need of replowing. Treating perennial tree crops would avoid destroying the polymer layers by regular plowing and also reduce the area of soil to be treated per productive unit. The cost of treatment could then be discounted over an 8 to 10 year period. Applications around lime trees and date palms are probably more cost beneficial than to vegetable fields.

5.2 Subsurface Sealant for Soils with low Water Holding Capacity

Many soils in Oman contain large amounts of gravel and sand overlaid by a desert pavement. The potential for irrigating these soils is low, even though they often occur in areas having good quality irrigation water. Farmers cultivate these soils by removing the desert pavement, excavating the coarse material and back filling with fine soil. An improvement to this method would be to apply the polymer around the bottom of the excavation, prior to back filling. The treated layer would act as a buried geomembrane preventing rapid water loss out of the rooting zone.

5.3 Combined Surface and Subsurface Sealant for Soils.

In this application, the subsurface and surface treatment proposed above would be combined to produce, in effect a 'water proof buried pot'. Both evaporation and drainage losses could thus be minimized.

5.4 Weed Control

Guilspare applications, from visual observation of field experiments, appeared to be effective in reducing germination of air-borne weed seedlings. This effect may be commercially important since, at present, few herbicides are registered for weed control for many vegetable crops, including okra fields [11].

5.5 Increasing the Efficiency of Salt Leaching

The possibilities of increasing the amount of salt leached by added water through reducing surface evaporation has been described above. Salt accumulation occurs in almost everywhere in irrigated agriculture and is often worst in greenhouse. Polymer applications in Greenhouse may contribute to the problem of treating soil salinity under plastic.

5.6 Subsurface Salinity Barrier

It is well established that Guilspare acts as a barrier to water vapour diffusion. If the amount of water vapour passing through a soil treated with Guilspare is reduced, then it is most likely that the rate of accompanying salt transfer would also be reduced. If this is the case then a saline soil would be cultivated by first applying a 2-3 cm layer of Guilspare treated soil, and then a thicker layer of untreated non-saline soil. The new surface soil may now be irrigated as normal, reducing the hazard of subsurface salt rising to the surface

5.7 Reducing losses in water Conveyance

Since polymers are capable of water proofing any surface containing silicon, then it **is** possible to treat falaj channels. Water flow through the falaj will be accompanied by few losses, possibly through reducing weed and algal growth infestation, repair of minor damage to treated channel surfaces, and any water retention by cement. In addition, it may be possible to waterproof unlined channels currently used for water conveyance.

5.8 Runoff Entrapment

In some arid countries receive winter rainfall in sufficient quantity for local farmer to harvest by entrapping run off. Run off channels could be water proofed to provide a more complete gathering of surface runoff water.

5.9 Water Surface Protector

In volume of unshaded, stored water can be reduced by evaporation. It may be possible to protect open bodies of water from evaporation by floating low density, Guilspare treated particles on the surface, so as to achieve an almost complete coverage.

5.10 Tree Surface Sealant

Trunks and branches of lime trees and date palms are often damaged during cultivation, and the uncovered areas are susceptible to water loss and insect infestation. Since these surfaces contain Silicon, polymers can effectively water proof any of these treated areas.

6. CONCLUSIONS

The results review show the mechanism of action of a surface applied hydrophobic polymer in reducing evaporation is reasonably well understood and these polymers can improve yields of irrigated crops growing in a hot climate in light soils. A combination of high cost and limited duration of action may limit the utilization of polymers by farmers. To overcome these problems alternative uses for hydrophobic polymers have been proposed. Essentially these uses either reduce costs by limiting the size of the treated area, or increase the duration of polymer action (from the period of one crop to many years for example). In addition, the added benefit of enhancing water movement below the treated layer can be taken advantage of in soil desalinization projects. It is probable that, following their first as a soil clod treatment by Hillel [1] and the current development of Guilspare, future formulations of hydrophobic polymers will be still more effective and the range of applications increased.

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Table 1. Influence of Guilspare Treatment on average Cumulative Evaporation (mm), with different water application rates and intervals.

Treatment		Guilspare Treated	Control
Rates (mm day'¹)	3	39.75 d	74.25 b
	6	46.25 c	110.75 a
Intervals (days)	4	43.50 c	94.75 a
	8	42.50 c	90.25 b

Figures followed by the same lower case letter are not significantly different at the 5% level of probability, according to DMRT.

Table 2. Volumetric Water Distribution ($m^3 m^{-3}$) with Depth, as influenced by Guilspare treatment with two intervals of water application.

Treatments		Guilspare		Control	
Interval Water Application (day)		4	8	4	8
Depths (cm)	0-5	0.215 c	0.232 c	0.185 e	0.203 d
	5-15	0.259 b	0.256 b	0.205 d	0.239 c
	15-25	0.254 b	0.262 b	0.171 e	0.188 e
	25-35	0.249 b	0.250 b	0.157 e	0.180 e
	35-45	0.257 b	0.250 b	0.103 g	0.137 f
	45-55	0.292 a	0.281 a	0.061 h	0.130 f

Figures followed by the same lower case letter are not significantly different at the 5% level of probability, according to DMRT.

Table 3. Salt Redistribution (EC, dSm^{-1}) with Depth, as influenced by Guilspare treatment with two rates of water application.

Treatments		Guilspare		Control	
Rate of Water Application (mm day^{-1})		3	6	3	6
Depths (cm)	0-5	4.225 a	2.255 b	3.808 b	3.270 c
	5-15	1.039 f	0.635 f	1.650e	0.959 f
	15-25	1.030 f	0.642 f	0.853 f	0.777 f
	25-35	0.999 f	0.638 f	1.513 e	0.882 f
	35-45	1.523 e	0.749 f	0.721 f	1.426g
	45-55	3.500 be	1.602 be	0.670 f	1.338 g

Figures followed by the same lower case letter are not significantly different at the 5% level of probability, according to DMRT.

Table 4. Effect of Guilspare and Irrigation Water Applied on Total Cumulative Okra Yields (T ha^{-1}).

Irrigation Water Application						
	Spring planting			*Autumn planting*		
	0.87 Eto	1.03 Eto	1.21 Eto	0.49 Eto	0.66 Eto	1.02 ETo
Gl	4.06	3.45	4.09	3.35 a	3.65 ab	5.04 a
G2	3.76	3.99	3.95	4.35 a	5.26 a	6.25 a
Control	2.26	2.27	2.45	2.90 a	2.74 b	3.14b
Mean	3.36	3.24	3.50	3.55 A	3.88 AB	4.81 AB

1. Differences between cumulative yields were not significant at the 5% level of probability.

Least Significant Difference ($p<0.05$) between treatments = 2.057

2. Figures in each row followed by the same lower case letter are not significantly different at the 5% level of probability, according to DMRT, with SEM of 0.308.

Figures followed by the same upper case letter are not significantly different at the 5% level of probability, according to DMRT, with SEM of 0.178. ANOVA revealed that differences between column means were not significant, $F_{2, 2} = 13.76$.

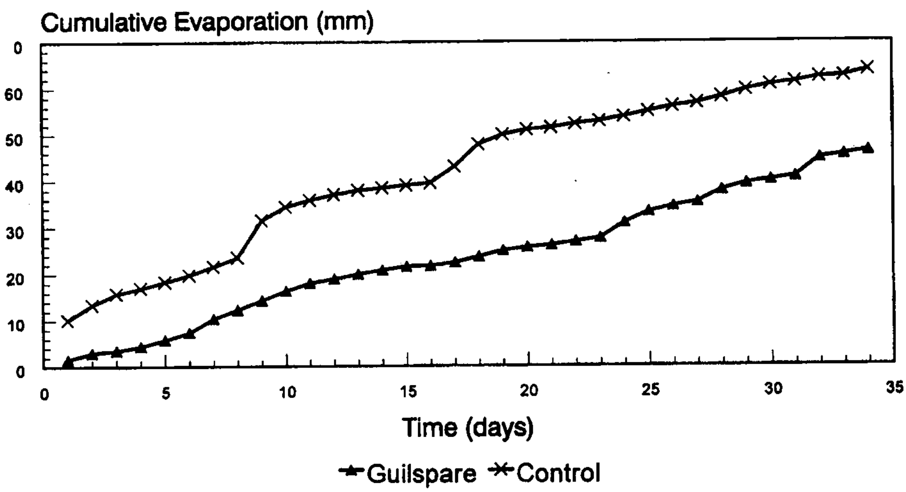

Figure 1. Effect of Polymer Treatment on Cumulative Evaporation from soil columns, irrigated at a rate of 3 mm/day every 4 days.

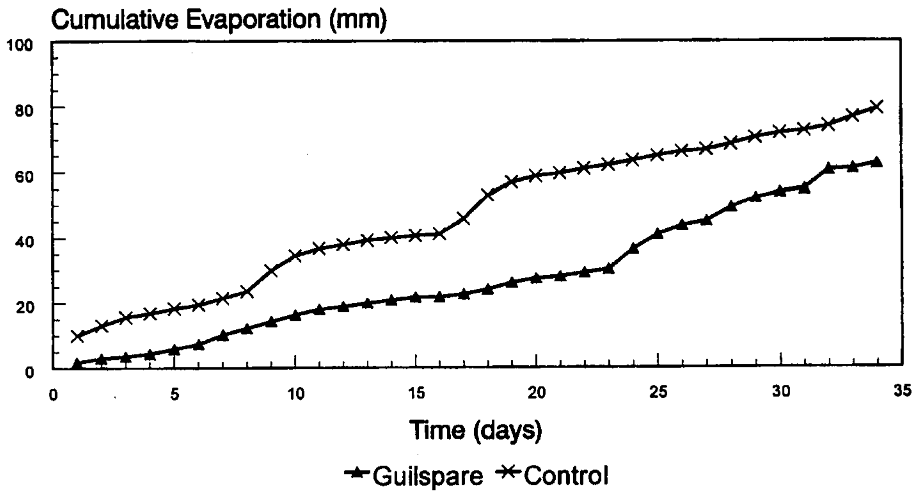

Figure 2. Effect of Polymer Treatment on Cumulative Evaporation from soil columns, irrigated at a rate of 6 mm/day every 8 days.

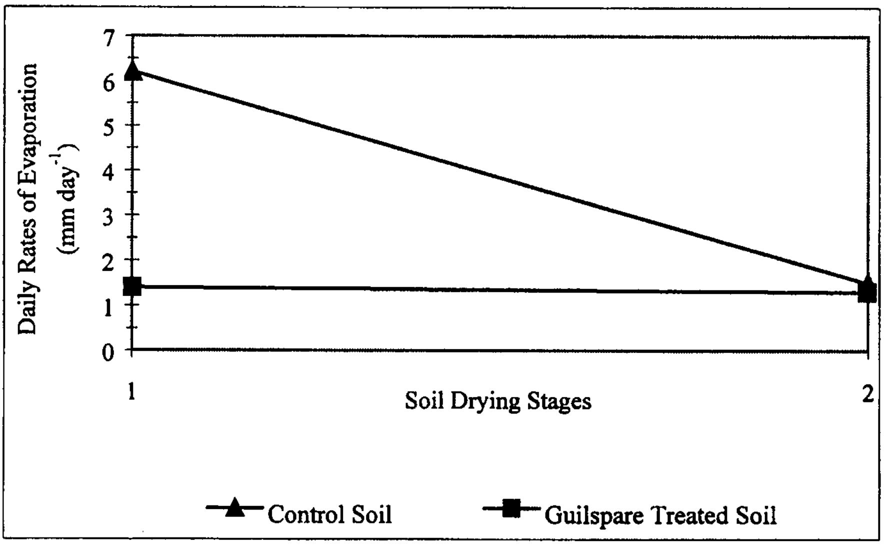

Figure 3. Daily Rates of Evaporation during Drying Stages I and II, from Guilspare treated and control soils.

Water Treatment

Utilization of Wastewater for The Next Millennium

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UTILIZATION OF WASTEWATER FOR THE NEXT MILLENNIUM

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ABSTRACT

Treated wastewater could be a major water resource in Kuwait. This resource is underutilized due to the presence of pollutants and pathogens. The efficiency of the current treatment of wastewater in Kuwait is effective but not stable and cumbersome. In recent years advanced wastewater treatment systems such as filtration, Microfiltration (MF) and Reverse Osmosis (RO) has received great interest as many secondary and tertiary treatment plants are being upgraded for removal of pollutants and other constituents that are not removed in the conventional wastewater treatment plants. The advanced wastewater systems are cost–effective and provide water with excellent quality that could be used for greenery, agriculture and for any future industrial applications. These advanced wastewater treatments have been tested and evaluated under three different studies carried out by Kuwait Institute for Scientific Research as polishing technique for tertiary treated wastewater in Ardiya plant at Kuwait.

This paper discusses the performance of the conventional treatment, MF and RO systems and their benefit to the water resources in Kuwait.

Key words : Pollutants, Microfiltration, Filtration, Reverse Osmosis, Agriculture

INTRODUCTION

The importance of wastewater reuse in the field of water resources management is now commonly acknowledged. Water resources in Kuwait is scares and limited, moreover, water demand is in the increase. Supplying Water to urban areas in Kuwait requires major capital investment in water resources development, treatment, storage, and distribution. It is evident that the demand for water in Kuwait is enormous and will continue to grow rapidly due to an increase in population, industrialization, urbanization, and agricultural development.

The main challenge facing Kuwait in the next decade is the availability of freshwater. At present, Kuwait is depending on three water resources and they are seawater desalination, brackish groundwater and treated wastewater. Freshwater is produced mainly from seawater desalination. Brackish water is mainly used to adjust the salinity of the distilled water produced by desalination plants. The wastewater is treated to secondary/tertiary level and mainly discharged to the sea with very limited amount being used for greenery purposes. The strategic master plan now being prepared for greenery development in nonresidential areas will focus on avoiding further use of brackish water [1]. Emphasis, instead, will be directed towards the use of tertiary treated wastewater [2].

Municipal wastewater contains approximately 99.1 % water [3]. The small fraction of solids includes organic and inorganic, suspended and dissolved matter. The removal of various contaminants from the liquid depends on the nature of the impurities and their concentrations. The coarse and settleable organic and inorganic solids are generally removed in primary treatment units that include bar screens, grit removal, and sedimentation facilities. The removal of dissolved organic is readily achieved in biological or chemical treatment processes that may be added to the primary treatment. The combined system is used in a secondary treatment plant. Many unit operations and processes may be added to the existing primary or secondary treatment systems to achieve the removal of nutrients and other contaminants. This is called tertiary treatment or advanced wastewater treatment [4].

The wastewater treatment process in Kuwait consists mainly of screening, grit removal and biological treatment by activated sludge. Sludge is thickened and digested before being further thickened and air-dried on open sludge beds where the sewage gas from the digestion process is burned off. All effluent from the treatment plant receives tertiary treatment. The flow passes of contact tanks where it is chlorinated and then passed through sand filters to remove the last remaining impurities and suspended materials.

Membranes separation technology such as MF and RO are increasingly adopted in the field of water and wastewater treatment and has shown good performance for removing many kind of contaminants. MF is a successful filtration technique that is already applied to surface seawater, brackish water and industrial and urban wastewater. MF membrane with a nominal pore size of 0.03 μm could remove turbidity of almost 100%, coliform bacteria more than 99% and viruses at least 85% [5]. RO is one of the most successful membrane separation techniques, which characterized by low energy consumption [6]. Energy consumption of the RO depends on the salinity of the feed water [7]. Consequently, properly treated wastewater effluent of low salinity (1000-1500 ppm) can be considered an excellent feed for the RO system. This advanced wastewater systems are cost–effective and has excellent quality of water could be used for greenery, agriculture and for any future industrial applications.

PROCESSES OF TREATMENT SYSTEMS

Kuwait has identified treated effluent as valuable resource for irrigation and beautification of Kuwait, therefore, the Kuwait Institute for Scientific Research implemented a three different study to assess the viability of using conventional treatment, MF and RO techniques for polishing treated wastewater. These studies were carried out at Ardiya wastewater plants.

A schematic diagram of the conventional treatment system is shown in Figure 1.The filtration system consists of two 1-m^3 glass reinforced plastic (GRP) filter tanks (multi-media sand filter), feed and backwash pumps, filtrate tank (pipes and valves), two chemical dosing pumps and agitators for Ferric Cloroulphate ($FeClSO_4$) and Polyelectrolyte. The buffer tank is fed with tertiary treated wastewater from the outfall of the Ardyia wastewater plant. The $FeClSO_4$ and Polyelectrolyte are dosed into the feed stream before entering the buffer tank. The feed pump delivers the effluent to the sand filters, where the water is filtered and fed to the filtrate tank. The Silt Density Index (SDI) is continuously monitored. The system is capable to produce 1 m^3/h. filtrate water.

The MF system used was a Memtec model 2M15C consisting of two (Memcor) modules with a total membrane surface areas 30 m^2. The modules contained polypropylene hollow fibers with 0.2 mm nominal pore size. The system was operated in direct flow mode, with water flowing from the outside of the fibers to the inside. The filtrate control valve was adjusted at least once per day to maintain a set filtrate flow rate. As the membrane fouled during the day, the flowrate would drop slightly. The unit was set for automatic, air-assisted backwash initiated automatically by a timer. The unit was chemically cleaned when Tran-membrane pressure (TMP) exceeded

140 kPa. The tertiary effluent is pumped by pool pumps to a break tank after passing through a coarse pump strainer and injected with 2.5 ppm of sodium bisulphite ($NaHSO_3$) to remove any traces of chlorine. The feed stream is fed to a direct flow MF membrane,where filtration takes place. A filtrate tank of 250 *l* is installed at the product side of the system to maintain a steady flow of filtrate water to the silt density index (SDI) system during the measuring time. Figure 2 shows schematic diagram of MF system.

RO is a demineralization process applicable to the production of high-quality water to meet the drinking water standards. The process consists of permeating liquid through a semi-permeable membrane at pressures up to (80 bar). The membranes reject most of the ions and molecules while permitting acceptable rates of water passage. A small RO plant of (2000 IGD) capacity is operated for 140 days at the Ardiya wastewater plant. It consist of pressure pump (10 bar), one pressure vessel with three Toray modules, fitted in series inside one pressure, cleaning/flushing pump and permeate tank. Flirted effluent is fed to the RO system, where the effluent is processed and collected in permeate thank. Figure 3 shows schematic diagram of RO system.

RESULTS AND DISCUSSION

All these systems utilized tertiary treated wastewater with an average TDS and SDI of 938 mg/l and greater than 6, respectively. Data obtain from the filtration system indicate that an addition of 2-4 Fe^{+3} PPM, 0.5-1.0 PPM cationic polyelectolyte and 3-5 ppm sanitizing agent (peroxyacetic acid) yield an acceptable filtrate water quality . The SDI values of the filtrate fluctuate between 4-6%.. Whereas, the quality of filtrate water for the RO and MF remained steady with an average SDI value 3% and $< 2\%$ respectively, without using any chemical addition in the feed. No significant reduction in chemical oxygen demand (COD) and biological oxygen demand (BOD) were noticed in the filtrate water of the conventional system, whereas these value were reduced substantially in the product of RO and MF systems as shown in table 1. The range of the recovery profile of the RO system was 17-25%, the permeate conductivity was between 20 and 30μS/cm, which considered a very good quality water. The feed pressure (9 bar) is also considered very low compared with the pressure applied to seawater desalination system. Bacteriological, viral and chemical analyses of the product indicate that the water is free from microorganisms and other biological matter. Economical assessment of these treatment systems indicates that the MF and RO systems are viable and reliable in production of an excellent quality of water free from impurities, when compared with conventional system. The reclaimed unit water costs for $9m^3/d$ RO plant are estimated to be Fils130/ m^3 [8], whereas they are Fils12.264/ m^3 for MF system as indicated in tables 1.

Table 1. Characteristics of the Feed and product of the three treatments systems.

Parameter	Average			
	Tertiary	Conv.Treatment	MF	RO
TDS (mg/l)	938	938	938	30
Filtrate SDI	> 6	5	3	<2
Total count	$4.3x10^3$	$4.0 x10^3$	$3.2 x10^3$	$2.5 x10^3$
BOD (mg/l)	8	3.8	2.6	1.2
COD (mg/l)	56	20	11.25	13.47
Fecal-Coli	5.0	0	0	0
Cost (fils/m^3)	120	28.153	12.264	130

CONCLUSION

The study presented in this paper indicates that the RO and MF techniques are technically and economically viable for treating wastewater, compared with conventional treatment. Moreover they are capable of producing water of stable quality of treated wastewater at lower cost. The product water from these technique can be utilized for greenery and other purposes which will reduce the increasing demand for freshwater, that produced at a rather high cost by conventional thermal processes (i.e., MSF). Also The product water, if it is acceptable from bacteriological, virological and organic point of views, can be recommended for direct injection in the underutilized brackish water network.

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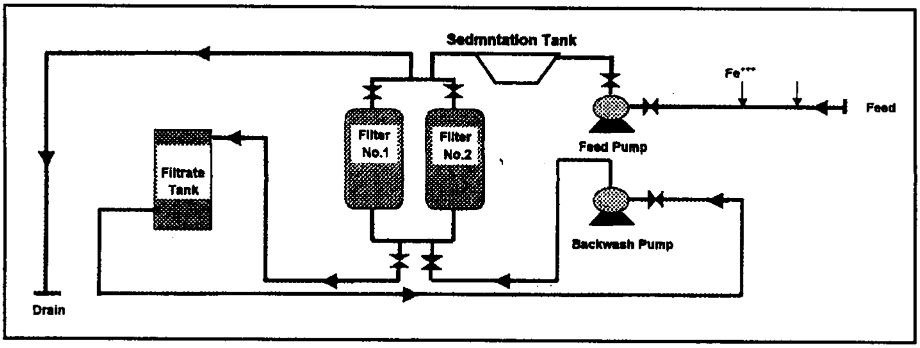

Fig. 1. Schematic diagram of conventional wastewater treatment system.

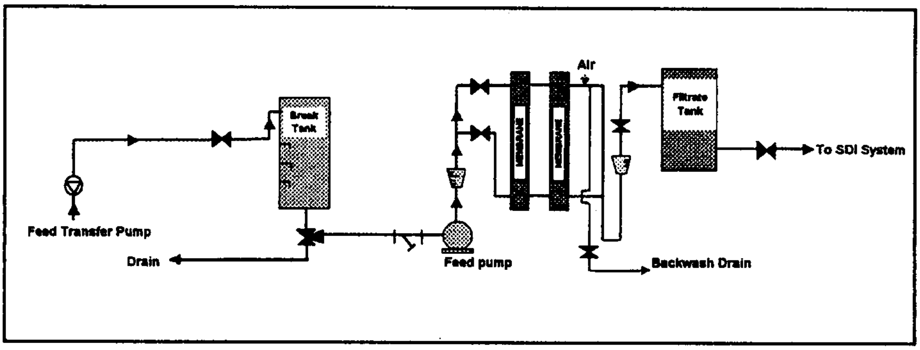

Fig. 2. Schematic diagram of microfiltration system

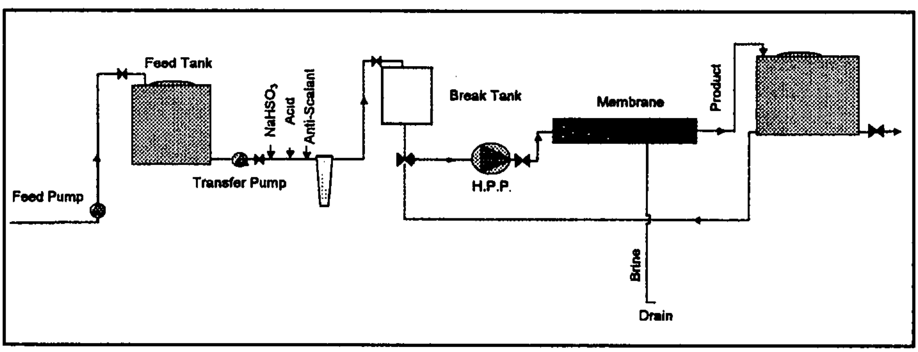

Fig. 3. Schematic diagram of RO system

“Wastewater Treatment and Reuse in the State of Qatar”

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WASTEWATER TREATMENT AND REUSE IN THE STATE OF QATAR

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ABSTRACT

"Wastewater Treatment and Reuse Technologies in the State of Qatar" paper is presented at The 5th Gulf Water Conference "Water Security in the Gulf" that will be held in Doha, Qatar in March 2001.

This paper addresses the fundamentals of wastewater treatment and reuse technologies and their relevant practices in the State of Qatar. This highly important subject is covered through presentation, discussion, and analysis of the following 4 topics:

INTRODUCTION

- **Topic 1** ***Wastewater Treatment Technologies***

Under this topic, the basic wastewater treatment operations and processes are explained and illustrated. The illustration addresses the objectives and needs for treatment prior to application. Understanding the nature and characteristics of the wastewater is important at this stage, therefore, the necessary information on wastewater sources and quality are given.
The two major processes; aerobic and anaerobic processes and their applications are clarified and evaluated. All the necessary operations associated with the wastewater treatment are addressed, such as; preliminary, primary, secondary and tertiary treatment operations.
As a result, the different and various treatment technologies are presented, discussed, and evaluated such as; Trickling Filters, Rotating Biological Contactors, Activated Sludge and its modifications, Wastewater Stabilization Ponds, ...etc.
Attention has been given to the sludge production, treatment, disposal and reuse issues.

- **Topic 2** ***Applications of Wastewater Treatment and Reuse in the State of Qatar***

Qatar has a wide experience in the fields of wastewater collection, conveyance, treatment, and disposal and reuse. This topic provides an overview on the wastewater collection, treatment and reuse practices in Qatar and evaluates the different systems that are employed for wastewater treatment and reuse. Technical information on the existing systems of treated effluent disposal and reuse will be provided and means of maintenance programs of the reuse system will be discussed.

- **Topic 3** ***Applications of Geographic Information Systems in the Field of*** **Wastewater Treatment and Reuse in the State of Qatar**

The Geographic Information Systems (GIS) is an organized and efficient source of drainage information, as it is responsible for creating and maintaining a detailed Wastewater Collection, Conveyance, Treatment and Reuse database that provides useful information on the collection network, conveyance systems and disposal and reuse systems.

WASTEWATER TREATMENT TECHNOLOGIES

1. OBJECTIVE

The principal objective of wastewater treatment is generally to allow human and industrial effluents to be disposed of without danger to human health, or unacceptable damage to the natural environment.

The main concept in wastewater treatment is based on the need to reduce organic and suspended solids to improve public health and limit pollution of the environment. Pathogen removal is of prime concern if treated effluents are to be used for irrigation. Although irrigation with wastewater is, in itself, an effective form of wastewater treatment (such as slow rate land treatment), some degree of treatment must be provided to untreated municipal wastewater before it can be used for agriculture or landscape irrigation.

Generally, treatment of wastewater is practiced for the following reasons:

1. protection of public health and improve hygienic conditions within communities.
2. prevention of pollution of the environment, mainly, receiving water bodies, such as surface and groundwater's.
3. Production of treated effluents that will not cause damage to crops and soils.

2. LEVEL OF TREATMENT

The level of treatment required is highly dependent on the end use, basically:

1. irrigation purposes "vegetables, lawns, fodder crops, forestry,..."
2. discharge to streams, wadis, reservoirs, ...etc.
3. groundwater recharge

In general, the level of treatment required prior to irrigation of many crops is often not greater than, and is sometimes less than, the level of treatment required for discharge to receiving waters. The level of treatment required for agricultural and landscape uses depends on the soil characteristics, the crop irrigated, the type of distribution and application systems, and the degree of public exposure.

3. MUNICIPAL WASTEWATER CONSTITUENTS AND COMPOSITIONS

Wastewater is the general term applied to the liquid waste collected in sanitary sewers and treated in a municipal wastewater treatment plant. Municipal wastewater is composed of domestic wastewater, industrial wastewater, and infiltration inflow. Domestic wastewater refers to the water collected from the community after it has undergone a variety of uses in residences, commercial buildings, and institutions. Industrial wastewater, on the other hand, comes from manufacturing plants.

The physical properties and the chemical and biological constituents of wastewater are important parameters in the design and operation of collection, treatment, disposal, and reuse facilities and in the engineering management of environmental quality. The constituents of concern in municipal wastewater treatment are listed in table (1).

Table (1) Constituents of concern in wastewater treatment

Constituents	Measured Parameters	Reason for Concern
Suspended Solids	Suspended Solids, including volatile and fixed solids	Suspended solids can lead to the development of sludge deposits and anaerobic conditions when untreated wastewater is discharged in the aquatic environment. Excessive amounts of suspended solids cause soil plugging in irrigation systems
Biodegradable organics	Biochemical Oxygen Demand, Chemical Oxygen Demand	Composed principally of proteins, carbohydrates, and fats. If discharged to the environment, their biological decomposition can lead to the depletion of dissolved oxygen in receiving waters and to the development of septic conditions.
Pathogens	Indicator Organisms, Total and Fecal Coliforms Bacteria	Communicable diseases can be transmitted by the pathogens in wastewater: bacteria, virus, parasites.
Nutrients	Nitrogen, Phosphorous, Potassium	Nitrogen, phosphorous, and potassium are essential nutrients for plant growth, and their presence normally enhances the value of the water for irrigation. When discharged to the aquatic environment, nitrogen and phosphorous can lead to the growth of undesirable aquatic life. When discharged in excessive amounts on land, nitrogen can also lead to the pollution of groundwater.
Hydrogen Ion	pH	The pH of wastewater affects metal solubility as well as alkalinity of soils. Normal pH range in municipal wastewater is 6.5-8.5, but presence of industrial waste can alter pH significantly.
Heavy Metals	Specific Elements (e.g. Cd, Zn, Ni, Hg, ..etc.)	Some heavy metals accumulate in the environment and are toxic to plants and animals. Their presence may limit the suitability of the wastewater for irrigation.
Dissolved Inorganics	Total Dissolved Solids, Electrical Conductivity, Specific elements (e.g. Na, Ca, Mg, Cl, B)	Excessive salinity may damage some crops. Specific ions such as chloride, sodium, boron are toxic to some crops. Sodium may pose soil permeability problems.
Stable (Refractory) Organics	Specific Compounds (e.g. phenols, pesticides, chlorinated hydrocarbons)	These organics tend to resist conventional methods of wastewater treatment. Some organic compounds are toxic in the environment, and their presence may limit the suitability of the wastewater for irrigation

Composition refers to the actual amounts of physical, chemical, and biological constituents present in the wastewater. The composition of untreated wastewater and the subsequently treated effluents depends upon the composition of the municipal water supply, the number and type of commercial and industrial establishments, and the nature of the residential community. Consequently, the composition of wastewater often varies widely among different communities. Typical data on the composition of untreated domestic wastewater are presented in table (2).

Table (2) Typical Composition of Untreated Municipal Wastewater

Constituent	Unit	Concentration			Qatar Average
		Strong	Medium	Weak	
Solids					
Total	mg/l	1200	720	350	2800
Dissolved	mg/l	850	500	250	2500
Suspended	mg/l	350	250	100	300
Bio-Chemical Oxygen Demand (BOD5)	mg/l	400	220	110	300
Chemical Oxygen Demand (COD)	mg/l	1000	500	250	400
Total Nitrogen-N	mg/l	85	40	20	50
Ammonia (NH4-N)	mg/l	35	15	8	30
Total Phosphorous-P	mg/l	15	8	4	15
Phosphate (PO4-P)	mg/l	10	5	1	10
Fat, Oil and Grease (FOG)	mg/l	150	100	50	50

4. CONVENTIONAL WASTEWATER TREATMENT PROCESSES

Conventional wastewater treatment consists of a combination of physical, chemical, and biological processes and operations to remove solids, organic matter and, sometimes, nutrients from wastewater. The general terms used to describe different degrees of treatment of increasing treatment levels are:

- Preliminary
- Primary
- Secondary
- Tertiary and/ or Advanced

A disinfection step to remove pathogens, sometimes, follows the last treatment step.

4.1 Preliminary Treatment

The objective of preliminary treatment is the removal of coarse solids and other large materials often found in raw wastewater. Removal of these materials is necessary to enhance the operation and maintenance of subsequent treatment units. Typically, preliminary treatment operations include:

Typical Preliminary Treatment Operations

Unit	Objective	Type	Disadvantage	Disposal
Screening	- Protect plant against bulky matter (papers, plastics, rags)	Manual	Frequent cleaning operations not hygienic	Recovered Dumped Buried Incinerated
		Mechanical	Bulky	
Comminution	- Disintegrate solid matter Carried in the wastewater	Static	Increase pollution load Liable to clogging	No Extraction
		Rotary	Costly to maintain	
Grit Removal	- Elimination of gravel, sand, And other mineral particles (@>0.2 mm) - Protects pumps and other Equipment against abrasion - Avoid overloading subsequent Treatment stages	Constant Velocity	Low Yield Efficiency depends on flow Rate	Recovered Dewatered Dumped Buried Reused
		Vortex Aerated Grit Tank	Requires air & energy	
Fat, Oil & Grease (FOG) Removal	- Remove oil, fats, and grease - Protect against grease deposits - Protect biological process	Separate Grit & FOG Combined Grit & FOG		Recovered Dumped

4.2 Primary Treatment

The objective of primary treatment is the removal of settleable organic and inorganic solids by sedimentation, and the removal floating materials (scum) by skimming.

Approximately, 25 to 50% of the incoming Biochemical Oxygen Demand (BOD_5), 50 to 70% of the Total Suspended Solids (TSS), and 65% of the oil and grease are removed during primary treatment. Some organic nitrogen, organic phosphorous, and heavy metals associated with solids are also removed during primary sedimentation, but colloidal and dissolved constituents are not affected.

Additional advantages of the primary treatment are to:

- provide early detection to the unusual characteristics of the incoming wastewater which might influence the efficiency of the subsequent treatment processes.
- equalize the incoming wastewater strength by the retention time provided.

4.2.1 Types

The most common unit operations employed in the primary treatment process are:

- Primary Settling Tanks

  Settled solids (primary sludge) are normally removed from the bottom of the primary sedimentation tanks or clarifiers by sludge rakes that scrape the sludge to a central well from which it is pumped to sludge processing units.

- Anaerobic Stabilization Ponds.

  Anaerobic ponds are used to treat domestic and high strength organic wastewater that has high concentrations of solids. Typically, anaerobic ponds are earthen basins with depths of up to 9 m. The increased depth of deeper ponds helps to conserve energy and maintain anaerobic conditions. Usually, these ponds are anaerobic through their depth, except for an extremely shallow surface zone. Added wastes settle to the bottom of the pond and the partially clarified effluent is commonly discharged to another treatment process for further treatment. Anaerobic ponds generate odors due to the formation of hydrogen sulphide gas during the anaerobic digestion of the settled solids.

- Imhoff Tanks

  Imhoff tanks are two-storey tanks, where the incoming wastewater flows through the upper compartment, allowing solids to settle to the bottom of the chamber, which is in the shape of a hopper. At the bottom of the hopper, the solids pass through a baffled outlet into the lower chamber in which anaerobic digestion of sludge takes place.

Imhoff tanks, because of the cost of constructing the very deep tanks, are generally considered economical only for relatively small communities, where their operating simplicity offers some advantages over separate sedimentation and digestion tanks.

Typical Primary Treatment Operations

Unit	Objective	Type	Disadvantage	Disposal
Primary Sedimentation	- Settling of organic & inorganic - Reduce Organic load on subsequent treatment processes	Square & Rectangular	-	Recovered Thickened Digested Dewatered Reused
		Circular	Bulky	
Anaerobic Ponds	- Settling of organic & inorganic - Reduce Organic load on subsequent treatment processes	Square & Rectangular	Generate Odors	Recovered Reused
Imhoff Tank	- Settling of organic & inorganic - Reduce Organic load on subsequent treatment processes - Digestion of Primary Sludge	Square & Rectangular	High Construction Cost	Recovered Reused

Several reasons may affect the efficiency of the primary treatment stage and cause poor suspended solids removal, such as:

- Hydraulic overloading. High hydraulic loads in excess of the design rates may cause scouring of sludge and solids carry over.
- Short-circuiting. Poor hydraulic design and inappropriate baffles may cause less retention time than the designed and required time.
- Industrial wastes. Certain industrial wastes, such as those high in carbohydrates or oil concentrations, may retard settling.

4.3 Secondary Treatment

The objective of the secondary treatment is the further treatment of the effluent from primary treatment to remove the residual organic and suspended solids. In most cases, secondary treatment follows primary treatment and involves the removal of biodegradable dissolved and colloidal organic matter using aerobic and / or anaerobic biological treatment processes. Aerobic biological treatment is performed in the presence of oxygen by aerobic microorganisms (principally bacteria) that metabolize the organic matter in the wastewater, thereby producing more microorganisms and inorganic end-products (principally CO_2, NH_3, and H_2O). Anaerobic biological treatment is performed in the absence of oxygen by anaerobic microorganisms, producing less microorganisms and inorganic end products (principally CH_4, H_2S, CO_2, and NH_3).

Several aerobic and/ or anaerobic biological processes are used for secondary treatment. These processes differ primarily in:

- the manner in which oxygen is supplied to the microorganisms, and
- the rate of which organisms metabolize the organic matter

... for the purpose of this paper, secondary biological wastewater treatment processes are grouped into two processes:

- High Rate Biological Processes
- Low Rate Biological Processes

4.3.1 High Rate Biological Processes

High rate biological processes are characterized by relatively small reactor volumes and high concentrations of microorganisms compared with the low rate processes. Consequently, the growth rate of new organisms is much greater in high rate systems because of a well controlled environment. The microorganisms must be separated from the treated wastewater by sedimentation to produce the clarified secondary effluent. The sedimentation tanks used in secondary treatment, often referred to as secondary sedimentation tanks or clarifiers, operate in the same basic manner as the primary tanks. The biological solids removed during secondary sedimentation (secondary sludge), are normally combined with primary sludge for sludge processing.

Common high rate processes include:

- Activated Sludge Processes
- Trickling Filters
- Rotating Biological processes

4.3.1.1 Activated Sludge Processes

This is a biological treatment process where the organic compounds which are present in wastewater are oxidized by micro-organisms to water and carbon dioxide. This waste conversion takes place in a reactor, or basin. This process is quite flexible and many modifications have been developed. The following are the major activated sludge modifications:

- **Conventional Plug Flow**

In the conventional activated sludge process, organic waste, after primary clarification, is introduced into an aeration tank where a suspended "mixed liquor" consisting of micro-organisms, dissolved oxygen, organic compounds and nutrients is continuously aerated. The term "activated" refers to the micro-organisms which degrade organic compounds in the waste to water and carbon dioxide. After a specified period of time the mixture is passed on to a clarification process. Here, the suspended solids in the mixed liquor settle out to form a biomass, or sludge, at the bottom of the tank. Most of the settled sludge is recycled back into the aeration tank to ensure a constant population of mature acclimated micro-organisms. A portion of the sludge is disposed of, or "wasted".

- **Extended Aeration**

The extended aeration process is similar to the conventional operation except that it requires a lower organic loading and longer aeration time. Extended aeration is often used without primary clarification. Extended aeration plants are typically run with long sludge ages, and they can also provide ammonia nitrogen removal (nitrification). However, it requires much more energy for treatment than the conventional activated sludge process.

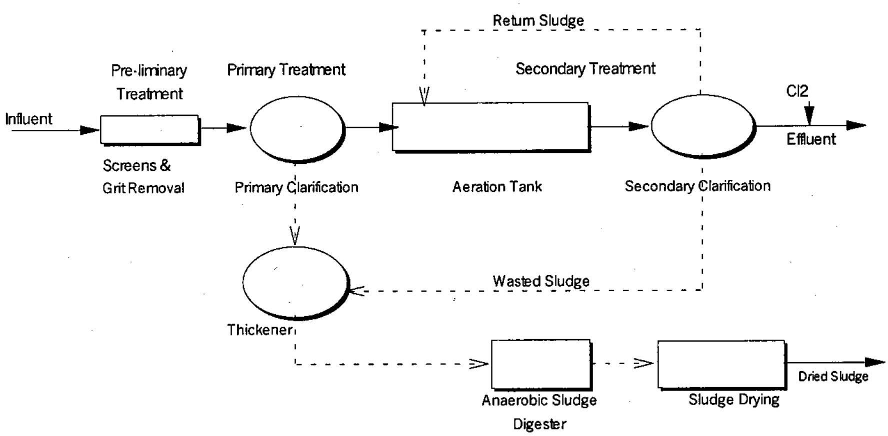
- **Oxidation Ditch**

This process differs from the conventional plug flow in its set-up. The oxidation ditch consists of a ring or oval shaped channel equipped with aerators. Typical oxidation ditches have long detention and solids retention times and are often used without primary clarification. Because dissolved oxygen levels can be varied as wastewater circulates around the race track, oxidation ditches can also provide nitrification and partial denitrification. This process can be considered as a variation of the extended aeration process.

- **Sequencing Batch Reactor (SBR)**

In this process, all steps in the activated sludge process occur in a single reactor. Wastewater fills the tank, the process runs its course and clarified effluent is decanted from the top of the tank. In between batches, a portion of the settled sludge is wasted. When wastewater is filling the tank, mechanical aerators mix the tank contents for a period of time to reactivate the sludge leftover from the previous cycle. Similarly to oxidation ditches, (SBR) has process flexibility to provide nitrification, denitrification and phosphorous removal.

Fig. (2) Activated Sludge Process - Process Flow Diagram



The main advantages of the activated sludge process are:

- High performance and reliability.
- High degree of flexibility.
- Under normal conditions, no obnoxious odors are generated from the system.

The main disadvantages are:

- High operational and maintenance costs.
- Highly sensitive to hydraulic and organic shocks.
- Large quantities of sludge are produced and have to be handled.

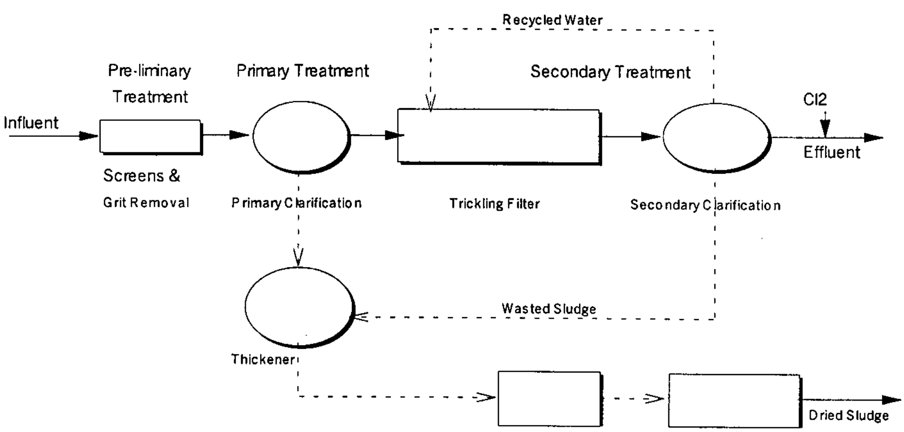
4.3.1.2 Trickling Filter

A trickling filter consists of a bed of highly permeable media through which wastewater seeps. The filter media generally consists of rocks or plastic. The micro-organisms attach themselves to this filter media to form a slime layer that adsorbs food and nutrients from the moving wastewater as it passes through the filter bed. Typically in the trickling filter process, the wastewater is distributed evenly over the top of the medium and is allowed to slowly move down through the bed. Treated wastewater and any biological solids that have become detached from the media are collected in an under drain system. This under drain also allows for either natural or forced draft air convection through the media bed to supply oxygen to the micro-organisms. The collected liquid then flows to a secondary settling tank where solids are separated from the treated wastewater. In practice, a portion of the liquid collected from the under drains or settled effluent is recycled.

4.3.1.3 Rotating Biological Contactors (RBC)

This process consists of a series of closely spaced circular disks of polystyrene or polyvinyl chloride. These disks are housed in chambers or basins through which wastewater flows. The disks are submerged in the wastewater and rotated slowly through it. During operation, biological growths become attached to the wetted surface of the disks to form a slime layer which biologically treats the wastewater. The rotation of the disks alternately contacts the slime layer with the organic material in the wastewater and then with the atmosphere for adsorption of oxygen. Disk rotation maintains an aerobic condition for the biomass (slime). As in the conventional trickling filter, this process (RBC) is followed by a secondary settling tank where solids are separated from the treated wastewater.

Fig. (3) Trickling Filter Process - Process Flow Diagram



The main advantages of the trickling filter process are:

- Relatively moderate operational and maintenance costs.
- Relatively less solids are generated than the activated sludge process.
- High degree of nitrification is achieved.

while the disadvantages are:

- Obnoxious odors may be generated.
- The process may provide breeding place for flies and mosquitoes.
- Less efficiency in organic matter removal compared with the activated sludge process.

High rate biological treatment processes, in combination with primary sedimentation, typically remove 85% of BOD_5 and SS originally present in the wastewater and some of the heavy metals. Activated sludge generally produces an effluent of slightly higher quality, in terms of these constituents, than trickling filters and RBC's. When coupled with a disinfection step, these processes provide substantial but not complete removal of bacteria and viruses. However, only a very small amount of phosphorous, nitrogen, non-biodegradable organics, and dissolved minerals can be removed through these processes.

4.3.2 Low Rate Biological Processes

Low rate biological processes are characterized by lower conversion rates as compared to the high rate biological systems discussed previously. In most low rate biological processes the microorganisms are not usually separated from the liquid in a separate step. In small treatment plants, primary sedimentation prior to low rate processes is often omitted.

Commonly used low rate biological processes include:

- Facultative Wastewater Stabilization Ponds
- Aerated Lagoons
- Aquatic Treatment Systems (Constructed Wetlands, Macrophyte Ponds)

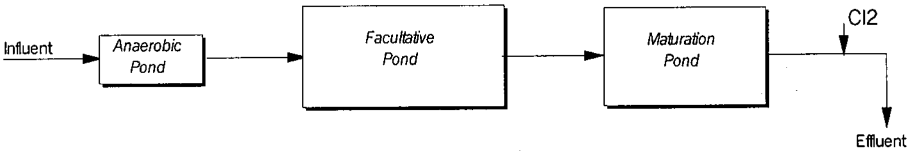


Fig. (4) Wastewater Stabilization Ponds (WSP)

4.3.2.1 Facultative Wastewater Stabilization Ponds

Wastewater Stabilization Ponds use biological treatment to stabilize the wastewater. These ponds have long detention times and do not have a mechanical mixing or aeration.

Facultative ponds are also called aerobic-anaerobic stabilization ponds because the stabilization of wastes is achieved by a combination of aerobic, anaerobic and facultative bacteria. Conventional facultative ponds are earthen basins that contain three zones. The first is a surface zone where bacteria and algae exist in a symbiotic relationship to aerobically degrade organic matter. The second zone is the intermediate layer where decomposition of organic wastes is carried out by facultative bacteria in a partly aerobic and partly anaerobic environment. The bottom is the third zone where bacteria anaerobically digest the sludge layer formed by the accumulation of large solids that have settled.

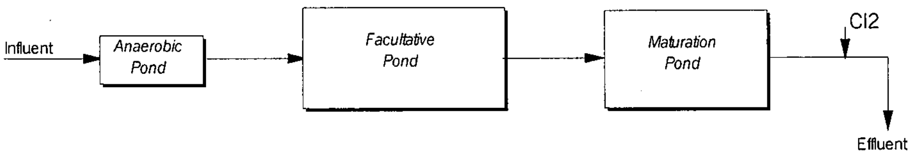


Fig. (4) Wastewater Stabilization Ponds (WSP)

4.3.2.2 Aerated Lagoons

Aerated lagoons are large basins in which natural biological processes convert organic wastes to water and carbon dioxide. A constant supply of oxygen from mechanical aeration is provided to maintain aerobic conditions in the basin. Aerobic conditions are essential in order to maintain oxidation process which do not produce offensive odors. Sludge that is produced settles in certain unmixed zones of the lagoons and forms a blanket on the bottom. The sludge then becomes anaerobic and a self-digestion process takes place. This self-digestion reduces the volume of the sludge considerably, but the lagoon requires sludge removal periodically to maintain hydraulic capacity. The main difference between aerated lagoons and the activated sludge is that there is no secondary clarifier, and, consequently, the sludge in aerated lagoons is not recycled. Several types of aerated lagoons are:

- Complete Mix Aerated Lagoons
- Partial Mix Aerated Lagoons

4.3.2.3 Aquatic Treatment Systems (Constructed Wetlands, Macrophyte Ponds)

These treatment systems are ponds which incorporate floating, submerged or emergent aquatic plant species. These aquatic species (such as; hyacinth, duckweed, cattails, bulrushes, reeds, and sedges) take up large amounts of inorganic nutrients (especially N and P) and heavy metals (such as Cd, Cu, Hg, and Zn) as a consequence of the growth requirements and decrease the concentration of algal cells through light shading by the leaf canopy and, possibly, adherence to gelatinous biomass which grows on the roots.

Aquatic treatment systems are most suitable in upgrading final effluents from wastewater stabilization ponds. They best perform in temperate climates where the air temperatures do not vary dramatically.

Sludge which is produced in these systems consist mainly of wastewater solids and plants detritus. It accumulates on the bottom of the ponds and need to be removed annually.

4.3.3 Comparison of High and Low Rate Biological Processes

Low rate biological processes are less costly and require less process control than high rate processes; however, in some of the low rate processes because solids are not separated from the liquid, the quality of the effluent is substantially lower than that from high rate processes. The higher suspended solids in wastewater stabilization pond effluent due to algal growth is a good example. Nevertheless, low rate biological processes provide a sufficient degree of treatment prior to reuse for agriculture for which secondary treatment is required. Low rate processes such as wastewater stabilization ponds, also provide considerable pathogen and nitrogen removal, depending on design, temperature, and detention time involved. Low rate processes are particularly important in many wastewater treatment and irrigation applications in developing countries.

4.3.4 Pathogen Removal

The effluent from secondary treatment will normally contain large counts of excreted pathogens such as bacteria, viruses, intestinal nematode eggs, ...etc. Disinfection is a treatment process to selectively destroy disease-causing organisms. The disinfection process does not destroy all unwanted organisms, instead it lowers the organisms concentration below a predetermined limit based on effluent disposal or reuse requirements. Different types of disinfectants are used, such as; chlorine, ozone, ultraviolet, ...etc.

A more important issue is the removal of intestinal nematode eggs. For this purpose, maturation ponds were used extensively. Maturation ponds are low rate stabilization ponds with a long retention time. Long retention time, natural die-off of bacteria and many unfavorable conditions help to improve the microbial quality of the incoming treated wastewater to the maturation ponds.

4.3.5 Anaerobic Treatment

Anaerobic digestion by definition is the conversion of organic compounds into methane and carbon dioxide by bacteria without the aid of oxygen.

Removal of organic materials occurs by microbial conversion, first to hydrogen, carbon dioxide, and acetic acid, which then act as precursors to the final by-products of methane and carbon dioxide.

Three different but related groups of bacteria are involved, namely:

- The hydrolytic fermentative bacteria. These bacteria hydrolyze a wide range of complex polymeric substrates to form simple organic compounds, carbon dioxide and hydrogen.
- The acetogenic bacteria. These bacteria convert these organic fermentation products into acetate, hydrogen, and carbon dioxide.
- The methanogenic bacteria. These bacteria convert acetate and hydrogen to methane and carbon dioxide.

The anaerobic method of treating high strength domestic and industrial wastewaters offer a number of significant advantages over conventional processes, namely:

- Environmental acceptance. The process, by its very nature, is totally enclosed and does not produce any environmental nuisance.
- Low sludge production. In anaerobic digester, due to the very long solids retention time and consequent low growth rate, the solids production is also extremely low.
- Energy production. In anaerobic treatment most of the carbon in the waste is available for methanogenesis in which the biogas (CH_4) is the main by-product. The production of biogas is generally in excess of that needed to operate the anaerobic treatment system, and can be utilized to generate power for other on site services.

4.4 Tertiary and/ or Advanced Treatment

Tertiary and/ or advanced wastewater treatment is employed when specific wastewater constituents must be removed, but can not be removed by secondary treatment. As shown in fig. (1), individual treatment processes are necessary to remove nitrogen, phosphorous, additional suspended solids, refractory organics, heavy metals, and dissolved solids. Because advanced treatment usually follows high rate secondary treatments, it is sometimes referred to as tertiary treatment. However, advanced treatment processes are sometimes combined with primary or secondary treatment (e.g. chemical addition to primary clarifiers or aeration basins to remove phosphorous) or used in place of secondary treatment (e.g. overland flow treatment of primary effluent).

Where the probability of public exposure to the treated wastewater or residual constituents is high, treatment is usually designed so as to minimize the possibility of human exposure to enteric viruses. Effective disinfection of viruses is believed to be inhibited by suspended and colloidal solids in the effluent. Therefore, these solids must be removed by advanced treatment before the disinfection step. The sequence of treatment often specified is: secondary treatment, followed by chemical coagulation, sedimentation, filtration and disinfection.

4.5 Sludge Treatment

Treating wastewater means removing the solid, soluble and colloidal pollution in the wastewater, but also transforming these substances into solid pollution (sludge). As a result, the removal of pollution from wastewater will not be complete until the sludge has been treated.

4.5.1 Sludge characteristics

The characteristics of sludge are fundamental in the treatment choice to be applied to it, and in determining the performances of the facilities to be used. The composition of sludge depends both on the nature of the initial wastewater characteristics and the purification process to which the wastewater has been subjected. Depending on the treatment process used in the wastewater treatment plant, there are three main types of sludge:

1. Primary Sludge: generated by the primary sedimentation process. This sludge contains 50 to 70% of organic matter, therefore it is highly fermentable.
2. Biological Sludge: generated by the secondary biological treatment.
3. Tertiary Sludge: generated by the tertiary treatment.

4.5.2 Sludge Treatment

Sludge treatment is intended to reduce the volume of the wastes and their fermentability by:

- Increasing the dryness of the sludge by a thickening stage.
- Stabilizing the organic matter content of the sludge (reducing the fermentability of the sludge) by a biological treatment.

The choice of the sludge treatment is determined on the basis of the following:

- the planned end-use
- its influence on the wastewater treatment
- the size of the plant for economic reasons

Sludge handling and treatment consists of the following unit processes:

4.5.2.1 Thickening

Thickening aims at separating the water from the sludge by force of gravity (Gravity Thickening) or by the use of fine bubbles (Flotation Thickening).

4.5.2.2 Stabilization

This process aims at stabilizing the volatile organic solids into a biologically stable and odorless end product. The following is a description of the most applicable sludge stabilization processes:

- **Aerobic Digestion**

Aerobic digestion is the biochemical oxidation of the volatile organic solids. This process can be used to stabilize any organic sludge including waste activated sludge, primary sludge and/ or sludge from extended aeration and trickling filter plants. The aerobic digestion process requires a large amount of energy to supply oxygen through aeration.

- **Anaerobic Digestion**

Anaerobic digestion is the biological degradation of organic matter in the absence of oxygen. Organic matter is converted to methane, carbon dioxide, and water and any remaining solids are rendered stable. Methane gas from the anaerobic digestion process is often recovered for heating or power generation. In comparison to aerobic digestion, energy requirements for anaerobic digestion are much lower.

- **Lime Stabilization**

In the lime stabilization process, lime is added to untreated sludge in sufficient quantity to raise the pH to 12 or higher thereby creating an environment which is not conducive to the survival of microorganisms.

Therefore, lime stabilized sludge will be less likely to putrify, create odors, or pose a health hazard with respect to pathogens.

4.5.2.3 Conditioning

Wastewater sludges are often "conditioned" in order to improve their dewatering characteristics. The two most common sludge conditioning processes are the addition of chemicals and thermal treatment.

- **Chemical Conditioning**

Chemical conditioning is typically used in advance of mechanical dewatering systems to reduce the sludge moisture content and increase the amount of dry solids at a relatively faster rate. Common conditioning chemicals include: ferric chloride, lime, alum, and organic polymers. Sludges which can be conditioned with chemicals include primary, secondary, and digested sludge.

- **Thermal Conditioning**

Thermal conditioning is a combination of sludge stabilization and conditioning that involve heating sludge under pressure. Heat treatment is most applicable to sludges that are difficult to stabilize or condition by other means. Due to the high capital costs, feasibility is generally limited to facilities where available land area is limited. An energy source to produce heat is also required.

4.5.2.4 Dewatering

The dewatering of wastewater sludge consists of reducing the moisture content of the sludge in order to reduce disposal costs, improve the handling characteristics, or to better prepare the sludge for final disposal by composting, incineration, or land filling. The dewatering of wastewater sludge can also help to minimize odors and render the sludge non-putrescible. Sludge dewatering can be accomplished by either natural evaporation or mechanical processes.

- **Sludge Drying Beds**

A commonly used method of naturally dewatering digested sludge is drying beds. The principal advantages of this method include low capital cost, minimal operator attention, and high solids content of the dried sludge.

- **Sludge Drying Lagoons**

Drying lagoons are most applicable in arid climates since evaporation is the prime mechanism for dewatering. Lagoons are typically not suitable for dewatering untreated sludges with high strength supernatant because of their potential for odors. Supernatant from the lagoons is typically decanted and returned to the treatment facility.

4.5.3 Ultimate Disposal of Sludge

The ultimate disposal of wastewater sludge is dependent upon a number of public health, economic and climatic factors which must all be examined prior to selection of a final alternative. A number of ultimate disposal options are:

4.5.3.1 Land Application

Land application is defined as the spreading of sludge on or just below the soil surface. Sludge disposed of in this manner must be stabilized prior to application. Characteristics of sludge that affect its suitability for land application include organic content, nutrients, pathogens, metals, and toxic organics. Depending on health standards, sludge may be applied to agricultural land or dedicated land disposal sites. Land applied sludge can act as a soil conditioner, increase water retention, as well as serving as a partial replacement for chemical fertilizers.

4.5.3.2 Incineration

Incineration is used to convert dewatered sludge cake into an inert ash through a combustion process. Incineration processes are reliable but they are complex and require specially trained operators. They are most applicable where land disposal of sludge is limited

4.5.3.3 Composting

In the sludge composting process, organic constituents in the sludge are decomposed aerobically to stable humus-like material. There are a number of environmental factors which affect the speed and course of the composting cycle but composting is not considered to be complete until pathogens have been reduced enough that the compost can be handled with the minimum health risk and can be without nuisance odors. There are several techniques for composting municipal sludges including windrows, aerated static piles, and confined processes. However, each composting technique involves some fundamental steps including the addition of bulking agents such as wood chips, regulation of moisture content and temperature, extended storage of the compost for stabilization purposes, and air drying.

4.5.3.4 Landfilling

Sludge landfilling involves the planned burial of wastewater sludge at a designated site. Solids are placed in a prepared site and covered with a layer of soil. Depending on the landfill method utilized, sludge may be landfilled either prior to or after dewatering. Landfilling of sludge is usually conducted when there are regulatory constraints to sludge reuse or when there are high levels of metals or other toxic substances in the sludge.

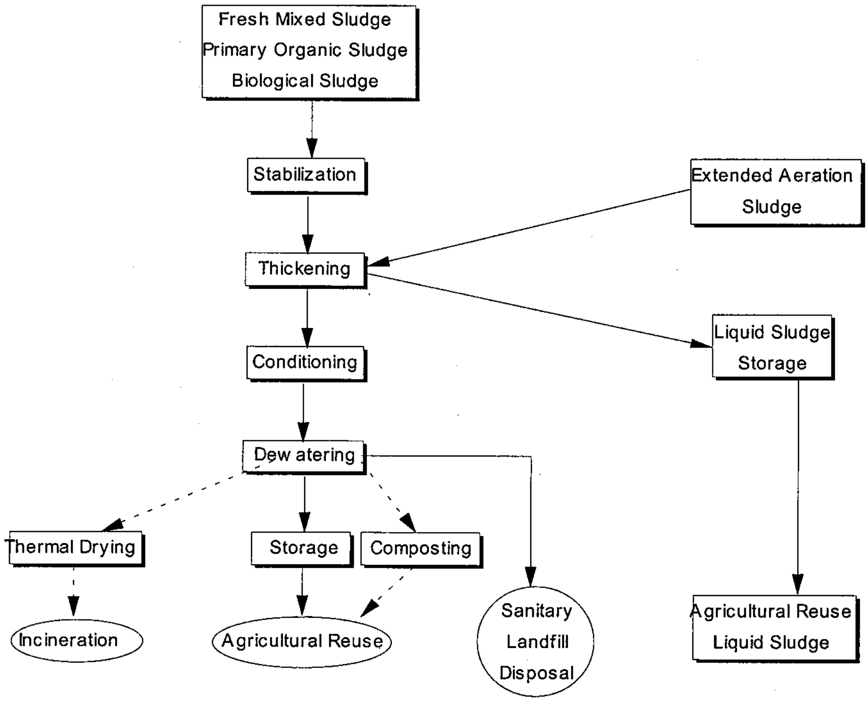


Fig. (5) General Flow Chart of Organic Sludge Treatment

WASTEWATER TREATMENT IN THE STATE OF QATAR

Over the past years, development has taken place within the State of Qatar to provide wastewater collection, treatment and reuse facilities for the majority of the population.

The first wastewater collection and treatment facility was commissioned in the capital city of Doha. In the year 1977, the total volume of wastewater generated was about 18,000 m3/d. Due the many capital projects that have been completed ever since, the total volume of the wastewater generated is about 97,000 m3/d, as shown in table (3).

Year	Wastewater Generated M3/Day	Wastewater Generated M3/Year	Population Served
1977	18,000	6,570,000	66,667
1978	19,000	6,935,000	70,370
1979	20,000	7,300,000	74,074
1980	21,000	7,665,000	77,778
1981	22,000	8,030,000	81,481
1982	23,000	8,395,000	85,185
1983	25,000	9,125,000	92,593
1984	26,000	9,490,000	96,296
1985	27,000	9,855,000	100,000
1986	34,000	12,410,000	125,926
1987	60,000	21,900,000	222,222
1988	60,000	21,900,000	222,222
1989	61,000	22,265,000	225,926
1990	68,000	24,820,000	251,852
1991	70,000	25,550,000	259,259
1992	73,000	26,645,000	270,370
1993	75,000	27,375,000	277,778
1994	74,900	27,338,500	277,407
1995	85,500	31,207,500	316,667
1996	92,300	33,689,500	341,852
1997	93,300	34,054,500	345,556
1998	93,800	34,237,000	347,407
1999	97,100	35,441,500	359,630

Table (3)

Quantities of Generated Wastewater and Population Served During the Period 1977-1999

Graph (6) illustrates the growth of the wastewater generation during the period of 1977 - 1999

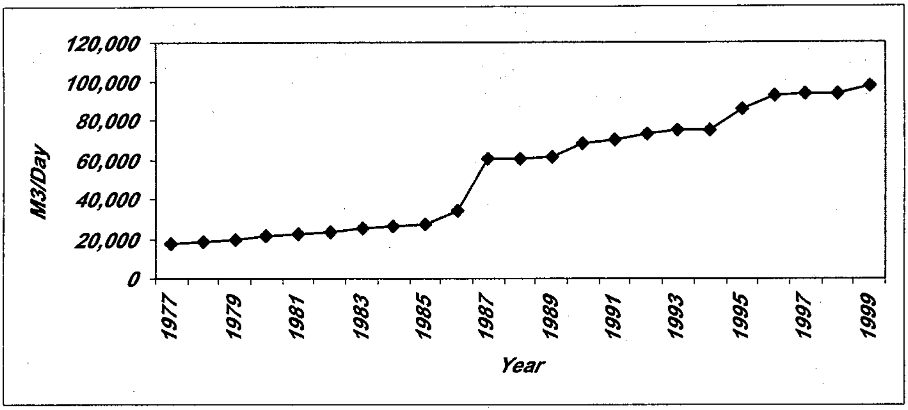


Fig. (6) Growth of Wastewater Generation During the Period 1977-1999

Currently, the Ministry of Municipal Affairs & Agriculture is managing 15 wastewater treatment facilities. These facilities are operated and maintained by the Drainage Division of the Civil Engineering Department. More details in the table (4)

Table (4) List of the Wastewater Treatment Systems in The State of Qatar

No.	Treatment Plant	Type	Design Capacity M3/D	Actual Capacity M3/D	Date of Commencement	Effluent End Use
1	Doha South	Activated Sludge Conventional	40,500	50,000	1977	Agriculture Irrigation
2	Doha West	Activated Sludge Conventional	54,000	50,000	1991	Agriculture Irrigation
3	Al-Khor	Activated Sludge Extended Aeration	2,500	1,200	1978	Irrigation
4	Al-Thakheera	Activated Sludge Extended Aeration	1,620	750	1999	Irrigation Lagoon
5	Slaughter House	Package Plant Capitox	810	200	1997	Discharged to Sewers
6	Shahaniyya	Package Plant Capitox	810	750	1985	Lagoon
7	Jumeliya	Package Plant Capitox	540	200	1991	Lagoon
8	Al-Khraib	Package Plant Biospiral	30	30	1992	Lagoon
9	Abu-Fontas	Package Plant Capitox	540	220	1997	Lagoon
10	Duhail	Package Plant Capitox	810	300	1990	Irrigation
11	Sailiyya	Package Plant Capitox	810	465	1980	Irrigation
12	Barzan	Package Plant Biospiral	160	160	1992	Irrigation
13	North Camp	Package Plant Extended Aeration	60	60	1991	Irrigation
14	A-Ghazal	Package Plant Biospiral	50	40	1994	Lagoon Irrigation
15	Joan	Package Plant Biospiral	50	40	1995	Lagoon

A. Doha South Wastewater Treatment Plant

Description

Doha south STW is situated on the southern outskirts of the capital City of Doha on a site of some 30 ha extent. It provides full treatment to the wastewater originating from about half the population of the city. The site has been used for wastewater treatment since 1960's. The original treatment process utilized trickling filters which were abandoned in the 1980's to be superseded with an Activated Sludge treatment process plant.

The treatment process at Doha South STW employs conventional activated sludge process followed by tertiary treatment. Sludge is being collected, treated anaerobically and dewatered in filter press units. The treatment process is essentially carried out in five stages, namely:

1. Preliminary Treatment in the inlet works comprising screening, screening disposal, grit removal and flow measurement
2. Primary Treatment comprising primary settling
3. Secondary Treatment comprising biological oxidation of wastewater utilizing vertical surface aeration, final settling and returning of activated sludge
4. Tertiary Treatment comprising of rapid gravity sand filters
5. Sludge collection and treatment comprising consolidation, thickening and anaerobic treatment and dewatering by filter presses

Design Criteria

Doha South STW was essentially designed to handle the following loads:

1. Population Served	= 150,000
2. Average Dry Weather Flow	= 40,500 m3/d
3. Peak Flow	= 121,500 m3/d
4. BOD5 of the Raw Wastewater	= 300 mg/l
5. BOD5 Load to be applied to Plant	= 36,450 Kg BOD5/d
6. TSS of the Raw Wastewater	= 300 mg/l
7. TSS Load to be applied to Plant	= 36,450 Kg TSS/d

The treatment process at Doha South STW is designed to produce an effluent with the following quality:

* BOD5:TSS of 20:20 following secondary treatment
* BOD5:TSS of 10:10 following tertiary treatment

Performance

Despite being overloaded hydraulically and organically by the rapid urban development and the consequential increase in population of Doha City, Doha South STW performance is considered satisfactorily producing an effluent quality of 20:20 standard or sometimes better.

The following table (5) illustrates the effluent quality of Doha South STW.

Table (5) Quality of the Final Treated Effluent of Doha South STW

	Turbidity NTU	pH	Ec us/cm	TDS mg/l	TSS(*) mg/l	COD mg/l	BOD5 (*) mg/l	Chloride mg/l	Cl2 mg/l	Ca mg/l	Mg mg/l
January	6	7.7	3055	2139	20	53	10	718	1	368	297
February	10	7.5	2976	2083	20	94	10	392	2	303	73
March	8	7.0	2895	2060	20	75	10	460	3	356	98
April	8	7.0	2895	2060	20	75	10	460	3	356	98
May	9	5.9	2602	2030	20	83	10	422	3	246	47
June	13	7.3	2822	1974	20	100	10	412	2	241	39
July	11	7.1	2993	2238	20	104	10	461	3	214	99
August	7	7.1	2900	2030	20	80	10	488	5	258	115
September	6	7.2	2767	1936	20	69	10	447	6	452	65
October	4	7.2	2928	2036	20	49	10	377	4	562	61
November	7	7.2	2905	2033	20	55	10	425	3	405	74
December	9	7.3	3009	2106	20	69	10	475	4	507	105
Average	*8*	*7.1*	*2896*	*2060*	*20*	*76*	*10*	*461*	*3.3*	*356*	*98*

B. <u>Doha West Wastewater Treatment Plant</u>

Description

The Doha West Sewage Treatment Plant (DWSTW) was commissioned in June 1992 to relief the hydraulic and organic overloading conditions at Doha South Sewage Treatment Plant.

The treatment process at DWSTP employs conventional activated sludge process followed by tertiary treatment. Sludge is being collected, treated anaerobically and dried naturally in drying beds. The treatment process (as shown in fig. 1) is essentially carried out in five stages, namely:

1. Preliminary treatment in the inlet works comprising screening, screenings disposal, sampling, grit removal and flow measurement
2. Primary treatment comprising primary settling

3. Secondary treatment comprising biological oxidation of wastewater utilizing vertical surface aeration, final settling and returning of activated sludge
4. Tertiary treatment comprising of rapid gravity sand filtration
5. Sludge collection and treatment comprising consolidation of primary sludge, thickening of wasted (surplus) activated sludge followed by anaerobic sludge digestion and air drying on sludge beds.

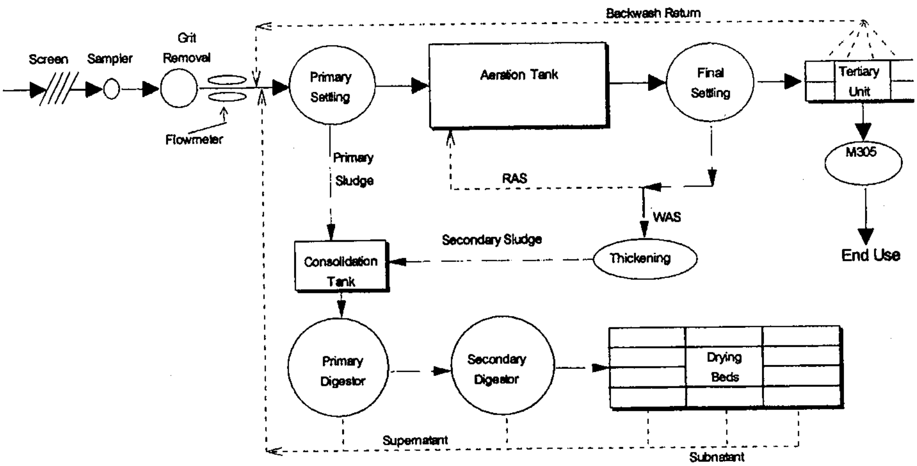


Fig. (7) Treatment Units, Processes and Operations at DWSTP

Design Criteria

Doha West STW is essentially designed to handle the following loads:

1.	Population Served	= 200,000
2.	Average Dry Weather Flow	= 54,000 m^3/d
3.	Peak Flow	= 183,600 m^3/d
4.	BOD_5 of the Raw Wastewater	= 300 mg/l
5.	BOD_5 Load to be applied to Plant	= 8,100 Kg BOD_5/d
6.	TSS of the Raw Wastewater	= 300 mg/l
7.	TSS Load to be applied to Plant	= 8,100 Kg TSS/d

The treatment process at DWSTP is designed to produce an effluent with the following quality:

* BOD_5 : TSS of 20:20 following secondary treatment
* BOD_5 : TSS of 10:10 following tertiary treatment

Performance

The treatment process in Doha West STW is quite efficient as it is capable of producing an effluent of 10:10 standard. The overall quality of Doha West STW effluent is shown in table (6).

Table (6) Quality of the Final Treated Effluent of Doha West STW

	Turbidity NTU	pH	Ec us/cm	TDS mg/l	TSS (*) mg/l	COD mg/l	BOD5 mg/l	Chloride mg/l	Cl2 mg/l	Ca mg/l	Mg mg/l
January	1.7	7.7	3781	2647	10	35	2	971	0.4	427	385
February	1.8	7.5	3990	2793	10	25	2	675	3.8	462	198
March	1.7	7.5	4138	2896	10	49	2	664	1.0	409	93
April	2.5	7.3	3690	2583	10	45	2	650	1.2	414	126
May	4.6	7.5	4110	2877	10	51	2	687	1.0	396	67
June	1.9	7.4	3891	2724	10	57	2	642	2.0	375	53
July	1.3	7.1	3661	2563	10	59	2	619	1.0	219	116
August	1.2	7.1	3421	2394	10	48	2	620	1.0	253	114
September	1.0	7.1	3357	2350	10	38	2	620	1.0	476	76
October	1.8	7.1	3495	2446	10	42	2	536	1.0	586	70
November	6.7	7.2	3337	2336	10	44	2	547	0.5	473	90
December	3.5	7.1	3411	2387	10	45	2	564	1.0	473	122
Average	*2.5*	*7.3*	*3690*	*2583*	*10*	*45*	*2*	*650*	*1.2*	*414*	*126*

Reuse of Final Treated Effluents in the State of Qatar

It is of high importance that wastewater be treated and discharged in a manner that will not be detrimental to the receiving environment and its users. The degree of pollution removal that is achieved during wastewater treatment varies according to the treatment process, time, and mechanism of reduction and removal of the different pollutants.

The provision of wastewater collection, treatment, disposal, and reuse systems will significantly improve hygienic conditions within the communities. Thus the overall impact of these systems is beneficial to the public health. Care must be taken, however, to ensure that all potential health problems are avoided to the fullest extent practical.

Full use of treated wastewater is a government policy in the State of Qatar. Qatar is presently using treated wastewater in a variety of locations for irrigating green areas, along streets and parks in Doha City, as well as irrigated agriculture. However, irrigation of vegetables (of any kind) is prohibited.

In order to safeguard the public health and to keep the environment clean, wastewater regulations have been proposed which require tertiary level of treatment of the wastewater. Moreover, regulations concerning reuse are strictly enforced.

Basically, one hundred of the treated effluents produced by the two major wastewater treatment plants (Doha South & Doha West STW's) is used for agriculture and irrigation purposes. There are (4) effluent outlets:

1. Al-Rakhiyya Government Farm

Al-Rakhiyya Government Farm is located about 60 km south east of Doha City. Treated effluent from both Doha South and Doha west is conveyed via two pumping stations and two pipes extending from the two plants down to the farm. In this farm, animal feed such as, alfa alfa is irrigated by treated effluent.

In the following table, the monthly quantities of treated effluent pumped from both Doha South and Doha West to al Rakhiyya Farm.

Table (7) Amounts of Final Treated Effluents Pumped to Al-Rakhiyya Farm

	Doha South STW	Doha West STW	Total
	M3/Month	M3/Month	M3/Month
January	756,571	1,529,644	2,286,215
February	678,342	1,061,078	1,739,420
March	815,300	1,295,356	2,110,656
April	702,105	1,199,653	1,901,758
May	761,360	1,295,356	2,056,716
June	701,190	1,257,356	1,958,546
July	838,147	1,133,089	1,971,236
August	600,548	1,144,327	1,744,875
September	615,300	1,126,432	1,741,732
October	754,261	1,144,327	1,898,588
November	606,855	1,192,451	1,799,306
December	722,068	1,269,086	1,991,154

2. Doha Green Areas and Parks

Final treated effluent is pumped to the 1160m3 capacity for reuse in irrigating green areas, along streets and parks in Doha City. For this purpose, multiple systems of storage tanks, conveyance pipelines and distribution network is used.

In the following table, the monthly quantities of treated effluent pumped from Doha South and to Doha City.

Table (8) Amounts of Final treated Effluents Pumped to Doha City

	Doha South STW M3/Month
January	605,256
February	600,817
March	652,240
April	580,407
May	609,088
June	579,650
July	670,518
August	480,438
September	508,648
October	603,409
November	501,667
December	577,654
Average	*580,816*

3. New Farm

Adjacent to Doha West STW, a new private farm has been commissioned. Treated effluent from Doha west is conveyed via a pumping station and single pipe extending from the plant to the farm. In this farm, animal feed such as, alfa alfa is irrigated by treated effluent.

Following a table which illustrates the quantities of treated effluent pumped to the private farm.

Table (9) Amounts of Final treated Effluents Pumped to Private Farm

	Doha West STW M3/Month
January	241,443
February	353,292
March	241,443
April	269,751
May	241,443
June	233,388
July	505,685
August	401,687
September	312,445
October	343,237
November	229,293
December	551,163
Average	*327,023*

4. Tanker Filling Gantries

Twin submersible pumps in the final effluent pumping station in Doha South STW pump final effluent on demand to four double outlet tanker filling gantries. Here road tankers receive effluent for use in those approved irrigation areas which are not supplied by the effluent distribution network.

Following a table which illustrates the quantities of the tankered effluent

Table (9) Amounts of Final treated Effluents Pumped to Tankers

	Doha South STW M3/Month
January	43,603
February	43,603
March	43,603
April	43,603
May	43,603
June	43,603
July	43,603
August	43,603
September	43,603
October	43,603
November	43,603
December	43,603
Average	*43,603*

Application of Geographic Information System for Wastewater Treatment and Reuse in the State of Qatar

In the State of Qatar, it is the responsibility of Drainage Division of the Ministry of Municipal Affairs and Agriculture, to collect, treat and dispose the domestic wastewater generated by the different cities and villages in the State. Normally, wastewater flows into manholes and sewers via house connections. Wastewater is transported to the pump stations and then pumped to the treatment plants where it is treated. The treated effluent is stored in water towers which distribute the effluent to irrigate gardens and government farms. All this is done by designing, constructing, operating and maintaining a sophisticated system of pipes, pumps, treatment works and water towers for the three networks, namely, wastewater, surface/ ground water and treated effluent.

Geographical Information System (GIS) is being used intensively to achieve this sophistication, because it is an organized and efficient source of drainage information, as it is responsible for creating and maintaining a detailed wastewater collection, conveyance, treatment and reuse database that provides useful information on the collection network, conveyance, disposal and reuse systems.

Using GIS, both positional data (geographic position of a particular drainage feature) as well as attribute data (size, height, material, length of a sewer, ...etc.) are stored accurately. For the wastewater treatment and reuse, the treated sewage effluent (TSE) network data is the key factor, although the GIS database at Drainage Division is having detailed data on each and every drainage feature. TSE network comprises of TSE mains, valve chambers, valves, pumping stations and water towers. All these drainage features have been surveyed for its positional and attribute data. Drainage Division is having a separate survey unit, wherein the surveyors go into the field and collect data on day-to-day basis. Surveys have been conducted for 19,000 manholes, 40,000 house connections, 1,400 valve chambers, 132 pump stations and 9 water towers. In addition, information has been acquired on the diameters of, and materials used for the 1,000 km of pipes in all the three networks. Coordinates for all these features are available to within 20 cm accuracy.

A special data entry form has been developed for each drainage feature and the surveyors use these forms to fill in the collected data. The data are then fed into the computer in a properly designed database. The GIS database is then subject to severe quality control, such as checking for errors both by manually and by means of computer-automated programs. Positional accuracy is being cross checked with already existing drainage feature. Discrepancies in the attribute data, if found, are notified to the survey team for rechecking. Once rechecked, the data are being entered again, which are then subjected to quality control checks. Thus, quality control on GIS database is done extensively and to make sure that the data delivered to the public are accurate.

Various user-friendly computer applications have been developed to use this database efficiently to solve problems by producing maps and by performing analysis. GIS applications assist in two ways. Firstly, it quickly provides the user with information that would normally take a few days to acquire if he were to rely on other sources. Using GIS, the data that he requires is on-line and available to him in seconds. Secondly, the application presents the user with options on how to perform various analyses that can be produced on a hard copy map as an output. As for as wastewater treatment is concerned, the engineer is capable of accessing all the data related to TSE network in a matter of no time. The detailed data pertaining to all the drainage features in a particular area assists the engineer in decisions on the amount of TSE water to be pumped from a pumping station or to be supplied to an area, ...etc.

GIS in Qatar is having the unique concept of societal GIS, wherein all the government agencies share their data for the public cause. A high band width optical fibre cable network is connecting all the government departments in Qatar, with the nodal control facility at the Center for GIS. With the cooperation from other government departments, GIS is making it easier to overlay roads, planning, water and electricity network on top of drainage network to have a realistic approach for treated water disposal purposes. Thus, Qatar is adopting the state-of-the-art technology in the wastewater treatment and reuse system.

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Water Recovery from Process Effluents Using Efficient Evaporation Technology

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WATER RECOVERY FROM PROCESS EFFLUENTS USING EFFICIENT EVAPORATION TECHNOLOGY

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ABSTRACT

Pure fresh water is an exhaustible natural resource. In industrial processes fresh water consumption and effluent flows can and have been reduced by internal re-use. As a result of these measures the solids content in the process waters will increase. For further water loop closure efficient "kidneys" are required for dissolved solids removal. Using evaporation, high quality, recyclable water can be produced and, on the other hand, the flow containing the separated substances can be minimized. Conventional evaporation has been associated with high capital and energy costs. Features and applications of a new, efficient evaporation technology, specifically developed for purification/ recycling/concentration of industrial effluents and landfill leachates as well as for production of high quality process water, are described.

INTRODUCTION

In industrialized countries the time of using huge amounts of process water and discharging it diluted to the environment is over. Based on both pure economical reasons and more stringent discharge limits, lot of efforts have been put on water conversation and more efficient utilization of process water.

In industrial processes fresh water consumption can and has been reduced and effluent flows minimized by internal re-use. As a result of these measures the solids content in the process waters are increasing. At some point process disturbances, product quality and corrosion problems will set a limit to additional water conservation. For further water loop closure efficient "kidneys" are required for dissolved solids removal.

Coarse particles can be separated by simple mechanical filters and suspended solids down to micron particle range using flotation and microfiltration. Using ultra- and nanofiltration macromolecular size substances can be partly separated, but for efficient macromolecular and ionic range separation reverse osmosis or evaporation must be used. Efficient separation of salts and other dissolved solids is essential for furher or total water loop closure.

The technological benefits of evaporation are well known. It is one of the most efficient methods for separation of dissolved solids and water. Using evaporation, high quality, recyclable water can be produced and, on the other hand the flow containing the separated substances can be minimized. Concentration of aqueous solutions by evaporation is an old and widely used method in a number of industries and applications, including treatment of industrial effluents and landfill leachates.

Evaporation has long been associated with high energy use; however, improvements in evaporation systems have been made to increase their energy efficiencies. The most significant progress in energy savings has been achieved through increasing the number of effects in Multi Effect evaporators and reusing the vapor in

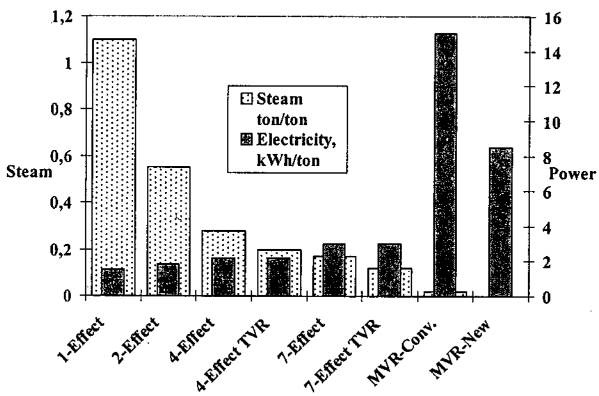


Fig. 1. Energy consumption in evaporation

different ways. In Thermal Vapor Recompression (TVR) evaporators, the inertia of steam is used for recycling part of the vapors through ejectors. In Mechanical Vapor Recompression (MVR) evaporators, all vapor is recycled back as heating steam using vapor compression with high pressure fans or compressors. In many applications, MVR evaporators are superior because they do not need steam or cooling water. In case waste heat or low cost thermal energy is available Multi-Effect (ME) evaporators can be the feasible option. The energy use of evaporators with different configurations is summarized in Figure 1.

EVAPORATION IN TREATMENT OF EFFLUENT

The global need to minimize environmental impacts caused by effluent discharges and to save fresh water resources is resulting to stricter discharge limits and higher water costs. In many applications, simple, conventional treatment methods will not be sufficient. Evaporation is one of the most efficient methods for separation of dissolved solids and water. Using evaporation, high quality, recyclable water can be produced.

From a technological point of view, evaporation is an ideal method for the following reasons:

- Water recovered from the effluent stream is of high quality and can, in most cases, as such be reused in the process or discharged into the nature
- All non-volatile substances can be completely separated from the "distillate" stream
- Harmful solids can be recovered for appropriate disposal or valuable solids can be recovered and reused

Evaporation covers the whole particle size spectrum as shown in Figure 2 and removes most efficiently dissolved substances such as salts and organic material (COD) as shown in the example of Figure 3.

Treatment of effluent with conventional evaporation processes does have some drawbacks. Among them are:

- High operating costs
- High capital costs especially in corrosive environment, where noble construction materials are required
- Problems with scaling and fouling, especially with varying influent quality and composition
- Steam and cooling water availability for thermal evaporative systems

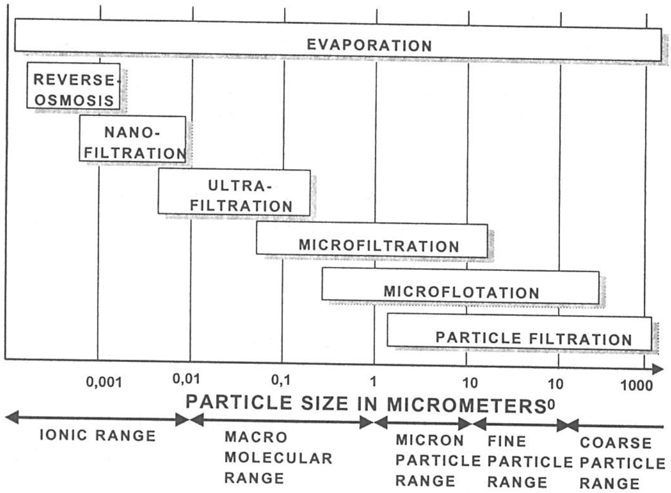


Fig. 2. Filtration spectrum of separation methods

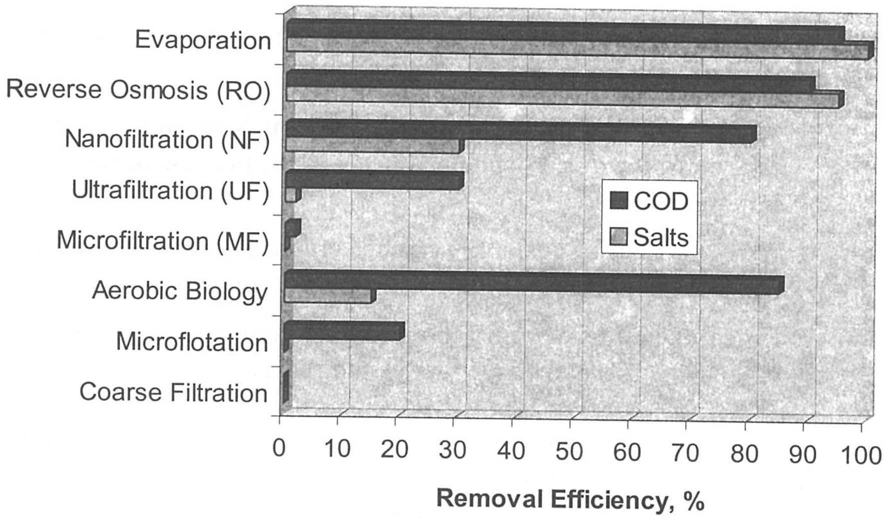


Fig. 3. Salt and COD removal efficiency in a pulp & paper industry effluent stream

THE ADVANCED EVAPORATION TECHNOLOGY

The target in the development work of the new evaporation technology has been to solve or minimize the above drawbacks of conventional systems and make the evaporative process competitive for industrial effluent and landfill leachate applications. The new evaporation technology applies the

Mechanical Vapor Recompression (MVR) or Multi-Effect (ME) principle and combined with Falling Film (FF) evaporation method.

The operating principle of the evaporator applying the MVR principle is shown in Figure 4 and the ME principle in Figure 5.

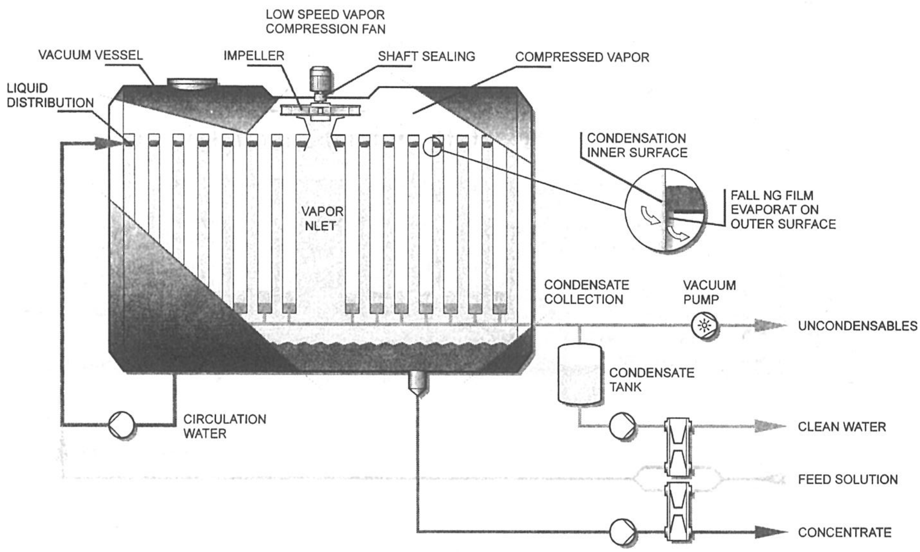


Fig. 4. MVR operating principle

The main components of the MVR system are: vacuum vessel, evaporative heat transfer surface installed inside the vessel and vapor recompression fan. The polymeric heat transfer surface element has a square, double-sided structure. The solution to be concentrated is circulated to the top of the element and distributed on both of the outer surfaces of the element. As the solution reaches its saturation state, it begins to evaporate when heated. The vapor generated flows into a fan, which adds energy by increasing the pressure and temperature of the vapor. After the fan, the compressed vapor is introduced inside the heat transfer element. Here the vapor condenses; latent heat is released and transferred through the polymeric surface causing the solution on the exterior surface to release more water vapor. The condensed vapor, the product of the process, is then discharged from inside the element as clean condensate. The concentrated liquid is discharged from the bottom of the vessel for disposal.

In the Multi Effect evaporators (Figure 5.) effluent pumped to the first effect is heated with thermal energy. The energy source is a hot liquid solution (temperature >70°), hot gas, vapor or low pressure steam. The vapor evaporated from the effluent in the first effect is used as heating steam in the second effect and vapor from second effect again in the third effect and so on. After the final effect the vapor is condensed in condenser. The total number of effects will chosen based on the required capacity as well as based on temperature levels of the energy source and the cooling system.

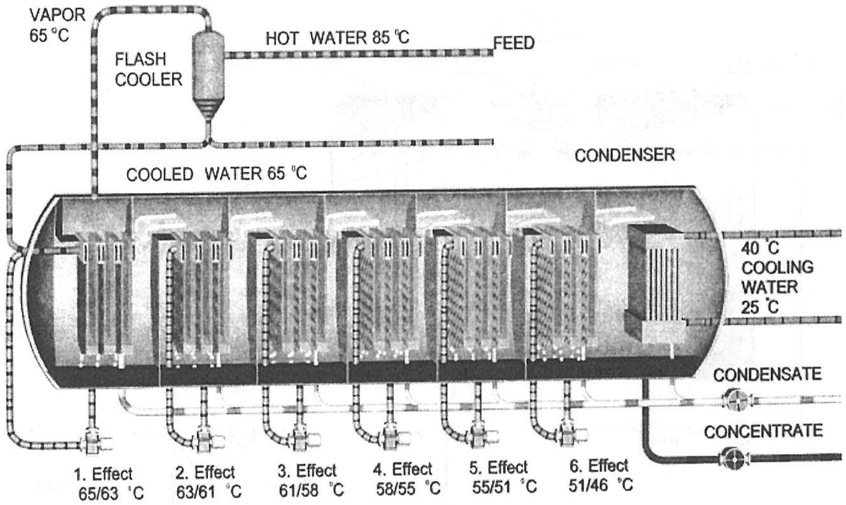


Figure 5. Multi Effect operating principle utilizing hot water as energy source

THE INNOVATION

The key innovation of the technology is the new heat exchanger concept, where the surface is manufactured of high-tech polymeric film. The innovation is related to the construction, special welding and assembly methods of the film itself and involves specially designed liquid distributors and condensate collectors that are also fabricated of polymeric materials.

Low cost polymeric materials can be used as basic component in the film because of the low operating temperature (55-65°C). On the other hand, low temperature also leads to a low pressure differential over the heat transfer surface. The pressure difference over the heat transfer surface is equal to approximately 200 mm water gauge (at 60°C), which enables the use of thin films in the evaporator surfaces. The use of thin walls is essential because thermal conductivities of polymeric materials are low, typically in order of 0.15-0.5 W/(m×K). These are very low figures compared to metallic materials; for example thermal conductivity of stainless steel is typically 16 W/(m×K). Wall thickness of metallic surfaces is usually between 0,5 to 1,5 mm. The polymeric heat transfer surfaces have wall thickness between 0,02 - 0,04 mm. Using such thickness, the relatively low heat conductivity of the surface material is overcome and heat transfer coefficients rival that of metallic surfaces.

Suitable polymeric materials can be found for practically every aggressive process environment and application. In addition, the cost of the heat exchanger is reduced to a fraction of the metallic counterpart. Because of the low cost, it is economically feasible to have a much larger surface than before.

THE EVAPORATOR UNITS

Fifty individual heat transfer surface elements are combined to form a single, modular heat transfer surface cartridge (Figure 6).

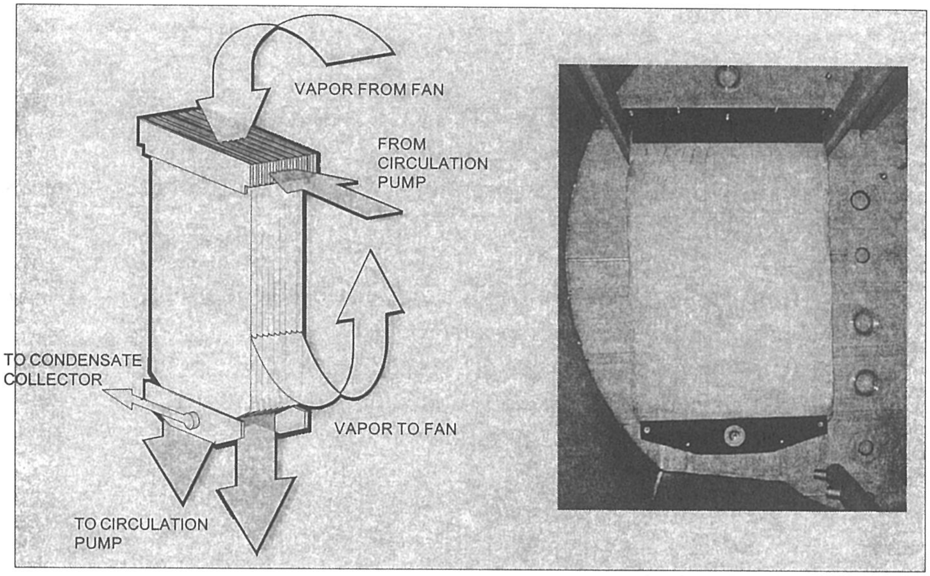


Figure 6. Hadwaco polymeric heat transfer cartridge

The number of identical cartridges of each system will depend on the required capacity. The cartridges are installed in a horizontal, cylindrical vessel. In the MVR evaporators a simple, low speed fan integrated with the vessel is used for vapor recompression. During a cold start-up, the system is heated to the operating temperature using an energy outer source (steam, hot water, electricity); however, once the process starts, only the electrical energy needed by the recompression fan and the pumps is required.

The feed effluent entering the evaporator is preheated by heat recovered from both the condensate and concentrate streams being discharged from the system, as well as the vapor stream to the fan. Typically, the discharge temperatures of these streams are 2°C higher than the inlet temperature of the effluent stream.

Modular concept. Both the heat transfer cartridges and the evaporator system are modular in concept. Two basic designs of the heat transfer cartridges are used, depending on the vessel diameter, and they are identical within each individual system. This feature allows the user to maximize their coverage while keeping spare parts inventory to a minimum. Additionally, the heat transfer cartridges, unlike their metallic counterparts in conventional evaporators, are easy to handle and replace.

This modular concept also allows stepwise enlargement of the installation, which can grow as the needs of the user grow. The units are delivered fully assembled with only external connections to the user's outer systems being completed on site. The units are built for outdoor operation thus requiring no additional building.

BENEFITS OF THE ADVANCED TECHNOLOGY

In the Mechanical Vapor Recompression (MVR) evaporators the benefits achieved include:

- Low cost evaporative surface (see enclosed equations)
  - large heat transfer surface
  - small temperature difference
  - simple, low speed fan as vapor compressor
  - low power consumption
- Polymeric surface
  - corrosion resistant
- Flexible surface
  - easy cleaning

$$\mathbf{\Phi = K} \text{ x } \mathbf{A} \text{ x } \mathbf{\Delta T}$$
$$\mathbf{P_C = C} \text{ x } \mathbf{\Phi} \text{ x } \mathbf{\Delta T}$$

Where:
F = Evaporating capacity
K = Heat transfer coefficient
A = Evaporative surface area
P_C = Fan power consumption
C = Fan coefficient

(Basic equations in MVR evaporator design)

Benefits in multi-effect (ME) evaporation:

- Very large surface available at reasonable cost
- Low temperature difference over each effect
    - more effects and larger capacity with a given energy amount
    - efficient utilization of waste heat in form of hot liquid, vapor or gas

EFFLUENT TREATMENT COSTS (Example)

The operating costs of the advanced technology evaporators applying both the MVR and the ME concepts (waste heat as energy source in the ME) are shown in Figure 7. The figures are based on electrical energy cost of 0,05 EUR/kWh. These figures are calculated for a typical paper mill case, but are applicable in different applications – usually only the chemical cost (for cleaning, precipitation or foam control) varies from case to case. The maintenance/spare part cost includes the cost for replacing the cartridges every two years.

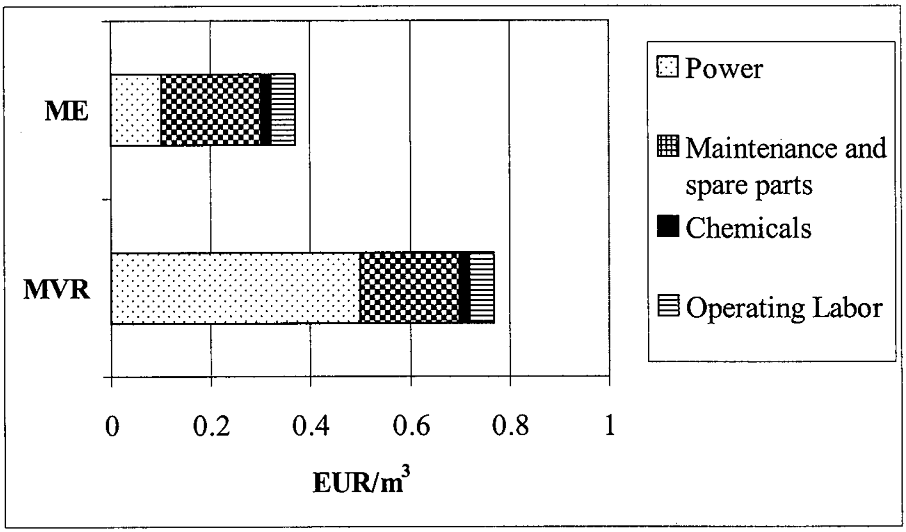


Figure 7. Advanced technology evaporators, operating cost

The following example describes a situation, where biological treatment has been used for effluent treatment in paper mill. In case the capacity or performance of the biological treatment reaches its limit because of any of the following reasons:

- The production capacity is increased
- Product quality is improved by applying more bleaching or with new additives
- The mill is facing new, stricter discharge limits

The solution is:

- Concentrate the impurities to a small effluent flow through internal re-use and process optimization
- Evaporate this stream
- Return clean condensate to the most demanding users in the process
- Direct foul condensate from the evaporator and other easily degradable effluents to existing biological treatment

Typically, the COD content of the wastewater entering the biological treatment is around 2.000 mg/l and operating cost 0,5 EUR/m^3 corresponding to 0,25 EUR/kg of removed COD.

After process optimization and efficient water re-use the stream discharged from the process has typically a COD content of 10.000 mg/l. With above evaporator operating the COD removal cost is EUR 0,08/kg of COD in the MVR and EUR 0,04/kg in the ME evaporator as shown in Figure 8.

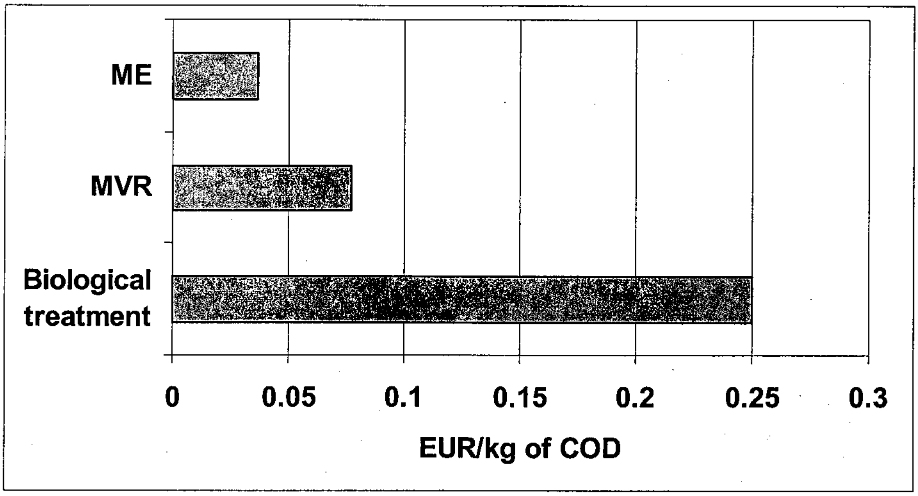


Figure 8. Operating costs in COD removal

APPLICATIONS

Treatment concepts and operational data of some applications are described in this chapter.

Paper mill process water/effluent treatment

Saudi Arabia.
The technology has been used in the paper mill of Arab Paper Manufacturing Co. in Saudi Arabia. The mill produces 200 t/d liner and fluting from recycled paper. The plant set-up is shown Figure 9.

The facility contains two evaporator units, both of which consist of two modules. The plant has been designed so that any of the four units can be taken off line for cleaning or maintenance while the other three units remain in operation

- Location: Dammam, Saudi Arabia
- Production: 200 t/d Liner/Fluting Cartonboard from recycled paper
- Evaporation Plant in operation since January 1996
- 2 x Hadwaco 600/54L-2.3
- Capacity 1.200 m^3/d
- concentration factor 10
- Streams processed
    - 50-80% brackish ground water concept
    - 20-50% mill effluent
- Chemical dosing: 50 ppm of anti scaling agent

The feed water quality data and purification results are shown in Table 1 and plant view in Figure 10.

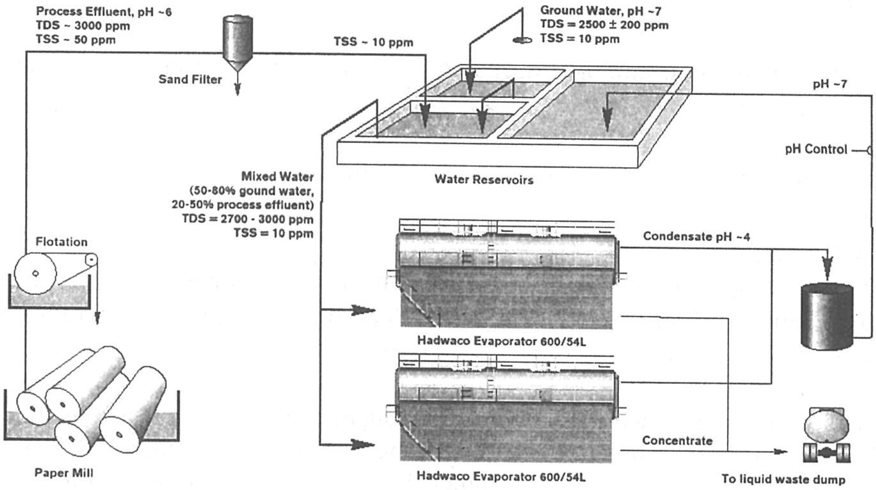
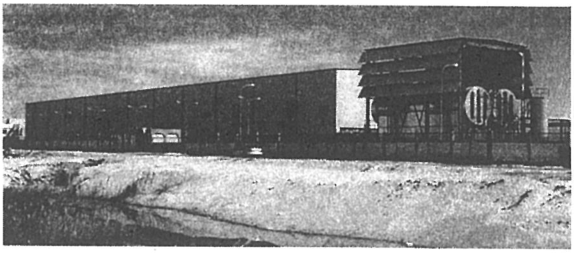


Figure 9. Set-up of Saudi Arabian paper mill water treatment plant

		Ground Water	Mill Effluent	Feed	Concen-trate	Conden-sate	Reduction %
Volume	m^3/d	1053	280	1330	133	1200	90
pH		7,6	5,2	5,3	5,5	4,4	
Conductivity	µS/cm	3150	4050	3250	31800	95	97,4
TDS	mg/l	2510	4116	2548	24060	-	
TSS	mg/l	<1	314	92	468	-	
COD_{Cr}	mg/l	11	14400	3196	16330	179	95
BOD_5	mg/l	10	2300	750	2550	56	93,3
Na	mg/l	530	900	555	5875	10	98,4
K	mg/l	58	85	63	515	-	
Mg	mg/l	70	60	70	696	-	
Ca	mg/l	205	713	335	3102	-	
Fe	mg/l	<0,05	0,6	0,36	2	-	
Cl	mg/l	787	213	624	6736	-	
SiO_2	mg/l	26	76	45	280	-	
SO_4	mg/l	432	446	394	3620	-	
CO_3	mg/l	0	0	0	0	0	

Table 1 Feed water quality and purification results of the APM Water Treatment Plant

During the first months of operation scaling due to precipitation of calcium sulfate and carbonate (coming in to the system with the ground water) caused problems in the operation (i.e. decreased capacity due to clogging of liquid distributors). This problem has now been overcome by mixing the ground water and the lower pH mill effluent together and by optimizing the use of anti-scaling agents. In this plant a cartridge lifetime of two years has been reached.



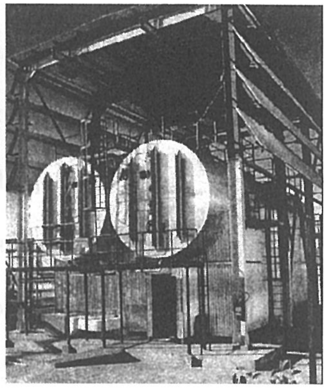


Figure 10. APM/ Waraq, Dammam, Saudi Arabia, water treatment plant

Metal surface treatment - rinsing water purification

<u>Aluminum profile surface treatment</u>

A closed water loop metal surface treatment facility has been in operation since beginning of 1998 in Finland. The process concept of this plant is shown in Figure 11.

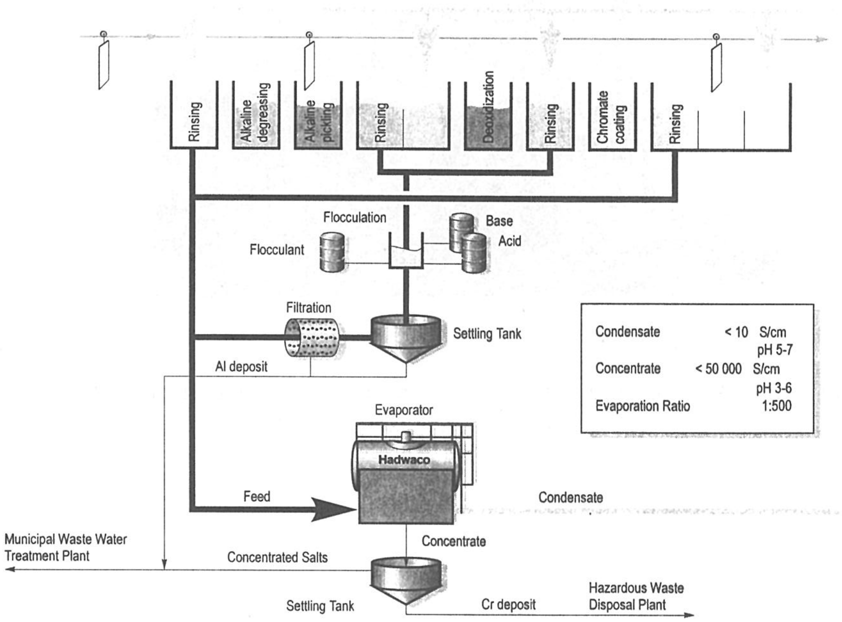


Figure 11. Aluminum profile surface treatment - closed water loop concept

Other applications

Some of the applications, where full scale Units are in operation include:

- Landfill leachate treatment (Finland, Spain, Italy)
- Desalination of sea water (Malta)

- Purification of animal feed industry effluent (Finland)
- Heavy duty (wiper) laundry effluent concentration (The Netherlands)
- Concentration of cooling tower blow-down (South-Africa)
- Purification of metal surface treatment effluent (Finland)
- Purification of road tanker washing effluent (Sweden)
- Reclamation of contaminated ground water (USA), Figure 12.
- Copper rod pickling rinse concentration/recycle (CAN)

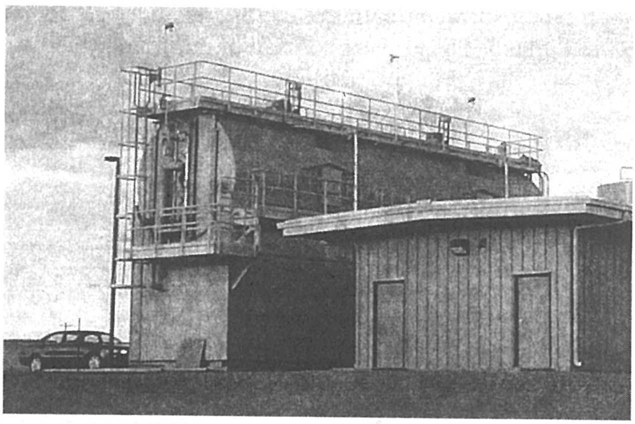


Figure 12. MVR Evaporation Unit (540 m^3/d) fo reclamation of contaminated ground water, Arizona, USA

Phytoremediation of Heavy Metal Contaminated Wastewater for Agricultural Purposes

A.A. Soaud, S.A. Aboulroos, M.A. Ali and Assia A. El Falaky

PHYTOREMEDIATION OF HEAVY METAL CONTAMINATED WASTEWATER FOR AGRICULTURAL PURPOSES

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ABSTRACT

There is a lot of concern about the clean up of toxic pollutants from the environment. Most of the remediation technologies that are currently in use are very expensive and relatively inefficient. Moreover they, sometimes, generate a lot of waste, that has to be disposed off. Phytoremediation is a novel, environmentally friendly and cost-effective technology to clean up heavy metals and other toxic compounds from contaminated environments. The present study was conducted in greenhouse to investigate the uptake patterns of the metals Cd, Co, Ni and Pb at different concentration by two common aquatic plants (*Eichhornia crassipes and Ceratophyllum demersum).* The possibility to accelerate the removal of high concentrations of heavy metals by the two plants and to test their effectiveness on the removal of heavy metals from primary and secondary treated wastewater. Results showed that both plants were able to remove the four heavy metals from the solution over the entire range of the used concentrations (0.1 to 5.0 PPM). The rate of removal decreased in the order: Co > Pb > Ni > Cd, in case of *Eichhornia crassipes* and in the order: Ni > Co > Pb > Cd, in case of *Ceratophyllum demersum.* The plant uptake of these heavy metals followed a biphase rate of uptake that made the plants unable to completely purify the solution from heavy metals when the concentration was higher than 0.5 PPM. Replacing the standing plants by new ones every two days accelerated metal removal even after exposure to very high concentration of heavy metals (20 PPM). A practical test for wastewater purification proved that either *Eichhornia crassipes* or *Ceratophyllum demersum* was capable of totally depleting various heavy metals after 24 and 36 hours from secondary and primary treated water, respectively. These results indicated that the potential for the use of aquatic plants for heavy metals removal in wastewater treatment plants is high.

Key words : Phytoremidiation, heavy metals, wastewater, aquatic plants

INTRODUCTION

There is a growing concern on the reuse of wastewater as an additional source of water for different applications in agriculture, aquaculture or forestry to meet the growing demand for water. Egypt produces a huge amount of wastewater per year. During 1989 and 1990, the estimated amount of treated wastewater (primary treatment) was $2.55x10^9$ m^3. In the year 2000, this amount will increase to $3.0x10^9$ m^3 (Thanh and Visvanathan 1991). Wastewater in Egypt contains some elements such as heavy metals, which are harmful to human and animal health and to plant growth (Abdel-Gaffer et al 1988; Abdel-Gawad 1992; Thanh and Visvanathan 1991).

Traditionally, metals have been removed from wastewater using methods such as chemical precipitation, ion exchange, filtration, or membrane technology. Yet, for a remediation project, the traditional methods are prohibitively complex, expensive and inefficient. Moreover, the resulting concentrated residues may be a hazardous waste problem.

Phytoremediation is the use of plants, including trees, grasses and aquatic plants, to remove, destroy or sequester hazardous substances from the environment. It is an emerging technology for environmental remediation that offers promise as a low-cost, versatile technique suitable for use against a number of different types of contaminants in a variety of media (Glass, 1999). A variety of naturally-occurring and specially-selected plant species are used in phytoremediation. A number of terrestrial and aquatic plants are known to be natural hyperaccumulators of metals. In the past decade, the use of the aquatic plants cultured in shallow ponds and artificial wetlands for purification of metal-containing wastewater has proven economically feasible for small communities (Crites and Mingee 1987, Duffer 1982, Reed et al. 1988). The research into this field may contribute new, relatively simple and cheap alternative of heavy metal removal. However, metal accumulation in aquatic plants varies with plant species (Dietz 1973, Abo-Rady 1980, Low et al. 1984, Chen et al. 1990, Ornes et al. 1991, Sawidis et al. 1991), the kind of metal and its concentration in growth media (Lee et al. 1981, Taylor and Crowder 1983, Mortimer 1985, Geeta et al. 1991) and exposure period of the aquatic plants to these metals (Wolverton and McDonald 1978, O'keefe and Hardy 1984, Jamile et al. 1985, Delgado et al. 1993).

The objectives of this study were (1) to investigate the uptake prototype of selected four heavy metals (Cd, Co, Ni and Pb) at different concentration by two common aquatic plants in Egypt (*Eichhornia crassipes and Ceratophyllum demersum),* (2) to accelerate the removal of high concentrations of heavy metals by studied plants, and (3) to check the effectiveness of the aquatic plants on the removal of heavy metals from primary or secondary treated wastewater.

MATERIALS AND METHODS

Greenhouse experiments were conducted to study the ability of aquatic plants as bioaccumulators for heavy metals and their possibility for use in water purification purposes. *Eichhornia crassipes* (floating plant) *and Ceratophyllum demersum* (submerged plant) were chosen for this investigation, since previous study indicated that they were good accumulators of heavy metals (Ali 1996). The Cd, Co, Ni and Pb were the tested metals as the Environmental Protection Agency (EPA, 1973) has labeled them as priority pollutants. The two aquatic plants were collected from the River Nile, near Geziret ElDahab in Giza and were cultured in 10% modified Hoagland and nutrient solution until used in the experiments (Reddy and DeBusk 1985). All plants were in good condition (healthy) and in similar state of growth, root and leaf color to ensure uniform absorption areas. Two hundred grams of wet biomass of *Eichhornia crassipes* (two plants) or *Ceratophyllum demersum* were rinsed and held in six liters opaque low-density polyethylene buckets (20 cm diameter by 25 cm deep). Each bucket contained five liters of 10% Hoagland solution (initial pH = 6) with mixture of the four heavy metals. The levels of each metal in the mixture were 0.1, 0.5, 2.0 and 5.0 PPM. The Cd, Co, Pb were supplied as $Cd(NO_3)_2$, $Co(NO_3)_2$ and $Pb(NO_3)_2$, while Ni was supplied as $NiSO_4$. The plants were grown in the solution for a period of 6 days. One bucket for each level of heavy metal was kept unplanted during the exposure period as control treatment. Since the transpiration rate was high (temperature range was 30–35 °C), the solution volume was constantly monitored during the exposure period and additional 10 % Hoagland solution was added to maintain the volume at 5 liters. Each treatment was replicated three times. The Cd, Co, Ni and Pb concentrations in solution of unplanted (control) and planted treatments were measured daily to determine their rate of removal.

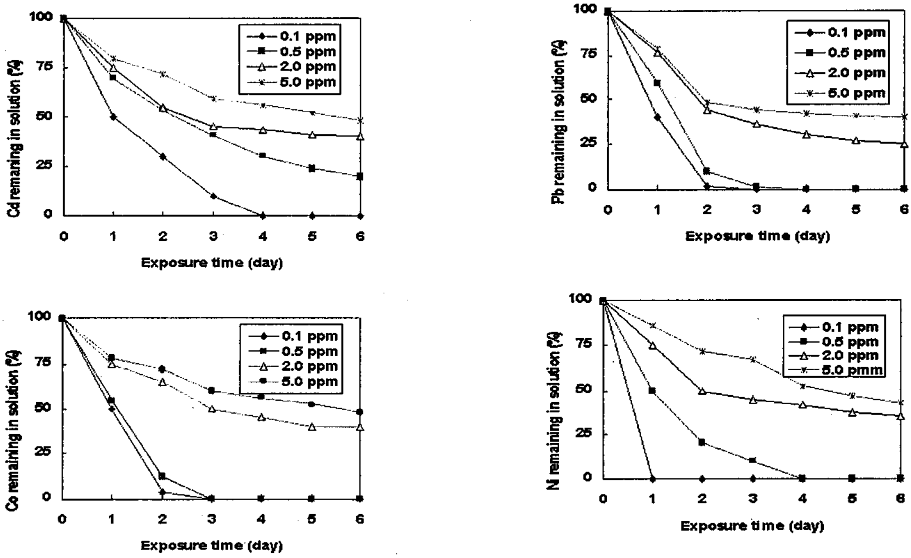
Results of the previous experiment showed that the removal of metals from water by plants was very high at the beginning, then decreased sharply after two days. The present experiment was thus designed to study the possibility of accelerating the removal of heavy metals through introducing new plant after certain time intervals. Each of the Cd, Co, Ni or Pb was superimposed over the 10 % Hoagland solution at a concentration level of 20 PPM. The plants of *Eichhornia crassipes* or *Ceratophyllum demersum* were grown for two days, then removed and replaced by new ones. This technique was repeated several times until the metal concentration in solution comes down to zero.

To carry out a practical test for wastewater purification, wastewater samples were collected from Zenin Water Treatment Plant. The samples were collected after primary or secondary treatments. Constant weight (300 g), from either *Eichhornia crassipes* or *Ceratophyllum demersum* plants was grown in 17 liter opaque polyethylene buckets containing either the primary or secondary treated sewage effluent and each treatment was repeated two times. The Fe, Mn, Cu, Cd, Co, Ni and Pb concentrations in the water of the treatments were determined every 12 hours until the heavy metals were totally depleted.

RESULTS AND DISCUSSION

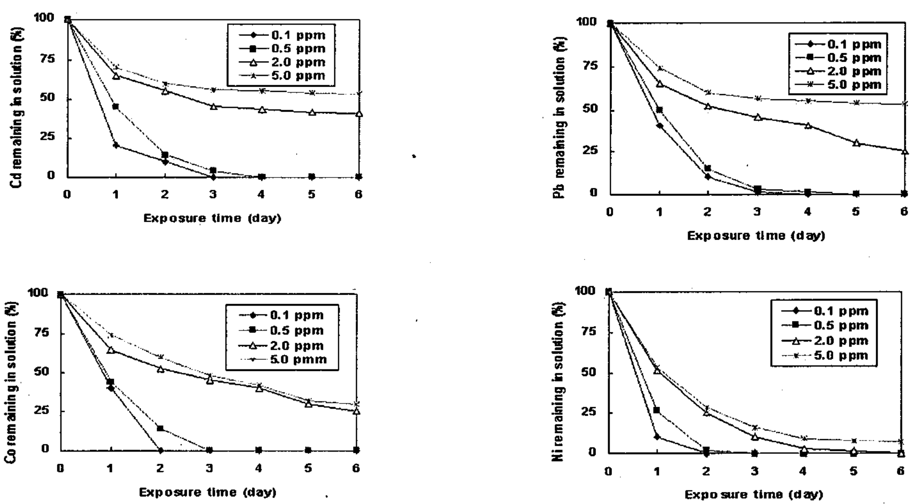
Patterns of heavy metals removal by aquatic plants

The rate of removal of heavy metals from their solutions by *Eichhornia crassipes* and *Ceratophyllum demersum* was studied by determining the concentration of metal remaining in solutions over the time of exposure. Figures 1 and 2 show metals remaining in the treatment solutions as percentage of each metal concentration in control treatment. The data indicted that both plants were capable of removing the four heavy metals over the entire range of the used concentration, but with different rates. Generally, the rate of removal decreased in the order: Ni > Co > Pb > Cd, in case of *Ceratophyllum demersum* and Co > Pb > Ni > Cd in case of *Eichhornia crassipes*. For each plant, however, the rate of removal of each metal varied widely depending on its concentration, i.e., the lower the concentration of the metal the higher was the rate of its removal. For example, *Ceratophyllum demersum* completely removed the four metals from 0.1 and 0.5 PPM solutions within the first 2 to 3 days of exposure. This was also found in case of the removal of Co and Pb, and to a less extent Ni, by *Eichhornia crassipes*. However, removal of Cd from the 0.5 ppm solution by *Eichhornia crassipes* proceeded in a very slow rate to the extent that the plant was not able to completely remove Cd from solution even after 6 days of exposure. In this respect Delgado et al. 1993 found that *Eichhornia crassipes* needed 24-day period in order to completely remove Cd from concentration levels higher than 0.1 PPM. The results also indicated that the absorption of the four heavy metals from the high concentration levels (2.0 and 5.0 PPM) proceeded in a very slow rate. Even after 6 days of exposure, there were 25 to 50% of the metal remaining in solution. An important feature of Figure 1 and 2 is that plant uptake of heavy metals followed a biphase rate of uptake when exposed to solutions having concentrations higher than 0.5 ppm, e.g., 2 and 5 ppm. This biphase rate of uptake included an initial concentration dependent rapid uptake phase followed by a much slower rate of uptake.



Percentage of Cd, Co, Ni, and Pb remaining in solutions (containing mixture of the four heavy metals at different concentrations) cultivated with *E. Crassipes.*

Fig. 1. Percentage of Cd, Pb, Co and Ni remaining in solution (containing mixture of the four heavy metals at different concentrations) cultivated with *E. crassipes*.



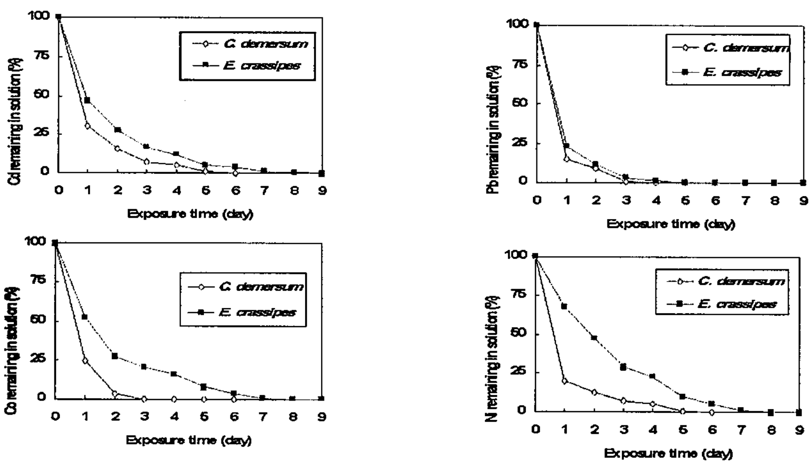
Percentage of Cd, Co, Ni, and Pb remaining in solutions (containing mixture of the four heavy metals at different concentration) cultivated with *C. demersum*

Fig. 2. Percentage of Cd, Pb, Co and Ni remaining in solution (containing mixture of the four heavy metals at different concentrations) cultivated with *C. demersum*

Irrespective of the plant type, the duration of the rapid initial phase of uptake varied from one metal to another, being one day for Cd and Co and 2 days for Pb and Ni. Thereafter, the much slower rate of uptake (second phase of uptake) continued for the remaining time. For example, and as a general trend, about 50% of each metal in the 2 ppm solution was removed by the plants in the first 2 days of exposure, while only 20% removed during the remaining 4 days of exposure. Another example was that 50% of total Cd and Pb were removed from the 5 ppm solution by *Ceratophyllum demersum* during the 6 day period; 90% of this removal occurred during the first 2 days of exposure. The results of the present experiment have practical implications. Due to the biphase rate of uptake, the plants were not able to completely purify the solution from heavy metals when the concentration is higher than 0.5 ppm. Thus, from the practical point of view, introducing new plants to the solution every 2 days could accelerate purification of highly polluted water. By this way, one makes use and benefit from the first rapid uptake phase by the plant and avoid the second much slower rate phase. This was tested in the following experiment.

Accelerating metal removal from solution

In this experiment, the two aquatic plants were exposed to a very high concentration (20 ppm) of either of Cd, Co, Ni or Pb, and new plants were replaced every two days. Figure 3 shows the daily percent metal remaining in solution. It could be seen that the metals were totally removed from the solution during a period that ranged from 3 to 7 days depending on the type of metal and kind of plant. *Eichhornia crassipes* was able to completely



Percentage of Cd, Co, Ni and Pb remaining in solution (containing single heavy metal at level of 20 ppm) after introducing new plants of E. crassipes or C. demersum to the solution every two days

Fig. 3. Percentage of Cd, Pb, Co and Ni remaining in solution (containing single heavy metal at level of 20 ppm) after introducing new plants of *E. crassipes* or *C. demersum* into the solution every two days.

purify the water from Pb in 4 day, Co and Ni in 7 day, Cd in 8 day. *Ceratophyllum demersum* showed greater ability in this respect. Thus, it was able to completely purify the water from Pb in 3 days, from Co and Ni in 5 days and Cd in 6 days. These results clearly confirm the feasibility of the technique of changing the absorbing plants at time intervals in accelerating the purification of highly polluted water.

Effectiveness of tested plants on wastewater purification

The two aquatic plants were grown on treated wastewater from Zenin treatment plant to study the capability of these plants to remove various metals from either primary or secondary treated wastewater. The concentrations of Fe, Mn, Cu, Cd, Co, Ni and Pb remaining in effluent over time and recommended maximum concentrations of heavy metals in irrigation water (RMC) are shown in Table 1. In comparison with RMC, both treated wastewater contained much higher concentration of Mn, Cd, Co and Ni. While the concentrations of Fe, Cu and Pb were lower than the permissible limits. After introducing the aquatic plants to the effluent, both plants were capable of totally deplete various heavy metals after 24 and 36 hours from secondary and primary treated water, respectively.

Table 1. The concentration of various heavy metals in treated wastewater cultivated with *Eichhornia crassipes and Ceratophyllum demersum.*

Heavy metal concentration (mg/l)	Primary treated wastewater Exposure time (hours)				Secondary treated wastewater Exposure time (hours)				R M C * (mg/l)
	0	12	24	36	0	12	24	36	
					E. crassipes				
Fe	1.95	0.10	nd	nd	0.82	0.03	nd	nd	5.00
Mn	0.73	0.05	nd	nd	0.59	0.02	nd	nd	0.20
Cu	0.11	nd	nd	nd	0.06	nd	nd	nd	0.20
Cd	0.10	0.04	nd	nd	0.08	0.02	nd	nd	0.01
Co	1.20	0.37	0.10	nd	0.92	0.20	0.07	nd	0.05
Ni	0.72	nd	nd	nd	0.47	nd	nd	nd	0.20
Pb	0.92	0.50	0.13	nd	0.82	0.05	0.05	nd	5.00
					C. demersum				
Fe	1.95	0.08	nd	nd	0.82	0.02	nd	nd	5.00
Mn	0.73	0.03	nd	nd	0.59	0.01	nd	nd	0.20
Cu	0.11	nd	nd	nd	0.06	nd	nd	nd	0.20
Cd	0.10	0.03	nd	nd	0.08	0.01	nd	nd	0.01
Co	1.20	0.40	0.08	nd	0.92	0.10	nd	nd	0.05
Ni	0.72	nd	nd	nd	0.47	nd	nd	nd	0.20
Pb	0.92	0.45	0.07	nd	0.82	0.20	nd	nd	5.00

* Recommended Maximum Concentrations of heavy metals in irrigation water for water used continuously on all soils (Westcot and Ayers 1985).

Thus, introducing *Eichhornia crassipes* and / or *Ceratophyllum demersum* to one of the stages of the wastewater treatment systems will increase the treatment efficiency by reducing the high concentration of heavy metals down to levels well below the permissible limits.

CONCLUSIONS

From previous results, it can be concluded that the rate of metal removal depended on its concentration, kind of metal and plant type. Replacing the standing plants by new ones every two days can accelerate metal removal from the solution containing high concentration of heavy metals.

Introducing *E. crassipes* and/or *C. demersum* to one of the stages of the wastewater treatment system can increase the treatment efficiency by reducing the high concentration of heavy metals down to levels below the permissible limits.

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Rasgas Water Systems

Shk. Khalid Abdulla Al-Thani

RASGAS WATER SYSTEMS

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ABSTRACT

RASGAS is a company specialized in Liquified Natural Gas (LNG). There are utilities, including the water system, which support the LNG operation. Basically there are three water systems used in RasGas: the Fresh Water System, the Sea Water System and the Waste Water System. The Fresh Water System is an important and integral part of RasGas, as it is required for firewater, boiler feed water, drinking water and make-up to the plant's closed circuit cooling water system. The water quality and quantity depend on the user's requirements, hence the water quality is monitored constantly to meet the highest standards. Seawater is used to cool the closed circuit fresh cooling water system in the plant. RasGas Company is one of the few companies that produce clean energy; which has no harmful impact on the environment. That is the main reason why RasGas' needs to treat all effluent waste water that is produced in the plant to meet the environmental specification for use the water for irrigation purposes.

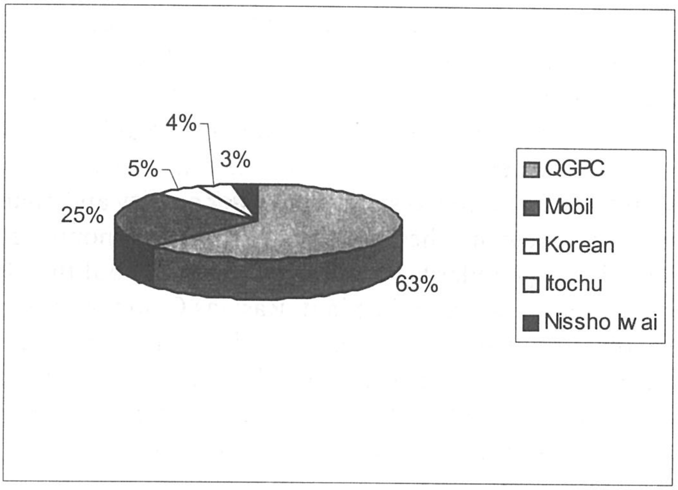
This paper discusses and summarizes the three water systems:

1. Fresh water system: to provide treated water to the process unit.
2. Seawater: for cooling the close circuit cooling water system
3. Wastewater system : to provide treated water for irrigation.

ABOUT RASGAS

Company History

Since the 1980's the state of Qatar has pursed a commitment to become a worldwide leader in LNG (Liquefied Natural Gas) production and sales. RasGas (Ras Laffan Liquefied Natural Gas Company Limited) was established by Emiri Decree in 1993. Currently, RasGas is owned by Qatar General Petroleum Corporation (QGPC), Mobil QM Gas Inc, Itochu Corporation and Nissho Iwai Corporation. A Korean Entity has an option to acquire a 5 % interest after which respective participating equity interests in RasGas will be:



RASGAS AS A MAJOR LNG PROJECT FOR THE NEW MILLENNIUM

RasGas will plateau at sales of 4.8 MMTPA (millions tons per annum) in 2003 and the first step in RasGas growth is to become a supplier of choice for LNG from the Arabian Gulf. With the signing SPA (Sales Purchase Agreement) with Petronet LNG of India to sell 7.5 MMTPA of LNG, and Heads of Agreement (HOA) with Dakshin Bharat Energy Consortium, also of India, for 2.6 MMTPA of LNG, RasGas has further enhanced its position as a major LNG supplier in the Middle East. The vision is a staged development, a multi-train LNG facility, responsive to the long-term needs of its buyers in existing and emerging markets. The onshore plant has been sized to accommodate up to six trains with potential to produce in excess of 15 MMTPA.

1. FRESH WATER SYSTEM

1.1 Objective

The primary purpose of RasGas Fresh Water System is to provide treated water for the process and utility equipment in the plant. As required water qualities are different depending on the purposes of users, the following types of water are produced and supplied to the users in the plant :

- Desalinated Water
- Service Water
- Potable Water
- De-mineralized Water
- Softened Water
- Fresh Cooling Water

1.2 Water Source

The water source used in each water system is fresh water produced in the desalinated water system. Seawater is a source of the desalinated water. The desalinated water is further treated in each water system to meet the required quality. The quality and quantity of targeted water are very important in choosing the required process.

1.3 Desalination unit

The desalinated water system consists of two thermo-compression desalination units that consist of a thermo-compressor and four-effect evaporator, Figures 1 and 2. The system also includes desalinated water storage tanks, pumps, and chemical feed equipment. Seawater is evaporated and condensed through the first effect evaporator to the fourth effect producing desalinated water. Desalinated water is collected and pumped to a common outlet header into desalinated water tanks. A conductivity analyzer is provided on the outlet of the desalinated unit with a high conductivity alarm in the CCB (Central Control Building). Normally the conductivity is less than 5 micro mhos. From the desalinated water tanks, the desalinated water is pumped to the following locations:

1. Softener unit for the boiler feed water and closed cooling water makeup

2. Remineralization unit for potable water production

3. Demineralization unit for gas turbine compressor washing

4. Fresh fire water tank

5. Service water system header

6. Fresh cooling water system

7. Irrigation water tank

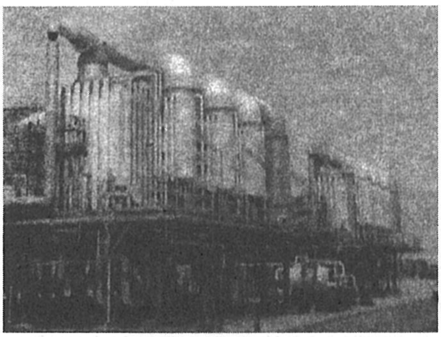


Figure (1) RasGas Desalination Unit A/B

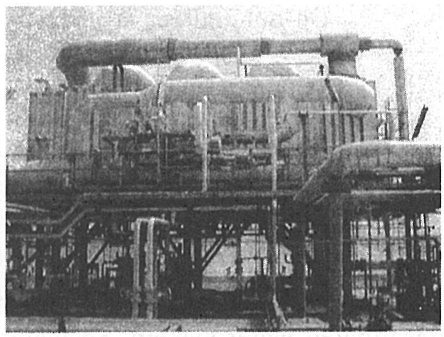


Figure (2) RasGas Desalination Front View

2.1 Softened Water System

The main objective of this system is to remove hardness from the desalinated water to prevent scale formation in the boilers, Figure 3. The desalinated water enters the exchanger and passes through the ion exchange bed. The exchange reaction takes place substituting the hardness ions (Ca^{2+} , Mg^{2+}) with sodium ions. The resin bed is exhausted and the water leaving the exchanger softened. Hardness is measured by the presence of Ca^{2+}, Mg^{2+} in water. Water hardness can be divided into classes normally, ***Carbonate (temporary)*** and ***noncarbonate (permanent).***

For regeneration of the softener bed, sodium chloride solution prepared from pure sodium chloride is used. The spent regeneration water is disposed into the seawater return header. Softened water is also used as make-up water for the fresh cooling water system.

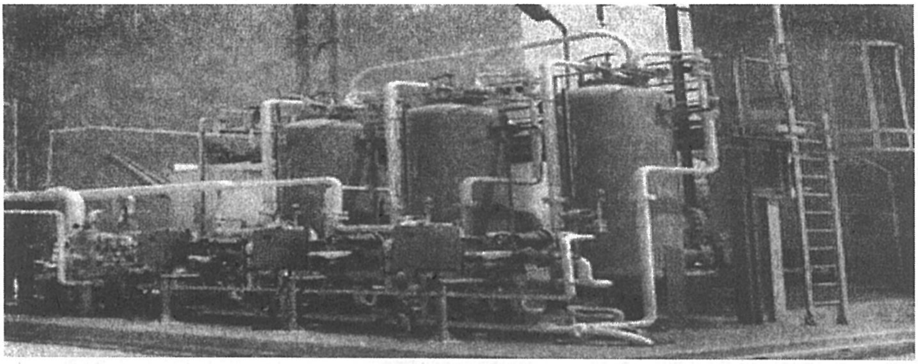


Figure (3) RasGas Water Softener Unit

FRESH COOLING WATER SYSTEM

The fresh water cooling system consists of fresh cooling water tanks, pumps, piping, heat exchangers and coolers, (Figure 4). Each LNG train is provided with a dedicated tank, pumps, piping and exchangers. The circuits can be interconnected for flexibility. Using a fresh water cooling system can save a lot of money from the material metallurgy and maintenance point view. Furthermore, a fresh cooling system protects the environment from any leakage in the plant process side contaminating the open sea.

The fresh cooling water pumps take suction from a common header, which is fed from both the fresh cooling water return and fresh cooling water tanks. The tanks provide a stable suction head for the pumps. The capacity of each tank is 1178 m^3. The fresh cooling water is supplied to the condensate stabilization unit, LNG trains, acid gas removal and utility plant. The return cooling water is cooled by exchange with once through seawater in 32 titanium plate fresh water coolers. From the coolers it is routed directly to the suction of the fresh cooling water pumps. The fresh cooling water supply and return design temperature are 38 and 47.3 deg C, respectively. Corrosion inhibitor and biocide are added to the cooling water with an eductor from drums using softened water as motive fluid. The eductor is located at the fresh cooling water pump suction.

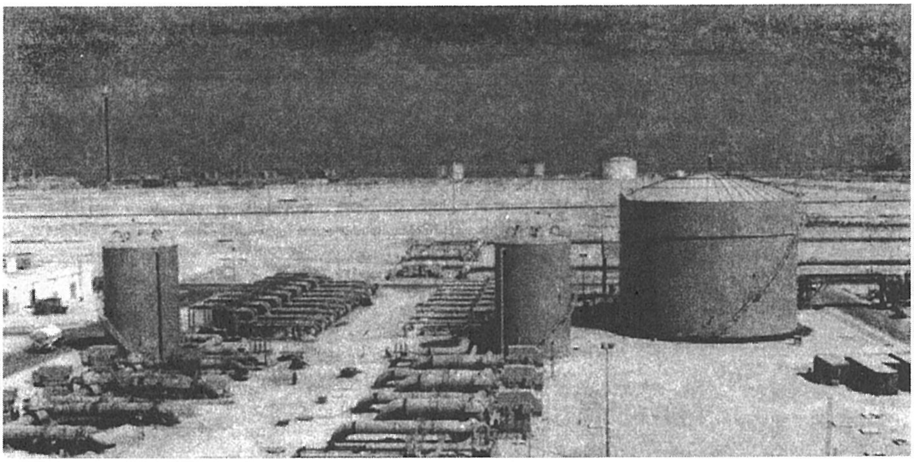


Figure (4) RasGas Fresh Cooling Water System

REMINERALIZATION WATER SYSTEM

The main objective of this unit is to add some minerals to the water to improve the taste since desalination water contains a very low concentration of dissolved solids. Remineralized water is used for drinking, Figure 5. The feed water to the Remineralization is the desalinated water. The desalinated water system consists of carbon dioxide injection, remineralization vessels chlorination, storage tank, pumps and piping. Piping material is primarily galvanized steel.

The remineralizer vessels are filled with dolomite, which gradually dissolves to add calcium and magnesium salt to improve the hardness in the desalinated water feed. The pH of the incoming water is adjusted by CO_2 injection to bring the hardness of the remineralized water into the range of 60 – 160 ppm. Additional CO_2 is injected downstream of the Remin beds to maintain the pH between 7 and 8.5

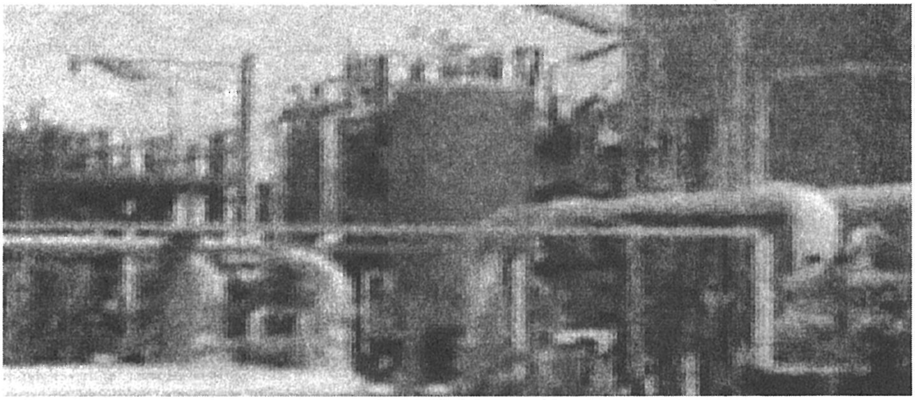


Figure (5) RasGas Remineralization Unit

SERVICE WATER SYSTEM

The service water system is a further distribution system of the desalinated water. The service is water utilized in the plant area as follows:

- Hose stations.
- Sanitary use in various buildings.
- Gas seals in oily water sewer manholes.

3. SEA WATER SYSTEM

Seawater is pumped from the intake system in Ras Laffan Harbour with a normal flow equal to 55,000 m^3/hr from 3 deep-well pumps. Separate pumps supply Seawater for emergency firewater. Seawater is filtered through bar screens and rotating mesh filters, and chlorinated to eliminate biological growth in the pumps and exchangers. Separate 66" supply headers are provided to each LNG train cooling water system, in which heat is exchanged with the circulating fresh cooling water in 16 titanium plate exchangers per train. A branch of Seawater Supply headers supplies the Desalination Plant with 52806 kg/hr. Seawater is returned to sea via a Wier box and open concrete channel, outside the harbour area with return temperature $< 10°C$ above supply temperature. Inlet chlorination is regulated to ensure < 0.5 ppm at the outfall.

4. WASTE WATER SYSTEM

The waste water system is comprised of four treatment facilities.

1. Process water is collected via the plant's oily water sewer system into either of two retention ponds, where oil is skimmed from the surface. The underflow from the retention ponds then passes through a Dissolved Air Floatation (DAF) separator and sand filtration to remove oil, suspended solids, and organics. A flocculent polymer demulsifier is injected into the feed line of the DAF separator.

2. A separate "chemical water" sewer system is provided to collect **waste water** which may need pH adjustment prior to discharge.

3. The sour water is degassed in a fuel gas stripper column to remove sulphide and mercaptans. Liquid hydrogen peroxide is added to the degassed sour water to oxidize the residual sulphide to sulphates.

4. The chemical water is collected into two batch operated neutralization sumps to adjust the pH. Sulphuric acid H_2SO_4 or caustic soda NaOH will be added to control the final pH to within 6 to 9.

5. The sanitary wastewater is biologically treated to remove the organics and suspended solids. Antifoam can be added to the aeration basin to control the froth on the basin's surface. The treated effluent is chlorinated to disinfect the wastewater. Calcium hypochlorite is dissolved and added for disinfection to maintain a residual chlorine concentration between 0.5 and 1.0 PPM.

The treated water is used to irrigate the land around RasGas administration building. Find the attached RasGas Plants layout around the administration building and type of the plants used where 60m^3/day amount of water that we use for irrigation and our main goal to meet the government specification.

CONCLUSION

RasGas's Fresh Water System can provide water for its industrial requirements, drinking and cleaning. RasGas's fresh water meets the highest standards as by the RasGas Key Performance Indicators (KPI). RasGas is also using the most innovative and effective process to treat all wastewater produced in the plant. This ensures that all environmental specifications are met and the can be used for water for irrigation purposes.

ATTACHMENT

1. Plants Layout.

2. Table 1 Concentration Limits for Water used for Irrigation.

3. Arabic Abstract.

REFERENCES

[1] Experience.

[2] RasGas Manuals.

Table 1 Concentration Limits for Water used for Irrigation

Determinant	Concentration (mg/l)	
	Maximum allowable	Monthly average-2
Oil and Grease -1	15	8
BOD5	10	-
Chemical Oxygen Demand	350	150
Total Organic Carbon	75	50
Dissolved Oxygen	-	2 (min.)
Total Dissolved Solids	1750	1500
Total Suspended Solids	10	-
Chlorine Residual	0.5	0.1
Ammonia (as N)	20	15
Phosphate (as P)	30	20
Total Kjeldahl nitrogen (as N)	60	35
Sulphate	400	300
Sulfide	0.1	0.05
Total Coliform	100 MPN/100ml	-
Chloride	1000	500
Cadmium	0.05	0.01
Mercury	0.005	0.001
pH	6-9	6-9
Sodium	1000	500
Sodium absorption ratio (SAR)	10	6
Aluminum	20	15
Arsenic	0.5	0.1
Barium	2.0	1.0
Boron	1.5	0.75
Chromium	0.2	0.1
Cobalt	0.5	0.1
Copper	0.5	0.2
Cyanide	0.1	0.1
Fluoride	15	10
Iron	10	5.0
Lead	0.5	0.1
Manganese	0.05	0.02
Nickel	0.5	0.2

Treatment of Water Turbidity

Nabil S. Abuzaid and Alaadin A. Bukhari

TREATMENT OF WATER TURBIDITY

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ABSTRACT

Ground water is a major water source in arid regions, however, if clay exists in the water containing aquifers, the discharged water becomes turbid. In fact, most clay particles are stable and remain in suspension. Therefore, they can not be separated by sedimentation or clarification. This study dealt with colloids separation from ground water in the Eastern Province of Saudi Arabia. The water was found to have a conductivity of 4400 umhos/cm, and a chloride and sulfate concentrations of 834 and 550 mg/l, respectively. The efficiency of using stainless steel soluble electrodes for the in-situ formation of ferric chloride and ferric sulfates coagulants has been investigated. The current input was found to be inversely proportional to the residual turbidity in the test water. At a contact time of 5 minutes and a natural chloride content, the highest turbidity removal efficiency (95 %) was achieved at a current of 1 A. When the current was reduced to 0.5 A and the contact time was increased to 10 minutes, the residual turbidity was reduced from 4.0 NTU to 1.6 NTU. Furthermore, similar turbidity removals were achieved at a much lesser contact time (2 minutes) when 1 g/l sodium chloride was added to the test water. Electro-coagulation is expected to be economically more feasible than conventional chemical coagulation. This is due to the cost items imbedded in chemical coagulation, mainly, the cost of the chemical, its transportation, storage, and handling. The findings of this study is important from practical point of view. The cost of turbidity removal by conventional coagulation can be reduced by the use of electro-coagulation.

Key words : Groundwater, Turbidity, Electro-coagulation, Current, pH

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INTRODUCTION

Ground water is a major water source in arid regions, however, if clay exists in the water containing aquifers, the discharged water becomes turbid. Most clay particles are stable and remain in suspension. Therefore, they cannot be separated by sedimentation. Bentonite causes turbidity to ground water because of its plasticity, colloidality, and stability in water (Mitchel, 1993). Colloidal dispersion may be stabilized by electrostatic repulsion between particles, arising from ions that are either adsorbed onto or dissolved out of the surface of the solid (Benefield et al., 1972). Destabilization of those colloids is essential in order to bring them in contact and aggregate. Coagulants can destabilize colloidal particles by four distinct mechanisms: double layer compression; charge neutralization; enmeshment in a precipitate, and interparticle bridging (Benefield et al., 1972).

Chemical coagulation is commonly used in water and wastewater treatment schemes. Among the coagulants commonly used are aluminum and ferric salts (Dental, 1991; Chion et al., 1983). Coagulation by the use of ferric chloride has been widely used and found very efficient in the removal of turbidity (Chion et al., 1983).

The use of electrochemical treatment methods for water and wastewater treatment has been investigated by different researchers (Ching et al., 1994; Awad and Abuzaid, 1997; Lin and Peng, 1997). Different types of electrochemical cells have been developed for that purpose. Among those are flow-through porous electrodes, fluidized beds, rolling tubes, packed beds, and rod electrode (Ching et al., 1994V). Electro-Coagulation was found to be a promising process for solid/liquid separation (Vik et al., 1984; Ogutveren, 1992; Donnini, 1994). It is expected to be economically more feasible than conventional chemical coagulation. This is due to the cost of purchasing, transportation, storage, and handling of chemical coagulants.

Generally, electro-coagulation results from the electrochemical dissolution of certain electrodes (a Faradaic process). In the case of stainless steel electrodes, iron ions are expected to be electrically generated. Dissolved ferric iron hydrates, coordinating six water molecules and forming an aquometal ion, $Fe(H_2O)_6^{3+}$ (Chion et al., 1983). The aquometal ion can then hydrolyze and form monomeric and polymeric ferric species, the formation of which is highly pH dependent. At any pH, the maximum dissolved concentration of ferric ions in equilibration with the hydroxide solid is determined by the solubility of the solid phase, the amorphous ferric hydroxide, $Fe(OH)_3$, and by the extent of formation of monomeric and polymeric hydrolyses species in solution (Chion et al., 1983). The amorphous solid ferric hydroxide flocs settles down causing sweep flocculation of the stable suspended colloids. On the other hand, the charged hydrolyzed hydroxides can neutralize the negatively charged colloidal clay particles.

Donini et al. (1994) have studied the operation cost of electro-coagulation. The cost items were the electrical power and the consumption of aluminum electrodes. The influence of sodium chloride concentration, flow rate, and the passivation layers on the economy of the process were studied. Ogutveren et al. (1992) worked on the electro-coagulation of dye stuff from wastewater using soluble steel electrodes. Complete removals were achieved within very short contact times (3-5 minutes). Vik et al. (1984) have investigated the electro-coagulation of humus material from potable water. Their work has shown electro-coagulation to be efficient for that purpose. Additionally, they have listed several advantages for electro-coagulation over conventional coagulation. Among those advantages are the reduced chemical dosages and sludges in the case of electro-coagulation compared to conventional chemical coagulation.

The main objective of this study is to investigate the electrochemical removal of turbidity from ground water (electro-coagulation) in the Eastern Province of Saudi Arabia. Stainless steel electrodes are used as a source of ferric ions.

MATERIALS AND METHODS

A schematic diagram of the electrochemical setup used in the study is illustrated in Figure 1. Two stainless steel electrodes, each with a surface area of 50 cm^2, were immersed in a 750 milli-liter container of turbid ground water resulting in a spacing between each anode and cathode of 3 cm. The power was supplied by a DC source (Hampden, USA) while the current was kept invariant in each test by a rheostat (Engield-Middlesex, UK) and measured by an ammeter (Hampden, USA).

The water used in the study was a ground water from the Eastern Province of Saudi Arabia. It was stored in a central tank that is connected to the lab by a pipe. The water was collected from the end valve in the lab and analyzed for different parameters. The analysis was performed following the procedures listed by standard methods (APHA, 1985).

A stock solution of turbid water was prepared in a 20 L glass container by adding a certain amount of bentonite to the natural ground water in order to produce a turbidity of 76 Nephlometric Turbidity Unit (NTU). In each experimental run, a solution volume of 0.75 L was withdrawn from the stock solution and placed in the electrolytic cell. Mixing was done by a magnetic bar-stirrer. Table salt (sodium chloride) was added in certain experiments to the turbid water in order to increase its conductivity and elucidate the role of chloride ion concentration on the process efficiency.

The effect of current on the coagulation process of the turbid water was investigated at a contact time of 5 minutes. The current responsible for optimal turbidity removal was chosen and experiments were conducted at different contact times. Experimental work was extended to investigate the effect of electrolyte concentration. This was done by varying the concentration of sodium chloride while keeping the optimum current and contact time found from previous experiments.

In each of the above mentioned experiments, after the pre-specified contact time in the electrochemical cell has passed, the water was removed, measured for pH, and placed under a variable stirring apparatus. The water was rapidly mixed (100 rpm) for one minute and moderately mixed (40 rpm) for an additional 20 minutes. Mixing was performed for the purpose of floc formation and enlargement. After the mixing stage, the water was left for 20 minutes in order to allow the flocs to settle. Subsequently, the supernatant turbidity was measured using a turbidity meter (Model 40 Nephelometer, USA).

RESULTS AND DISCUSSION

The results of the chemical analysis for the ground water under study are tabulated in Table (1). Those parameters were analyzed in order to have a clear idea about the chemistry of the water under study and to help in understanding the interaction of electro-coagulation with the test water.

Table 1. Results of the Chemical Analysis for the Ground Water under Study

Parameter	Value
pH	7.26
Total Dissolved Solids, mg/l	2680
Conductivity, umhos/cm	4400
Alkalinity, total as $CaCO_3$, mg/l	151
Chloride as Cl^-, mg/l	834
Sulfate as $SO4^{-2}$, mg/l	550
Sodium as Na, mg/l	540
Turbidity, NTU	0.6

The data presented in Table (1) shows that the ground water contains high concentration of total dissolved solids that has reflected on its conductivity. This property is extremely important from electrochemical point of view since high conductance allows the passage of current at a relatively low voltage. Additionally, the high concentrations of chloride and sulfate ions will enhance the electrically in-situ formation of coagulants.

From Table (1), it is obvious that the water has very low turbidity (0.6 NTU). Accordingly, the water turbidity was artificially increased by the controlled addition of bentonite as illustrated under the experimental section. This was done in order to be able to confirm the effect of electro-coagulation as a turbidity removal process for ground water.

As a result of electrical current, ferrous ions (Fe^{2+}) dissolute from the steel electrodes. Ferrous ions react with hydroxyl ions (OH^-) that prevail at elevated pH values to produce ferrous hydroxide ($Fe(OH)^2$) which reacts with dissolved oxygen to produce ferric hydroxide ($Fe(OH)^3$) flocs. Ferric hydroxide is used in solid liquid separation. In the electrochemical process, when the applied voltage exceeds a certain value, hydrogen evolution occurs. Therefore, the pH of the water starts to increase which would result in an increase of the concentration of the hydroxyl ions.

The Chloride ions naturally existing in the groundwater are oxidized at the anode to form chlorine gas (Montgomery,1985). If organic chemicals exist in the groundwater to be electro-coagulated, the generated chlorine gas would react with them producing chlorine derivatives of those compounds. For example, chloro-phenols result from the reaction of chlorine with phenol. Chlorinated organic compounds are harmful to health and many of them are carcinogens (Verschueren, 1977).

As indicated previously, the purpose of this work was to find the optimal conditions that would, in-situ, generate the amounts of chemical coagulants needed for turbidity removal. It is worth mentioning that generating surplus amounts of those coagulants would not be only uneconomic but also may lead to reverse results from turbidity removal point of view.

The effect of current was studied by conducting electro-coagulation experiments at different current values while other parameters such as contact time, initial turbidity, and electrolyte concentration were kept constant. A contact time of 5 minutes, a conductivity of 4400 µs/cm, and an initial turbidity of 76 NTU were adapted.

Figure 2 depicts the relationship between current variation and the test water final turbidity. The figure shows that the residual turbidity decreases with the increase in current. However, between currents of 0.8 and 1 A, the decrease in residual turbidity was very low. Therefore, it was not feasible to test the effect of higher currents. The Turbidity Removal efficiency was calculated as $\{(T_o - T_t)/ T_o\}$ where T_o is the initial turbidity in the test groundwater (76 NTU) and T_t is the turbidity at the end of the experimental run (at a certain value of current and detention time). The highest turbidity removal efficiency (95 %) was achieved at a current of 1 A. This finding suggests that electro-coagulation is an effective technique for the removal of turbidity from ground water. It should be also emphasized, here, that this high removal efficiency was attained without adding any electrolyte to the water.

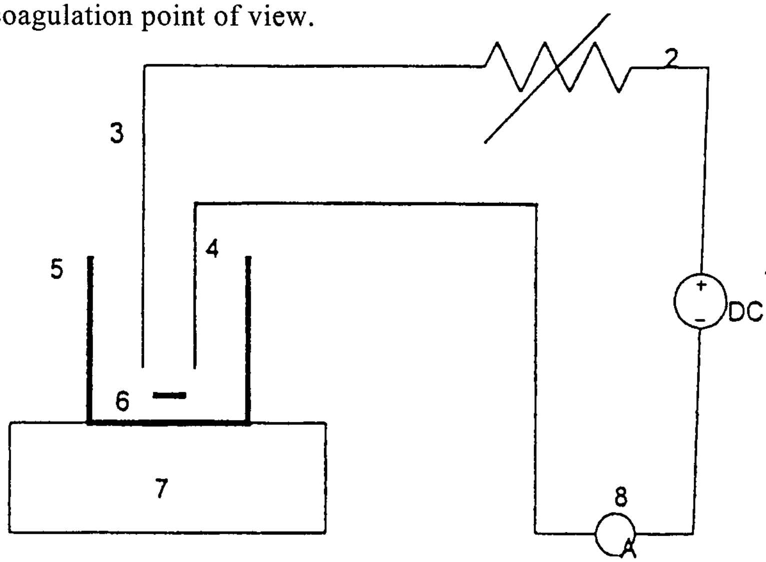
As stated earlier, the whole idea of electro-coagulation depends on the electrical dissolution of the metal from the electrode. Therefore, it is important to estimate the amount of iron electrically dissolved. This was done using the following form of Faraday's law :

$$w = \frac{itM}{ZF}$$

w	=	iron dissolving (g)
i	=	current (A)
t	=	time (s)
M	=	molecular weight of Fe (M= 55.85)
Z	=	number of electrons involved in the redox reactions (Z=2)
F	=	Faraday's constant = 96500.

The amounts of iron dissolved under each set of experimental conditions were calculated and shown in Figure 2. Although it did not result in the highest removal efficiency, it is suggested that a current of 0.5 A should be adapted. At this current, the turbidity removal efficiency was 94.8 % with a dissolved iron concentration of 43.4 mg/l. At a current of 1 A, the amount of dissolved iron calculated was twice as much as that calculated at a current of 0.5 A. However, the increase in current from 0.5 to 1 resulted in a slight increase in the removal efficiency (from 94.8 to 95%). Therefore, adapting the 0.5 A current would make more sense from economical point of view as will be discussed later.

The coagulation process is highly dependent on the pH of the water to be treated (Chion et al., 1983). In electro-coagulation, the solution pH increases with time due to hydrogen evolution and this increase is mainly current dependent. Figure 3. shows the relationship between solution final pH and the current applied. From the figure, it can be seen that up to a current of 0.05 A, the pH decreased with the increase in current. This is due to the fact that the voltages responsible for this current range is below the hydrogen evolution voltage. However, those low current values would be capable of converting the chloride in the water to chlorine gas (Table (1). The later forms hypochloric acid in water solution causing a reduction in the solution pH. After a current of 0.05 A, the solution pH increases with the increase in current. This behavior has affected the relation between the residual turbidity and the applied current (Figure 2). Ching et al. (1994) reported that although, coagulation with iron salts occurs at a wide range of pH due to different mechanisms, the amorphous ferric hydroxide is least soluble at a pH close to 8. The precipitation of ferric hydroxide at elevated pH gives rise to the sweep floc coagulation mechanism. Hence, the increase in pH in an electrochemical cell due to hydrogen evolution is advantageous from coagulation point of view.



1. D.C. Power Supply
2. Variable Resistance
3 & 4. Stainless Steel Electrodes
5. Pyrex Container
6. Magnetic Bar Stirrer
7. Magnetic Stirrer Controller
8. Ammeter

Figure 1. Schematic Diagram of the Electrochemical Cell

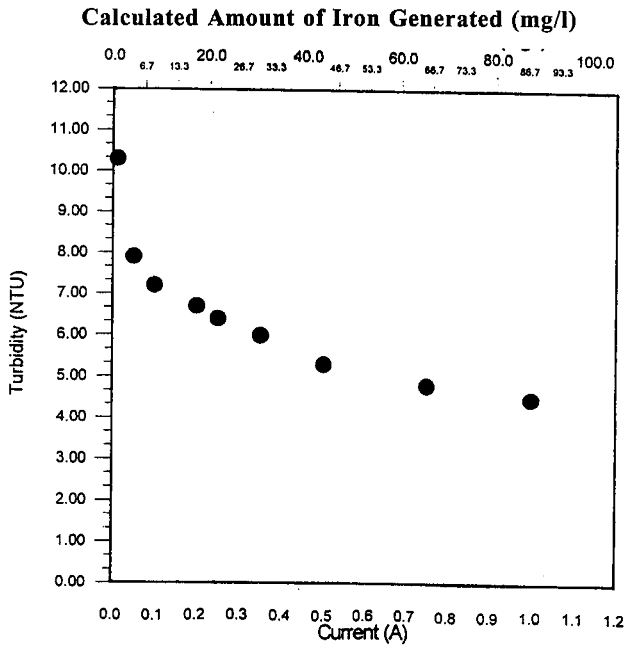


Figure 2. The Effect of Current Variation on the Residual Water Turbidity, (initial turbidity = 76 NTU, contact time = 5 minutes)

In order to find the contact time that yields optimum removal results, experiments were conducted at different contact times while the applied current was kept constant (0.5 A). The choice of this current was based on the previous findings. Figure 4 shows the temporal variation of residual turbidity under the effect of a current of 0.5 A. The figure also shows the residual turbidity as a function of the amounts of iron generated calculated using Faraday's law. An optimal residual turbidity of 1.6 NTU is found at a contact time of 10 minutes and a calculated amount of iron generated of 86.8 mg/l. This shows that by increasing the contact time from 5 to 10 minutes at a constant current of 0.5 A, the residual turbidity was reduced from 4.6 to 1.6 NTU.

Studying the results of Figures 2 and 4 reveals the importance of contact time as an operating parameter rather than a merely iron generating parameter. A calculated amount of iron generated of 86.8 mg/l was achieved at two sets of operation conditions, first, current of 1 A and contact time of 5 minutes and second, a current of 0.5 A and a contact time of 10 minutes. However, although, the two experiments generated the same amount of iron, residual turbidities of 4.2 and 1.6 NTU were obtained under the first and the second sets, respectively. Hence, the effect of contact time on the process performance is obvious. The effect of contact time on turbidity removal is thought to be due to three reasons; dissolved iron generation, coagulation reactions time dependence, and pH increase due to the time dependent hydrogen evolution. The third potential reason can be proven through the results of Figure 5 that depicts the relationship between the solution final pH and the contact time at a current of 0.5 A. The figure shows that at a fixed current of 0.5 A, the solution final pH increases with the increase in contact time. This is simply, due to the evolution of more hydrogen at longer contact times. As stated earlier, the mere increase in pH increases the coagulation and flocculation potential (Chion et al., 1983).

The conductivity and the chloride ions concentration were increased in certain experiments by the controlled addition of NaCl to the test solution at a constant current of 0.5 A and the initial turbidity of 76 NTU. The purpose was to have a better understanding of this process, particularly in relation to the contact time. Figure 6 shows the temporal effect of NaCl concentration on the turbidity removal at the optimum current found earlier. The figure shows that the lowest residual turbidity concentration was 1.6, 2.2, and 2.2 at natural chloride content, 0.5 g/l NaCl, and 1 g/l NaCl, and contact times of 10, 7, and 2 minutes, respectively. This means that for similar optimal residual turbidities at a constant current of 0.5 A and contact times of 10, 7, and 2 minutes, the calculated amounts of iron generated are 86.8, 60.7, and 16.4 mg/l, respectively. The reduction in the contact time needed to achieve similar residual turbidities due to the addition of NaCl may be attributed to the increase in conductivity and to the formation of more ferric chloride coagulant. This shows that the formation of ferric chloride was controlled by the natural chloride in the test water.

The explanation for having similar turbidity removals at different iron concentrations may be based on the findings of Stumm and O'Melia (1968). Figure 7 shows a schematic presentation of their results (1). At a given colloid concentration, in zone 1, insufficient coagulant dose has been applied, and destabilization does not occur. Increasing the coagulant dosage results in destabilization (zone 2) and hence, turbidity removal. Further increase of the coagulant dosage can lead to restabilization of the bentonite dispersion (zone 3). A sweep-floc coagulation occurs (zone 4) if the coagulant dosage is further increased.

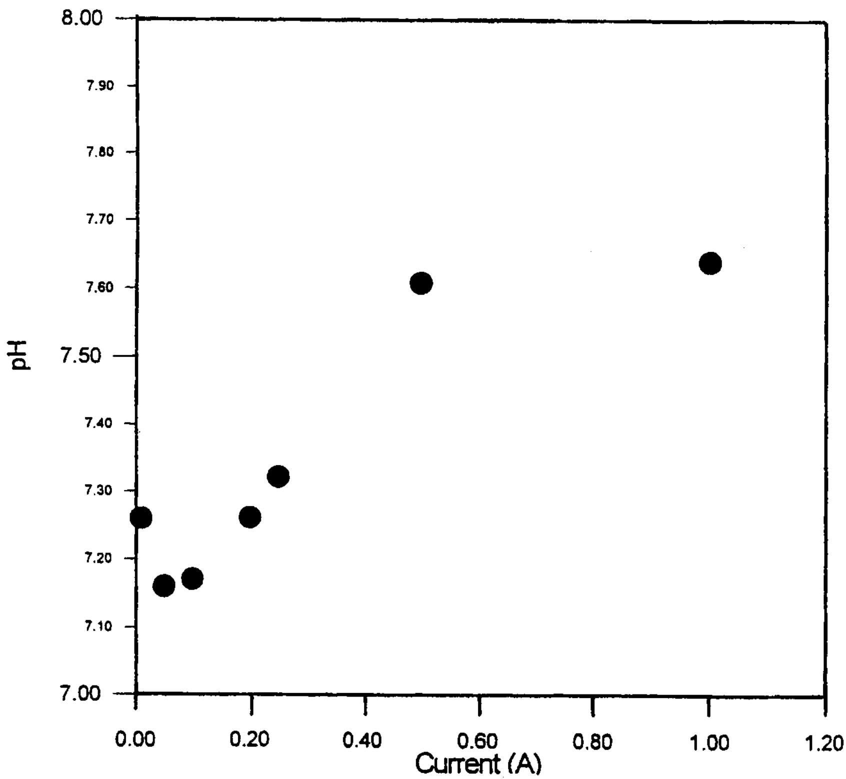


Figure 3. Relationship between the Water Final pH and the Applied Current at a Contact Time of 5 Minutes

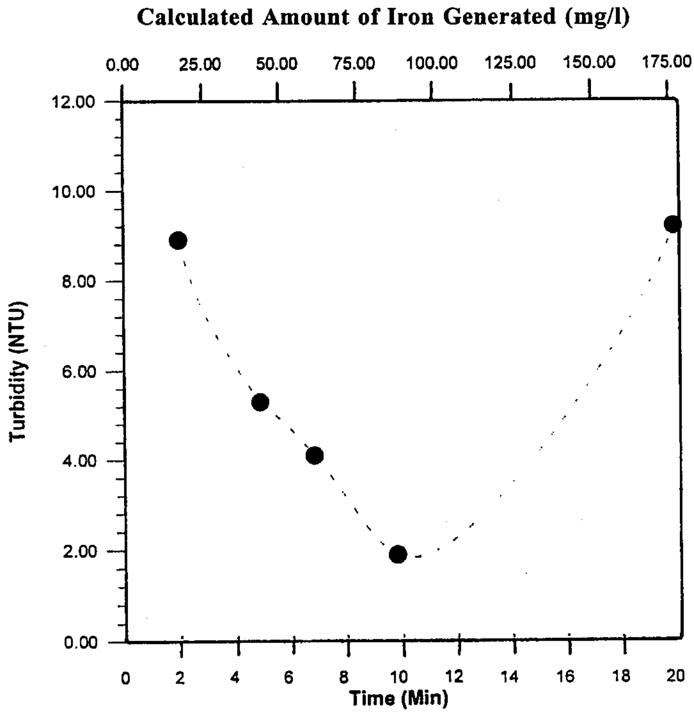


Figure 4. The Temporal Variation of Residual Turbidity under a Current of 0.5A, (initial turbidity = 76 NTU)

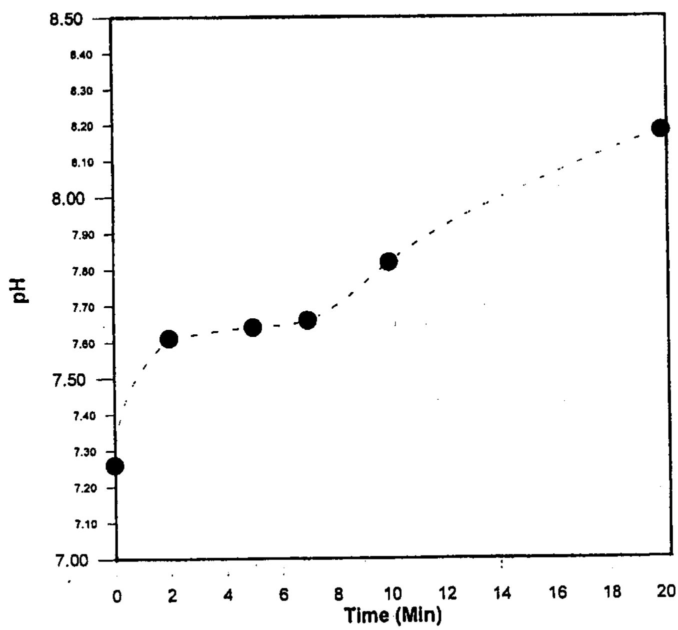


Figure 5. The Temporal Variation of Water pH under a Current of 0.5A

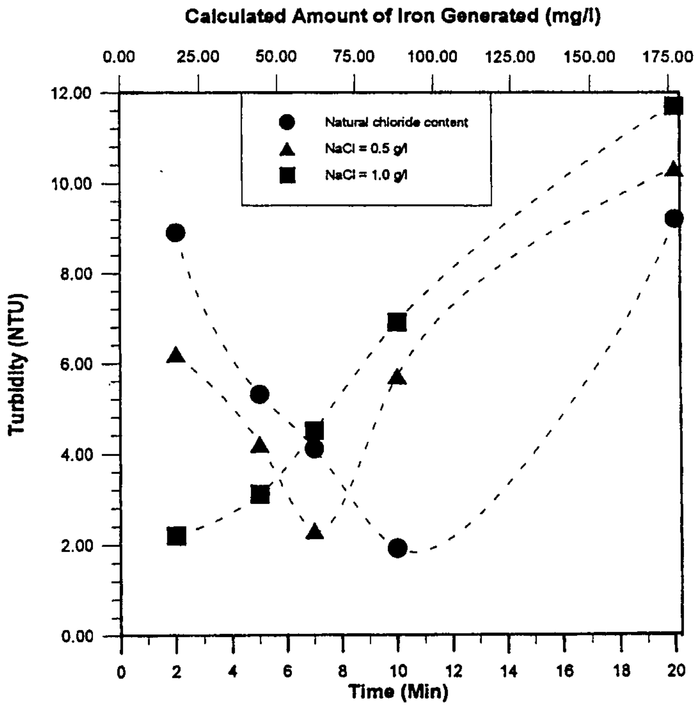


Figure 6. The Temporal Effect of NaCl Concentration on the Residual Water Turbidity at a Current of 0.5A, (initial turbidity = 76 NTU)

The relationship between solution final pH and NaCl concentration is depicted as a function of contact time in Figure 8. The figure shows that, generally, at a constant current of 0.5 A and all NaCl concentrations, the solution final pH increases with the increase in contact time due to the reason explained earlier. Additionally, no effect of the NaCl concentration on the solution final pH is seen up to a contact time of 7 minutes. After that, the pH is inversely proportional to the NaCl concentration. This trend is attributed to two reasons; first, the increase in NaCl concentration increases the conductivity of the solution which reduces the voltage needed to have a certain current. Voltage reduction would reduce the hydrogen evolution which is responsible for the increase in pH. Second, the increase in NaCl concentration increases the amount of chloride ions which can be electrically converted to chlorine gas. The increase in chlorine gas production increases the concentration of hypochloric acid in the test water. This tends to reduce the solution final pH.

CONCLUSIONS

This study investigated the efficiency of stainless steel soluble electrodes for the turbidity removal from ground water in the Eastern Province of Saudi Arabia. The work started by chemically analyzing the water different parameters. It was found to have a conductivity of 4400 ìs/cm, a chloride concentration of 834 mg/l, and a sulfate concentrations of 550 mg/l. In order to test the process at high turbidity levels, the turbidity of the water was increased to 76 NTU by the addition of bentonite. Actually, the process is looked at as an in-situe production of ferric hydroxide coagulant resulting from the dissolved iron generating from the electrodes. The experimental work was designed to investigate the effect of current input, contact time, water final pH, and electrolyte concentration on the turbidity removal efficiency of the cell.

The process has shown excellent turbidity removal efficiencies. The current input was found to be inversely proportional to the residual turbidity in the test water. The highest turbidity removal efficiency (95 %) was achieved at a current of 1 A, a contact time of 5 minutes, and a natural chloride content. Reduction of the current from 1 A to 0.5 A and increase of contact time from 5 minutes to 10 minutes has caused the residual turbidity to drop from 4.0 to 1.6 NTU. A similar residual turbidity was achieved at a much shorter contact time (2 minutes) when 1 g/l sodium chloride was added to the test water.

The influence of the pH increase due to the voltage induced hydrogen evolution on the coagulation efficiency was studied as well. While the solution final pH increased with the increase in current and contact time, it was found to decrease with the increase in sodium chloride concentration. However, due to its relevance to the coagulation process, more work needs to be done to elucidate the mechanism of the hydrogen evolution phenomenon.

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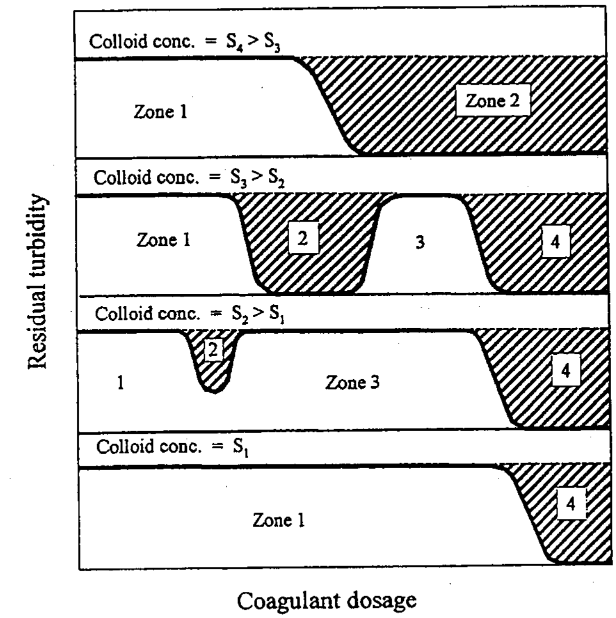

Figure 7. Schematic Coagulation Curves at Constant pH, Shaded Areas Represent Regions in which Coagulation Occurs (after Stumm and O'Melia (1968)

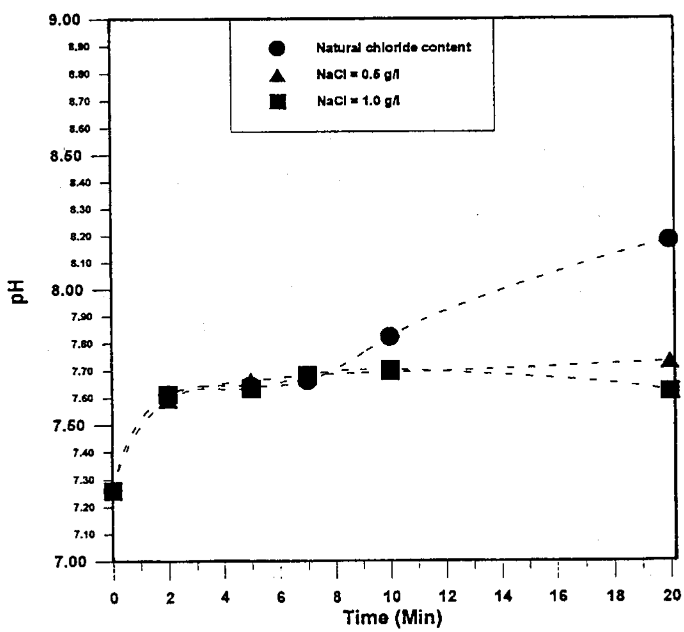

Figure 8. The Temporal Effect of NaCI Concentration on the Water Final pH at A Current of 0.5 A

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