MEDRC

MEDRC Series of R&D Reports
MEDRC Project: 97-BS-015

BEACH WELL INTAKES FOR SMALL SEAWATER REVERSE OSMOSIS PLANTS

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August 2000

MEDRC Series of R&D Reports
Project: 97-BS-015

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THE MIDDLE EAST DESALINATION RESEARCH CENTER

An International Institution
Established on December 22, 1996
Hosted by the Sultanate of Oman in Muscat

Mission Objectives of the Center:

1. to conduct, facilitate, promote, co-ordinate and support basic and applied research in the field of water desalination and related technical areas with the aim of discovering and developing methods of water desalination, which are financially and technically feasible

2. to conduct, facilitate, promote, co-ordinate and support training programs so as to develop technical and scientific skills and expertise throughout the region and internationally in the field of water desalination and its applications and related technical areas

3. to conduct, facilitate, promote, co-ordinate and support information exchange, including, but not limited to, electronic networking technology, so as to ensure the dissemination and sharing throughout the region and internationally of technical information concerning water desalination methods and research and related technical areas, and to establish with other states, domestic and other organizations such relations as will foster progress in the development, improvement and use of water desalination and related technical areas in the region and elsewhere

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GLOSSARY

BMS	-	Beach Management Systems
CF	-	Cartridge Filters
FV	-	Future Value (financial Function)
MEDRC	-	Middle East Desalination Research Center
MENA	-	Middle East and North Africa Region
MSF	-	Multi Stage Flash Desalination Process
O&M	-	Operation and Maintenance
PAC	-	Project Advisory Committee
PMT	-	Annual Payment (financial function)
PV	-	Present Value (financial function)
PWA	-	Palestinian Water Authority
RO	-	Reverse Osmosis
ROT	-	Reverse Osmosis Train
SDI	-	Silt Density Index
SUTRA	-	Saturated - Unsaturated Transport Model
SWRO	-	Seawater Reverse Osmosis
TAHAL	-	TAHAL Consulting Engineers, TAHAL Group
TDEM	-	Time Domain Electromagnetic Method
USGS	-	United States Geological Survey

ACKNOWLEDGEMENTS

The present study was initiated and financed by MEDRC - Middle East Desalination Research Center. The study was carried out by TAHAL Consulting Engineers Ltd. The following experts participated in the study:

J. Schwarz	-	Principal Investigator
E. Shachnai	-	Hydrogeologist
S. Lumelski	-	Groundwater Modelling
U. Hurwitz	-	Economics
Y. Maoz	-	Desalination Expert
I. Zacharias	-	Hydrogeologist
O. M. Awwad	-	Hydrogeologist

The first five experts listed are personnel of TAHAL Consulting Engineers Ltd-(Main Contractor) Israel. The last two experts listed were subcontracted from:

- NCMR, National Center for Marine Research - Greece
- PWA - Palestinian Water Authority West Bank/Gaza

An important contribution to the study was permitted by KTH - Kungl Tekniska Hoegskdan, Sweden, by Professor Georgia Destouni and Dr. Carmen Prieto who performed the SUTRA model runs for the Nitzanin area and parts of Appendix B on the Dynamics of Seawater Intrusion into Aquifers.

The Israel Water Commission granted permission to use and cite publications of their studies.

Valuable information was received from many other organizations and persons mentioned in the Report in response to questionnaires and queries.

ABSTRACT

The present study provides the desalination industry (in its small SWRO sector) an improved design solution for seawater Intakes. Non-surface Intakes *i.e*. Beach wells or galleries were proposed to replace the conventional surface seawater Intakes. These types of Intakes would enable the supply of feed-water to have improved quality and reliability. Under favourable site conditions (such as granular formations with sufficient hydraulic conductivity but still with adequate filtration capacity), a significant reduction of the Intake cost as well as a reduction of pre-treatment cost was expected. Such favourable conditions exist in many of the regional sand and sandstone coastal aquifers in the MENA countries. The potential impact on the total cost of desalination can be significant, particularly for small SWRO plants.

The study products included; principles and guidelines for selecting sites and technologies and for assessing site properties. A data processing framework has been developed including spreadsheets for cost estimates based on site properties. These enable scientifically based and competent decisions on the feasibility of non-surface Intakes.

Following the completion of the present study and prior to its implementation in a certain coastal region, additional efforts will be required to assess site properties by the proposed assessment methods.

EXECUTIVE SUMMARY

1. Introduction

Beach wells and galleries extracting groundwater of seawater quality exist in Israel and elsewhere for the purpose of *e.g.* swimming pools and industrial cooling. Experience in the application of similar Beach wells and galleries as Intakes for Seawater Reverse Osmosis (SWRO) is still limited. However, there are indications that such an Intake type can provide reliable quantity and better quality water than surface Intakes due to natural filtration and underground detention. This is a significant advantage in view of the history of failures of SWRO membranes caused by adverse marine conditions requiring advanced pre-treatment (which was not available).

Because of their positive impact on feed-water quality, non-surface Intakes promise an opportunity for better efficiency, reliability, cost effectiveness, performance of desalination plants in general and SWRO plants in particular and therefore are of a significant benefit to desalination technology.

There are several types of non-surface Intakes. They include Beach wells, seabed filtration and inflow galleries. These different Intakes represent design variations that utilize the same principle of extracting filtered seawater from below the surface near the shoreline. Each of these Intakes has its own advantages, capabilities, suitability, and cost-effectiveness for different site conditions.

Relevant site conditions are predominated by the hydrogeological properties. These determine the Intake structure size, available water flows, quality of the feed water and possible environmental impacts. Consequently, these properties determine the sites' total cost effectiveness, which is the criterion used for evaluating the feasibility of different surface and non-surface Intake alternatives.

A particular concern is, the identification of exploration methods and survey techniques to aid in determining these site properties required for the selection of appropriate sites and Intakes. These include conventional and new exploration and survey methods for determining the relevant hydrogeological site conditions.

2. Project approach

The objective and essential goal of the present study is to provide a comprehensive state-of-the-art document on utilizing Beach wells and similar non-surface seawater Intakes for SWRO systems, to develop and verify the criteria for the choice and design of these types of seawater Intakes.

This study presents the considerations in analyzing the possibility of the different types of Intakes, their advantages and disadvantages, and presents an economical analysis of the methods based on a model developed for this study.

The study focuses on identifying the site-specific hydrogeological conditions that determine the choice of Intake types and their feasibility. The main properties are:

aquifer lithology, thickness and hydraulic conductivity. Feasibility is expressed in terms of anticipated direct and indirect costs, advantages of the proposed technology in the desalination process and anticipated problems on its implementation.

The main concern for comparing different water Intakes was the resulting direct cost of feed-water at the inlet of the SWRO plant (after pre-treatment), and the reduction of treatment cost due to the improved feed-water quality.

The second concern was the indirect costs representing the environmental impact of pumping seawater on the groundwater system, *i.e.* its impact on the sustainability of a fresh groundwater supply from existing and planned pumping wells.

The quantitative assessment of these concerns needs some type of models. The following three types of model have been used in the present study:

1. A geohydrological model (SUTRA) represents the performance of Intakes in terms of quantity and salinity of the pumped water and their variations over time as well as the impact of the Intakes on the groundwater inland and the reduction of inland groundwater availability. This special dual fluid simulation model was applied to study the geohydrological considerations, to set the optimal location and depth of the Intake, and to assess the environmental impacts on the fresh groundwater stock and its use.

2. A techno-economic model represents the cost elements of the total system including delivery/disposal of product water and of the brine produced. This model also includes an assessment of the indirect costs based on the results of the geohydrological model. This model was used for compiling and assessing the values of the cost elements for the given site properties. In conclusion, the economic evaluation of product-water cost is shown based on the values of these cost-determining elements.

3. Models for estimating the energy requirements for desalination, based on the chemical composition of the feed water and the recovery ratio.

Data for the present study were collected in the following manner:

a) Literature survey
b) Direct contacts with data sources
c) Formulation of questionnaires and analysis of the responses received

The following sources were identified and approached for collecting the relevant data:

a) Literature survey of existing library documentation and Internet explorers
b) Senior hydrogeologists with experience in Beach wells and coastal hydrogeology
c) Beach well operators in the Mediterranean, Red Sea and other coasts
d) Desalination process experts and plant operators

e) Israeli and other groundwater modellers with experience in the hydraulics of the two phase flow in coastal aquifers and interface configuration

The study covered the following tasks:

a) Identification and description of the various types of non-surface seawater Intake methods
b) A literature survey and evaluation. Collection of existing experience reports, case studies, records of failure and success for both conventional SWRO with surface Intakes and Beach wells for SWRO and other purposes
c) Direct communication with experts and using questionnaires to collect experience reports, case studies and records of failures and successes
d) Description of the characteristics of the various non-surface Intake technologies with respect to raw feed-water flow, filtration, capacity, effect of soil/ground properties, effect on water composition, life time, maintenance, *etc*.
e) Identification of the seawater desalination processes and site conditions so as to determine which of seawater Intakes are viable and cost effective
f) Description and evaluation of simulation and other design methods
g) Development of guidelines and criteria for site-specific selection of Intake type
h) Description of existing and future site survey approaches and exploration techniques, which may be applicable in site and process selection
i) Assessment of the costs of Beach well Intake systems and comparison with surface Intake alternatives. Identification of site conditions (coastline geology, beach soil and subsurface stratigraphy, seawater quality, *etc*.) for which these Intakes are the best economic solution

The following data were collected:

a) Description of the characteristics and design principles of the various non-surface Intake technologies
b) Performance of existing Intakes used for small SWRO including the existing Beach wells
c) Performance of existing Beach wells and other non-surface Intakes used for purposes other than SWRO
d) Models for assessing the discharge of Intakes depending on aquifer properties and on the Intake design
e) Models for assessing the hydrogeological impact of abstraction on the near fresh water wells

The applicable products of the study were:

a) Guidelines for the selection of an appropriate Intake type, given the size of plant and site conditions
b) A simple general-purpose spreadsheet that summarized the underlying cost assessment procedure
c) Guidelines for the assessment of site properties that were determined in the cost and feasibility assessment

3. Findings

The following findings of the study are described in the present report:

a) The main purpose of the Intake system is to provide a reliable source of feed water to the desalination plant with a proper quality and nominal quantity

b) The problem of suspended matter and other contaminants in seawater Intake used for SWRO plants; is one of the major problems for plants installed along seacoasts. The clogging of feed water flow by fishes, shells, weeds, algae, sewage, oil residue *etc*., causes a decline in the lifetime of membranes, lower feed and product water flows, increase in pre-treatment equipment investments, higher energy consumption, and thus higher costs of desalted water

c) The main Intake systems are divided into two categories: surface systems and non-surface systems. The surface systems are based on installing one or two pipes on the bottom of the sea and pumping the feed seawater into the desalting plant. The main under-water works that are required for the surface system include: excavation, embedment, dredging, pipe laying, pipe anchoring, pipe joining and pit assembling. The non-surface systems are based on pumping feed water from wells or other subsurface structures located close to the seacoast, where the dissolved salts concentrations are similar to seawater concentrations

d) A non-surface system comprises of wells (drilled boreholes), well casing, a screen, submersible pump, pump starters, and interconnecting piping. A surface system usually includes one or two pipelines, or a channel, to convey seawater to the pump pit, a coarse trash removal system to prevent the intrusion of fish and large objects and seawater transfer pumps

e) Beach wells and other non-surface seawater Intakes have been proven a technique to replace the surface seawater Intake system. The Beach well system has been successfully used with the minimum pre-treatment for SWRO plants in some locations around the world

f) Beach wells using water for cooling installations and swimming pools were installed in Israel in the early 60's; near the Mediterranean Coast. The typical dimensions of such a well are: 90 m total depth with a 20 m screen of 25.4 cm (10") diameter. These wells are usually located at a distance of 20 m from the coastline. The discharge rate is approximately 4,000 $m^3.day^{-1}$ and the seawater is diluted with roughly 10% of fresh aquifer water

g) A survey was carried out on the performance of Beach wells in Malta, Balearic Islands, Canary Island, Greek Islands, and Arabian Gulf Countries. No special problems of Beach well performance was reported, however, two types of problems may face the performance of Beach wells:

    1. Problems of well operation and maintenance in general
    2. Corrosion and other problems resulting from the high ion concentration of seawater

   The risks of such damage can be reduced by following design criteria as defined in the present study.

h) Don Hornburg has recently summarized the design of ‘surface seawater Intake systems’ for desalination plants and the problems encountered with such systems (in press). The problems mainly decrease production rate and efficiency and increase maintenance and downtime. The main problems observed in surface Intakes for SWRO are:

- Trash, shells, and mussels from Intake
- Re-circulation from discharge to Intake
- Shallow flow to Intake point
- Oil observed in Intake
- Intake pipes fouled
- Low level in pump basin
- Shells lodging in tubes
- Sand in cooling water supply
- Deposits in tubes
- Ammonia or sulphides in the cooling water
- Fouled supply pipe
- Lack of chlorine residual

i) Many major problems in operating wells, particularly with high salinity water have been caused by improper or poor well construction. In many cases, well failure may cause a sudden change in the quality of water being pumped into the treatment plant. The most common causes of well failure are:

- Borehole collapse
- Corrosion of casing
- Improper or defective construction techniques
- Growth of organisms within the well bore
- Water intrusion from another source
- Formation of mineral concentrations
- Incrustation in the open hole or screened section of the well bore

The risk of such damage can be minimized by following design criteria, as defined in the present study.

j) Cost estimates prepared for typical conditions in the Israeli coastal plain, show that the cost of desalinated water can be reduced by 17% when a surface Intake is replaced with a Beach well. When considering the indirect environmental costs of Beach wells, the cost reduction is only 9%

k) The study deals with the location and depth of the Intake that will render optimal operation

To avoid adverse effects on the inland freshwater, the best location would be in the seabed as far as possible from the coastline. However, due to the high cost involved in offshore drilling and abstraction, continental sites are considered in more detail. The best of these are sites close to the coastline

The Intake depth is determined by the quantity to be extracted and by the depth required for effective filtration. However, it is also determined by the desire to minimize adverse effects on the freshwater zone. When the aquifer is not stratified, the Intake should be located as deep as possible

l) For site selection of SWRO Beach wells, it is clear that terrigenous, granular aquifers are the most favourable ones. Hence, there seems to be no justification for any groundwater Intake development in karstic, volcanic, igneous and/or metamorphic environments other than in granular aquifers such as sandstone, alluvial deposits and similar. Therefore, a reliable geological map/study or a reconnaissance report is a prerequisite for any site selection and advanced activities

m) The required coastal geology, *i.e.* non-rocky, granular, and preferably loosely packed sediments may be found in some of the Mediterranean and the Arabian Peninsula coastal areas. Within the northern Mediterranean region, *e.g.* Turkey, Greece, Adriatic coasts, Italian and French Riveras', are characterized mainly by steep, rocky coastlines (and tectonically emerging). However, in the south the coastal regions are sandy by nature due to tectonic submerging events

   This is a general outline, however, it should be noted that adequate geological configuration may be encountered; even within the most precipitous coastal environment, in some deltaic deposits, river outlets, closed harbours and short sandy shores. Numerous examples exist in the French Riviera, south east coast of Cyprus, the Balearic Islands, and Spain

n) The main advantage of using a non-surface Intake is the 'natural' filtration of the feed water to SWRO plants in sand formations often existing near the seacoast. However, non-surface Intakes have advantages with coarse granular ground formation with poor filtration. These advantages include: avoiding under-water piping installations, reduction of marine growth effects, temperature stabilization, prevention of sedimentation in inlet piping, flexibility of operating several wells (in comparison to being dependent on one Intake pipe, that might fail)

o) The proposed Design Criteria for Beach well Systems are as follows:
   - To minimize the freshwater fraction in the pumped water, the following design criteria should be followed:
     1. Wells should be located as near as possible to the coast line
     2. Wells should penetrate the full depth of the aquifer and reach its base
     3. The well screen should be as short as possible with a maximum percentage of perforated area
   - The pumping capacity of Beach wells depend exponentially (quadratically) on the thickness of the aquifer. The minimal saturated thickness of the aquifer should not be less than 50 m
   - Phreatic aquifers should be preferred over confined aquifers, which may have their seawater boundary at a large distance from the coastline, therefore, decreasing the rate of seawater flowing to the wells. In a multilayered aquifer, the upper aquifer should be selected
   - Under the hydrologic conditions prevailing in the coastal plain of Israel, the maximum pumping capacity of a Beach well battery running parallel to the coastline may not exceed 15,000 $m^3.d^{-1}.km^{-1}$. The assumed design capacity will be 10,000 $m^3.d^{-1}.km^{-1}$
   - With a single well capacity of 2,400 $m^3.day^{-1}$, the resulting spacing between wells will be 240 m
   - The cost of water transport limits the maximum length of such a battery supplying a single plant to 5-10 km

- When assuming pumping of 10,000 $m^3.day^{-1}.km^{-1}$ and a recovery of 50%, the maximum capacity of the SWRO plant that justifies non-surface Intake is 25,000 - 50,000 $m^3.day^{-1}$, considering the cost of the collecting system

p) Site exploration is required for the assessment of the relevant hydrological characteristics of a particular coastal stretch selected. The first step is by means of collecting and reviewing available data. In the case of no data or insufficient information concerning the local structure, stratigraphy or hydrogeological configuration, the required data have to be generated as follows:

Thickness, stratigraphy, hydrogeological outline and lateral extension of aquiferous units can be obtained by means of exploratory slim holes or exploratory/production wells, and/or a network of observation wells

- Prior to drilling, a preliminary geo-electrical/electromagnetic survey is suggested in order to acquire a rough indication, at least, as for the target lithological sequence
- Later, after drilling within a strip parallel to the seashore, the boreholes will serve, *inter alias*, as calibration points between the lithological and hydro-stratigraphic profile, and the geophysical findings. Suggested geophysical methods are the VES (Vertical Electrical Sounding) and the TDEM (Time Domain Electro-Magnetic) surveys
- The main object of the TDEM survey is, that once the depth and geometry of the seawater/fresh water interface has been established is to adjust and correlate the findings with the outline of inland aquifer extension and thickness of saturated beds above and below the interface
- The hydrological properties of the explored sequence are to be studied by means of hydrological tests in the exploratory/production wells. The main object of the tests is to determine the transmissivity value and the number of required Beach wells and their spread. In case of poor aquifer performance, the feasibility of galleries (pending SWL depth, lithology, and thickness of fresh water saturated beds) will be considered

The above information also serves for setting-up of a simulation model that may be used to predict the quantities of water that can be pumped and to determine optimal location and depth of the Intake.

4. Practical benefits to the desalination community

The present study provides the desalination industry (in its small SWRO sector) an improved design solution for seawater Intakes. This type of Intake will enable the supply of feed-water with improved quality and reliability. Under favourable site conditions (such as granular formations with sufficient hydraulic conductivity but still with adequate filtration capacity), a significant reduction of the Intake cost as well as reduction of pre-treatment cost is expected. Such favourable conditions exist in many of the regional sand and sandstone coastal aquifers in the MENA countries. The potential impact on the total cost of desalination can be significant, particularly for small SWRO plants.

The study products include guidelines for selecting sites and technologies and for assessing site properties. A data processing framework has been developed including spreadsheets for cost estimates based on site properties. These will enable scientifically based and competent decisions on the applicability of non-surface Intakes.

Following the completion of the present study and prior to its implementation in a certain coastal region, additional efforts will be required to assess site properties by the proposed assessment methods.

1. INTRODUCTION AND METHODOLOGY

1.1. Background

Beach wells and galleries extracting groundwater of seawater quality exist in Israel and elsewhere for swimming pools, industrial cooling and coast stability protection. Experience in the application of similar Beach wells and galleries as Intakes for seawater Reverse Osmosis (SWRO) is still limited. However, there are indications that such an Intake type can provide reliable quantity and better quality water than surface Intakes due to natural filtration and underground detention. This is a significant advantage in view of the history of failures caused by the bacterial and organic fouling of SWRO membranes requiring advanced pre-treatment, and of failures caused by adverse marine conditions.

Because of their positive impact on feed-water quality, these Intakes promise an opportunity for better efficiency, reliability, cost effectiveness, and performance of desalination plants in general and SWRO plants in particular, and therefore a significant advantage for the desalination technology; particularly in the MENA (Middle East and North Africa) region.

There are several types of non-surface Intakes. They include: Beach wells, seabed filtration and inflow galleries. These different Intakes represent design variations that utilize the same principle *i.e.* extracting filtered seawater from below the surface near the shoreline. Each of these Intakes has its own advantages, capabilities, suitability, and cost-effectiveness for different sites and required treatment conditions and for different treatment trains.

Relevant site conditions are predominated by the hydrogeological properties. These determine the Intake structure size, available water flows, quality of the feed-water and possible environmental impact. Consequently, these properties determine the sites' total cost effectiveness, which is used for evaluating the feasibility of different surface or non-surface Intake alternatives.

A particular concern is the identification of exploration methods and survey techniques to aid in determining the site properties required for the selection of appropriate sites and Intakes. These include criteria and strategies for conventional and new exploration and survey methods for determining the relevant hydrogeological site conditions.

The present study was initiated by MEDRC to address the above issues. A request for proposals was distributed in 1998. A project funding agreement was signed between MEDRC and TAHAL in July 1999 in which the 'terms of reference' were specified and a period of one year was set. The present report summarizes this study.

1.2. Project objectives

The objective and essential goal of the present study was to provide a comprehensive state of the art document on utilizing Beach wells, and similar non-surface seawater

Intakes for SWRO systems, and to develop and verify the criteria for the choice and design of these types of seawater Intakes.

The study covers different types of non-surface Intakes, their characteristics and possible use in SWRO desalination plants, design methods including simulation, the site requirements and survey methods for designing Beach wells or related seawater Intakes. The study also covers existing facilities and their possible improvements.

1.3. Methodology

The study focuses on identifying the site-specific hydrogeological conditions that determine the choice of Intake types and their feasibility. The main properties are: aquifer lithology, thickness and hydraulic conductivity. Feasibility is expressed in terms of anticipated direct and indirect costs, merits of the proposed technology in the desalination process and anticipated problems in its implementations.

The main concern for comparing different water Intakes is the resulting direct feed-water cost at the inlet of the SWRO plant (after pre-treatment), and the reduction of treatment cost due to the improved feed-water quality.

The second concern is the indirect cost representing the environmental impact of pumping seawater on the groundwater system, *i.e.* its impact on the sustainability of fresh groundwater for existing and planned pumping wells.

1.4. Direct cost factors

The direct -cost of feed-water is determined by site-specific physical cost factors. The main cost determining elements are:

a) Discharge capacity of a single well or of another Intake type
b) Spacing required between wells to avoid interference and excessive lowering of the water table
c) Depth and size of drilling or digging of the subsurface Intake structures
d) Lithological properties that determine the unit cost of drilling or digging
e) Probability of failure
f) Length, size, materials for screens, casing pipes or other linings
g) Size, type of pumps and motors required for lifting the water
h) Length, size, and material of pipelines or other conduits needed for raw water conveyance from the groundwater source to the desalination plant
i) Energy required for water lifting and conveyance
j) Expected life duration of the installation equipment with particular concern for corrosion, which is high in seawater and may be higher in saline groundwater Intakes
k) Savings in feed water pre-treatment, such as natural removal of suspended solids, organics, *etc*

The size and costs of the above elements depend both on site properties of the subsoil, and of the seawater. These are detailed below.

1.5. Indirect costs and environmental impacts

The major environmental impact to be considered with respect to SWRO desalination plants with well Intakes is on the inland fresh groundwater systems. The two main impacts on the inland groundwater system that are expected to be faced if pumping is maintained, inducing flows from the fresh groundwater body:

1) lowering groundwater tables within the usable inland fresh aquifer sections
2) loss of fresh groundwater by induced flow into seawater sections of the aquifer

The 1) and 2) are governed by the distance of the abstraction site from the coast line, the radius of influence (depending on the aquifer's Transmissivity and Storativity), the discharge rate, Intake depth, and the size and capacities of the inland aquifer.

Quantification of these indirect costs is by means of estimating losses of water to the inland water supply system and charging desalination costs with the production of additional desalinated water to cover the deficit. However, these indirect costs are nil if inland water is not used.

1.6. Relevant site properties

The cost of the elements described above depends on the following measurable site properties of the subsurface and of the water:

a) Stratigraphic layout in the selected site
b) Filtration capacity of the subsurface water formations
c) Porosity, storativity and hydraulic conductivity of the subsurface aquifer
d) Thickness of the aquifer
e) Land topography
f) Quality (suspended solids, corrosively *etc*.) of the feed water

The study addresses the site properties and shows the methods for their assessment.

1.7. Cost assessment

The study shows the methods for compiling and assessing the values of the direct cost factors outlined in section 1.4 from the site properties outlined in section 1.6. In conclusion, the economic evaluation of product-water cost is shown based on the values of these cost-determining elements. A special spreadsheet system was developed for compiling the cost estimates.

1.8. Preliminary considerations on site options

The study deals with the location and depth of the Intake that will render optimal operation.

To avoid adverse effects on the inland freshwater, the best location would be in the seabed as far as possible from the coastline. However, due to the high cost involved in offshore drilling and abstraction, continental sites are considered in more detail. However, the best of these sites are close to the coastline.

The Intake depth is determined by the quantity to be abstracted and by the depth required for effective filtration. However, it is also determined by the desire to minimize adverse effects on the freshwater zone. When the aquifer is not stratified, the Intake should be located as deep as possible.

A special dual fluid simulation model is required to study the geohydrological considerations and to set the optimal location and depth of the Intake. The existence and accessibility of such a model was studied within the present study.

1.9. Methods and survey techniques

The choice of appropriate sites and Intakes requires the assessment of site properties. Methods and survey techniques to be applied in this assessment were evaluated in the present study. The purpose of these methods and techniques was to establish and determine quantitatively the specific values of the relevant site properties outlined above. The following methods and techniques were considered:

a) Geological mapping
b) Geological cross sections
c) Well logs including oil wells in general and off shore wells in particular
d) Well pumping tests including interference tests
e) Sampling and physico-chemical analyses of water quality in wells
f) Conventional geophysical surveys
g) TDEM methods to estimate depth of seawater intrusion (interface configuration)
h) Regional groundwater balances, including Mathematical Flow Models
i) Remote sensing methods and GIS

1.10. Objective of data collection

The present study is composed of data collection and evaluation. Data were collected to evaluate the mode in which site properties affect the feasibility and selection of Intake technology and the real range of these values in coastal areas.

1.11. Data sources

Data for the present study were collected in the following manner:

a) Literature survey
b) Direct contact with data sources
c) Formulation of questionnaires and analysis of the responses received

The following sources were identified and approached for collecting the relevant data:

a) Literature survey of existing library documentation and internet search

b) Senior hydrogeologists having experience in Beach wells and coastal hydrogeology
c) Beach well operators in the Mediterranean, Red Sea and other coasts
d) Desalination process experts and plant operators
e) Israeli and other groundwater modellers having experience in the hydraulics of the two phase flow in coastal aquifers and interface configuration

The data collected from these sources was detailed in Section 2.3.

1.12. Follow-up

Follow-up activities were proposed at the conclusion of the study. The follow-up activities included: in depth advanced studies at specific favourable sites, design of pilot plants at such sites, and development or upgrading of site exploration methods that had been found relevant for the assessment of the site properties. Follow-up recommendations also included monitoring in existing non-surface seawater Intakes such as the performance of pumping Beach wells and their interference with inland aquifers.

2. WORK PERFORMED

2.1. Scope of work

The study covered the following tasks:

a) Identification and description of the various types of non-surface seawater Intake methods
b) An evaluation of the literature survey: collection of existing experience reports, case studies, records of failures and successes for both conventional SWRO with surface Intakes and Beach wells for SWRO and other purposes
c) Direct communication with experts using questionnaires to collect experience reports, case studies, records of failures and successes
d) Description of the characteristics of the various non-surface Intake technologies with respect to raw feed-water flow, filtration, capacity, effect of soil/ground properties, effect on water composition, lifetime and maintenance, *etc*
e) Identification of the seawater desalination processes and site conditions for which this type of seawater Intake is viable and cost effective
f) Description and evaluation of simulation and other design methods
g) Development of guidelines and criteria for site-specific selection of Intake type
h) Description of existing and future site survey approaches and exploration techniques, which may be applicable in site and process selection
i) Assessment of the costs of Beach well Intake systems and comparison with surface Intake alternatives. Identification of site conditions (coastline geology, beach soil and subsurface stratigraphy, seawater quality, *etc*.) for which these Intakes are the most economical solution
j) Examples of sites in the Middle East and North Africa regions where Beach wells or similar non-surface Intakes could be viable and cost effective

2.2. Main products

The applicable products of the study are:

a) Guidelines for selecting an appropriate Intake type, given the size of plant and site conditions
b) A simple general-purpose spreadsheet that summarizes the underlying cost assessment procedure according to local conditions
c) Guidelines for the assessment of site properties that are determinant in the cost and feasibility assessment

2.3. Data collected

The following data were collected:

a) Description of the characteristics and design principles of the various non-surface Intake technologies
b) Performance of existing Intakes used for small SWRO including the existing Beach wells
c) Performance of existing Beach wells and other non-surface Intakes used for other purposes (not SWRO)
d) Models for assessing the discharge of Intakes depending on aquifer properties and on the Intake design
e) Models for assessing the hydrogeological impact of abstraction on the near fresh water wells
f) Unit prices of works, materials and equipment for the different types of Intakes

2.4. Schedule

The duration of project activities was scheduled for twelve months. The project tasks were concluded with periodic progress reports and a final report. The following schedule outlines the reports and the tasks that were accomplished with each report.

First periodic report (four months after project start date):
Summary of the literature survey and data collected from other sources, description of the characteristics of the various non-surface Intake technologies, identification of the seawater desalination processes for which this type of seawater Intakes may be viable and cost effective.

Second periodic report (eight months after project starting date):
Description and evaluation of simulation and design methods, site selection and process selection criteria, and site selection survey approaches and technologies.

Draft final report & executive summary (eleven months after project starting date):
Preliminary assessments of the costs of Beach well Intake systems in representative sites and comparison with conventional surface Intakes, Spreadsheets for data collection and assessments, Identification of site conditions and seawater desalination processes for which these Intakes are the most economical solution, Identification of sites in the Middle East and North Africa regions where Beach wells or similar non-surface Intakes could be viable and cost effective, General guidelines for site and Intake type selection and follow-up recommendations. This report also includes a revised summary of the first reports.

Final report (twelve months after project starting date):
Revised draft final report incorporating the Project Advisory Committee suggestions.

2.5. First period activities

The main activities in the first period of work (1st August 1999 – 30th November 1999) were focused on data collection and identification of the main issues of the study.

The types of non-surface seawater Intake methods were identified and their characteristics studied. A literature survey was conducted to identify the leading issues. Field data on the performance of Beach wells in Israel were collected and analyzed. A questionnaire was prepared and mailed to approximately 100 producers and planners of RO Plants. Responses were collected and summarized. Data on planned SWRO in Gaza was collected as well as experience in Arab Countries and in Greece. Minimal data on Beach wells was found in these surveys. Models of the dynamics of seawater intrusion into coastal aquifers were also surveyed and their possible application to Beach wells was preliminarily analyzed.

In conclusion of this phase, a complete overview of the performance of Beach wells in Israel was prepared which seemed to be very promising. However, the information on other countries was insufficient.

The main conclusion at the first stage of work was that under favourable hydrogeological conditions *e.g.* Israeli coastal aquifer Beach wells are preferable compared to open Intakes for small SWRO plants. For large plants, the distribution and large number of wells may render this alternative inferior in terms of direct and indirect costs.

2.6. Second period activities

The main activities in the second period of work (1st December 1999 – 30th March 2000) were focused on description and evaluation of simulation and design methods, and developing site and process selection criteria as well as site selection survey approaches and technologies.

Some tasks of the first period were continued. More experience in Beach wells was collected in Israel, Greece, and in some Mediterranean and Canary Islands. A new summary on desalination in Arab and Gulf countries was prepared and some more responses were received to questionnaires distributed in the first period. However, the information available on the important experience in Arab and Gulf countries was found to be outdated and incomplete.

Experience in the use of coastal drains/galleries was collected to establish parameters of this Intake type. One of the main efforts was the application of a groundwater flow model by which main design parameters like salinity of the pumped water and the undesirable impact of Beach wells on the fresh groundwater aquifer were established. These design parameters were dependent on aquifer properties such as: depth, hydraulic conductivity, dispersion, storage coefficient and on design parameters such as: location of Intake (distance from the sea, depth of well screen) and total discharge rate.

Another main effort in this stage was the development of a model for cost estimates of desalination plant components and Intakes (Appendix C). This model was proposed to be used for the selection of the optimal Intake method and system design by comparing the total lifetime costs; direct and indirect, and of alternative methods and designs. The model includes all unit costs and design parameters of hydrology, finance and desalination technology, *etc*. Unit costs have been collected and inserted into the model.

2.7. Third period activities

In the third and last period of work (1st April 2000 – 31st August 2000), the following main activities were completed:

- Revision of the cost estimate model and running comparisons
- Assessments of costs of Beach well Intake systems in representative sites and comparison with conventional surface Intakes
- Description of existing and future site survey approaches and exploration techniques which may be applicable in site and process selection
- Preparing general guidelines for site and Intake type selection
- Revision and completion of the subjects presented in the first two progress reports
- Preparing the Final Draft Report

2.8. Report

The Final Draft Report is a summary of all the activities carried out and includes revisions and updates of the first two periodic reports and new sections related to the third period activities. The present report is a revision of the final draft report submitted in August 2000. The revisions are partly in response to PAC comments to the draft report received in December 2000.

3. EXISTING EXPERIENCE

3.1. Existing experience - A Literature Survey

The literature survey and the information collected from questionnaires and direct contacts enabled the assessment of existing experience in the following subjects of the study:

1. Desalination process
2. Existing SWRO facilities
3. Performance of Beach wells
4. Characterization of desalination plants with Beach well Intakes
5. Characteristics of other non-surface Intakes
6. Surface Intakes and their disadvantages

The following sections summarize these subjects focusing on approaches and results, which are relevant to the present study.

3.2. SWRO process and plant description

The Reverse Osmosis process is based on the natural phenomenon that every saline solution has an ‘osmotic pressure’, which is proportional to its concentration and on the existence of a semi-permeable membrane, which separates two saline solutions with different concentrations and osmotic pressures. The differential osmotic pressures create a force that drives the dilute solution through the membrane to the concentrated solution on the other side of the membrane. In the RO process, the natural osmosis is reversed by way of a high pressure generated by a pump with a higher pressure than the seawater, or brackish-water, *i.e.* osmotic pressure differential. The process is dominated by the quality and characteristics of the semi-permeable membrane. Two parameters define membrane performance:

1. ‘flux’, *i.e.* solvent flow per unit membrane area, which affects the total required plant membrane area and capital costs
2. ‘salt rejection’, *i.e.* the fraction of solute concentration rejected by the membrane

The two characteristics are contradictory: A ‘loose’ membrane will allow higher fluxes, but will have a lower rejection and a ‘tight’ membrane *vice-versa*. High operating pressure will increase flux, but will also increase pumping power and membrane compaction and lead to faster degradation with time of membrane performance. Seawater desalination plants operate at pressures of 60-70 bar and a salt rejection of 99.8%. Plant ‘recovery ratio’, *i.e.* ratio of product water to feed water is also an important parameter. SWRO recovery ratios are 45-55%. A high recovery ratio reduces pumping power requirements, increases average solution concentration, and product salinity.

A typical SWRO plant is given in Figure 3.1. It mainly composed of five sections:

1. seawater Intake
2. pre-treatment
3. desalination section, (contains high-pressure pumps and membranes)

4. post treatment
5. brine disposal section

The raw water, *i.e.* feed seawater, is pumped from the sea into an operation raw water tank (where required) and then again pumped to the pre-treatment section which pressurizes it through the dual-media filter that blocks the relatively large objects and particles. The seawater is then conditioned with acid that acts as an anti-scalant, and with chlorine-based chemicals for disinfecting the feed water, and with additional chemicals as required according to the raw water analysis. After the chemical treatment, the water flows to the check filters (cartridge filters) that trap small particles above 5 microns. The seawater is pumped through the high-pressure pump to the RO and energy recovery turbine unit *i.e.* the membranes section. The fresh water passes to the permeate side of the membranes and leaves through a permeate operation water tank (if required) to the post-treatment section. Post treatment consists of a decarbonation and chlorine dosing. The product water acidity, pH, is adjusted by the decarbonator and chlorine is dosed as a disinfectant. The concentrated seawater (brine) leaving the membrane section at high pressure operates the recovery energy turbine; and afterwards, it is disposed of into the sea. Permeate (the product water) is pumped and supplied to the local drinking water system.

3.3. Status of desalination and SWRO in the world

The status of desalination in general and SWRO in particular was recently summarized by Wangnick Inventory Report (2000), the data given in this report can be summarized as follows:

- The cumulative capacity of all operational and contracted desalting plants in the world at the end of 1999 was 26,000,000 $m^3.day^{-1}$. (The capacity for 1997 was 23,000,000 $m^3.day^{-1}$
- The annual growth of world desalination capacity in the last 5 years, based on all types of desalting processes, is 4.6%. This is a mean figure that may change from year to year
- The cumulative capacity of all seawater desalting plants in the world, based on all types of desalting processes at the end of 1999, was 15,000,000 $m^3.day^{-1}$, *i.e.* 72% of the total desalination capacity. Of this, the total capacity of SWRO plants at the end of 1999 was 2,300,000 $m^3.day^{-1}$, *i.e.* 11.5% of all seawater desalination plants
- The cumulative capacity of all RO plants in the world, treating all kinds of water, (seawater, brackish water, pure water, surface (river) water), at the end of 1999, was 11,000,000 $m^3.day^{-1}$
- The cumulative capacity of all SWRO plants in the world at the end of 1999 was 2,300,000 $m^3.day^{-1}$, *i.e.* approximately 21% of all RO plants
- The total number of all RO plants in the world at the end of 1999 was 2506, (at the end of 1997 – the total number of all RO plants was 2123)
- The total number of SWRO plants in the world at the end of 1999 was 401, *i.e.* 16% of all RO plants (at the end of 1997 – the total number of SWRO plants was 340)

- Table 3.1 presents the countries which had at the end of 1999, 4 SWRO plants or more compared to the figures at the end of 1997.

Table 3.1: SWRO Plants by Countries

Countries having 4 SWRO plants, and more	**Total no. of SWRO plants**		**No. of SWRO plants desalting 10,000 cum.day^{-1}, and more**	
	1997	**1999**	**1997**	**1999**
Spain	76	64	20	21
Saudi Arabia	40	47	6	7
Egypt	23	29		
U.S.A	22	29	4	6
United Arab Emirates	16	20		
Japan	17	17	5	5
India	8	16		1
Cayman Island	10	12		
Antilles NL	8	12		1
Indonesia	9	11		1
Mexico	7	10	1	1
Italy	9	9	1	1
United Kingdom	7	9	1	1
Libya	8	8	3	3
Malta	8	8	5	5
Bahamas	7	7	1	1
Virgin Island U.K.	5	5		
Oman	4	4		
Qatar	4	4		
Venezuela	4	4		
Bahrain	2	3	1	1
Cyprus	2	3	2	3
Gibraltar	2	2	1	1
Total	**289 (*)**	**334**	**51**	**59**

(*) The total number of SWRO plants is 401 plants for the end of 1999. The rest of the countries have less than 4 SWRO plants each.

Note: The cumulative capacities and numbers of plants are related to operational and contract plants; but it is not certain that all plants are in continuous operation

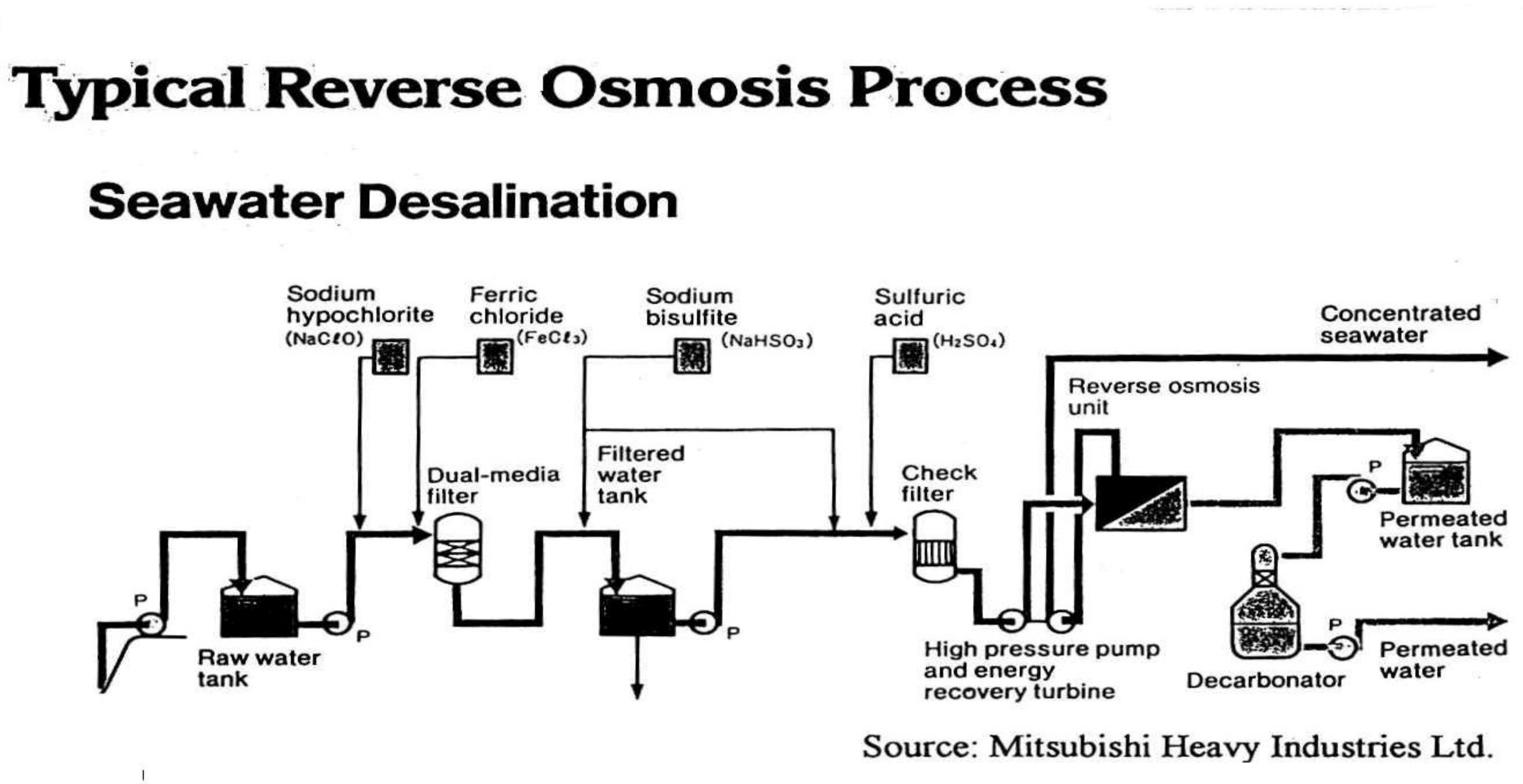


Chart 3.1: Typical Reverse Osmosis process

3.4. Beach wells in Israel

3.4.1. Beach wells in the Israel coastal aquifer

Beach wells using water for cooling installations and for swimming pools were installed in Israel in the early 1960's near the Mediterranean coast. The general characteristics of the coastal aquifer are summarized in Appendix F. Data on these Beach wells were collected from the Water Commission, Hydrological Service, TAHAL's archive, site visits and interviews with operators.

The typical dimensions of such wells are: 90 m total depth with a 20 m screen with a diameter of 25.4 cm (10"). These wells are usually located at a distance of 20 m from the coastline. The discharge rate is approximately 4,000 $m^3.day^{-1}$ and seawater is diluted with about 10% of fresh aquifer water. Detailed data collected for these wells are shown in the following tables and charts. The first well: Hasharon is in Herzeliya north of Tel Aviv. Hilton and Gordon are in Tel Aviv and Ashkelon is 50 km south of Tel Aviv. General information on these wells is given in Table 3.2. Chemical data were collected from records of the wells (GW) pumping seawater for hotels in the Tel-Aviv area. Samples taken directly from the nearby sea (SW) were analyzed. The results of the chemical analyses are shown in Table 3.3. Some data on the isotopic composition of these samples are shown in Table 3.4.

The variation of salinity over time in some of these wells is shown in Chart 3.2 – 3.6. It can be seen that variations in salinity may be very high, reflecting upcoming of seawater (see Appendix B) and variable volumes of fresh water flowing from the inland aquifer. Salinity of seawater is estimated at 22,500 Cl- (Table 3.3). The salinity in Beach wells compared to seawater (Figures 3.2 – 3.6), shows that the fraction of seawater pumped is about 90% and fresh water pulled from the inland aquifer constitutes a fraction of not less than 10%. The model studies for representative conditions of the Israel coastal aquifer as presented in section 6.1 show similar results.

3.4.2. Beach wells in Israel Red Sea – Eilat

An experimental seawater desalination plant was operated in Eilat near the Red Sea between 1963 to 1965. The desalination process was Zarchin's freezing process and the Intake of this plant was by Beach wells. Some data on these wells is summarized in Table 3.5 and Table 3.6.

Data on the characteristics of the aquifer were collected from two exploratory wells, which were tested and analyzed by Greitzer (1972). The bedrock in the Eilat area is ancient Granites and Schists. The (Syrian-African) rift in the beach area is filled with sands, sandstones, pebbles, conglomerates, boulders, marls, and silts to a depth exceeding 100 m. The capacities of the wells: 120 - 150 $m^3.hr^{-1}$ and specific capacities about 8 $m^3.h^{-1}.m^{-1}$.

The quality of the pumped water indicates that the fraction of groundwater pumped is above 10% (assuming that chlorides in the local saline GW are about 1,000 $mg.l^{-1}$). However, the calcium content in the Beach wells is much higher than in seawater. This may be a result of dissolution from the porous limestone media en route.

Table 3.2: Beach wells in Israel coastal aquifer - general data

Well Name	ID No.	Capacity $m^3.day^{-1}$	Distance from Coast Line (m)	Total Depth (m)	Bottom Screen		Depth to W. L. (m)	Specific Capacity $m^3.hr^{-1}.m^{-1}$	Salinity in chlorides ppm
					Length (m)	Diameter (cm)			
Hasharon	176 131 02	2,500	10	58	7	26.67 (10.5")	1.8	36	200 - 18,650
Hilton N.	166 128 03	4,000	15	94	21	25.4 (10")	14.7	-	21,274
Hilton S.	166 128 02	4,000	20	91	20		12.5		20,423
Gordon	165 128 04	5,000							18,000
Ashkelon	119 107 03			31	4	25.4 (10")	12.5	80	

Table 3.3: Chemical composition of Mediterranean seawater and saline ground water in the Tel-Aviv area, ($mg.l^{-1}$)

Location	Type	I.D.	Date	Ca	Mg	Na	K	Cl	SO_4	HCO_3	B
Hilton N.	GW	16612803	20/9/92	463	1239	10600	392	19812	2600	232	5.0
Hilton N.	GW	16612803	23/2/94	444	1065	11665	400	21057	2450	286	
Hilton N.	GW	16612803	19/3/95	388	1135	9650	345	17500	1950	211	
Hilton S.	GW	16612802	20/9/92	420	1049	9500	350	16980	2200	247	4.4
Hilton S.	GW	16612802	23/2/94	423	1012	10000	345	17144	2350	240	
Hilton S.	GW	16612802	19/3/95	510	1204	10200	345	17780	2350	259	
Hilton E.	GW	16612805	20/9/92	518	1184	10250	365	20387	2600	247	4.5
Hilton E.	GW	16612805	23/2/94	438	945	9615	315	16387	2250	264	
Hilton E.	GW	16612805	19/3/95	504	1200	10250	325	19100	2150	259	
Plaza	GW	16512803	20/9/92	498	885	8350	278	14967	1950	290	3.6
Plaza	GW	16512803	23/2/94	447	869	8460	280	15035	1700	296	
Gordon	GW	16612804	20/9/92	468	1058	9650	327	17026	2300	260	4.2
Sheraton	SW		27/2/93	695	1509	12050	455	22170	2350	185	
Dolphin	SW		27/2/93	688	1466	11925	450	22150	2450	198	
SW1	SW		8/2/94					22250			
SW4	SW		19/7/94			12100	415	23847	2950		
SW8	SW		6/4/94	459	1211	12500	435	21939	2700	169	
SW11	SW		12/2/95	504	1447	12250	425	22500	2350	174	
SW12	SW		12/2/95	499	1452	12250	425	22600	2000	183	
SW15	SW		14/8/95					22500			
SW16	SW		17/9/95					22500			

*GW - Ground water *SW - Seawater

Table 3.4: Isotopic composition of Mediterranean seawater and saline ground water in the Tel-Aviv area, ($mg.l^{-1}$)

Location	Type	Date	Tritium (T.U.)	δ18O (%0)	δ13C (%0)	14C (PMC)
Hilton N.	GW	20/9/92	3.0±0.3	0.18	-3.7	61.0+0.4
Hilton N.	GW	23/2/94	0.7±0.2		-5.7	84.9+0.8
Hilton N.	GW	19/3/95			-4.8	51.5+0.5
Hilton S.	GW	20/9/92				
Hilton S.	GW	23/2/94	5.0±0.3	-0.80		
Hilton S.	GW	19/3/95	1.9±0.5		-4.8	45.3+0.5
Hilton E.	GW	20/9/92	2.8±0.3	-0.10	-5.2	49.0+0.4
Hilton E.	GW	23/2/94	5.2±0.5		-8.9	46.2+0.2
Hilton E.	GW	19/3/95			-4.8	45.1+0.4
Plaza	GW	20/9/92	1.4±0.1	-1.28	-4.0	46.0+0.4
Plaza	GW	23/2/94	0.9±0.2		-2.5	43.9+0.5
Gordon	GW	20/9/92	.8±0.3	0.89		
Sheraton	SW	27/2/93	0.3±0.3	1.76	-2.5	117.5+0.9
Dolphin	SW	27/2/93	5.9±0.3	1.89	-2.1	115.2+0.6
SW1	SW	8/2/94	1.6±0.2			
SW4	SW	19/7/94		1.81		
SW8	SW	6/4/94	0.6±0.2			
SW11	SW	12/2/95	4.0±0.3			
SW12	SW	12/2/95	1.5±0.2			
SW15	SW	14/8/95			0.0	102.7
SW16	SW	17/9/95			0.0	101.6

Table 3.5: Beach wells in Eilat - general data

Well Name	Coordinates	Capacity $m^3.day^{-1}$	Distance from Coastline (m)	Total Depth	Bottom Screen		Depth to Water table (m)	Specific Capacity $m^3.h^{-1}.m^{-1}$	Salinity Chlorides ppm	Transmissivity $m^2.d^{-1}$
					Length (m)	Diameter (cm)				
Zarchin 1	144.295/883.559	3456	80	34.66			6.8	8	20,525	
Zarchin 2	144.309/883.572	2880	80	30.95			6.6	6.8		
Zarchin 3	144.479/883.627	10.560	25	100	20	40.64 (16”)	4	28.2		540-4,400
					50	31.75 (12.5”)				
Zarchin 4	144.43/883.62	4080	30	70	20	45.72 (18”)	4	4.7		120-2,400
					30	31.75 (12.5”)				

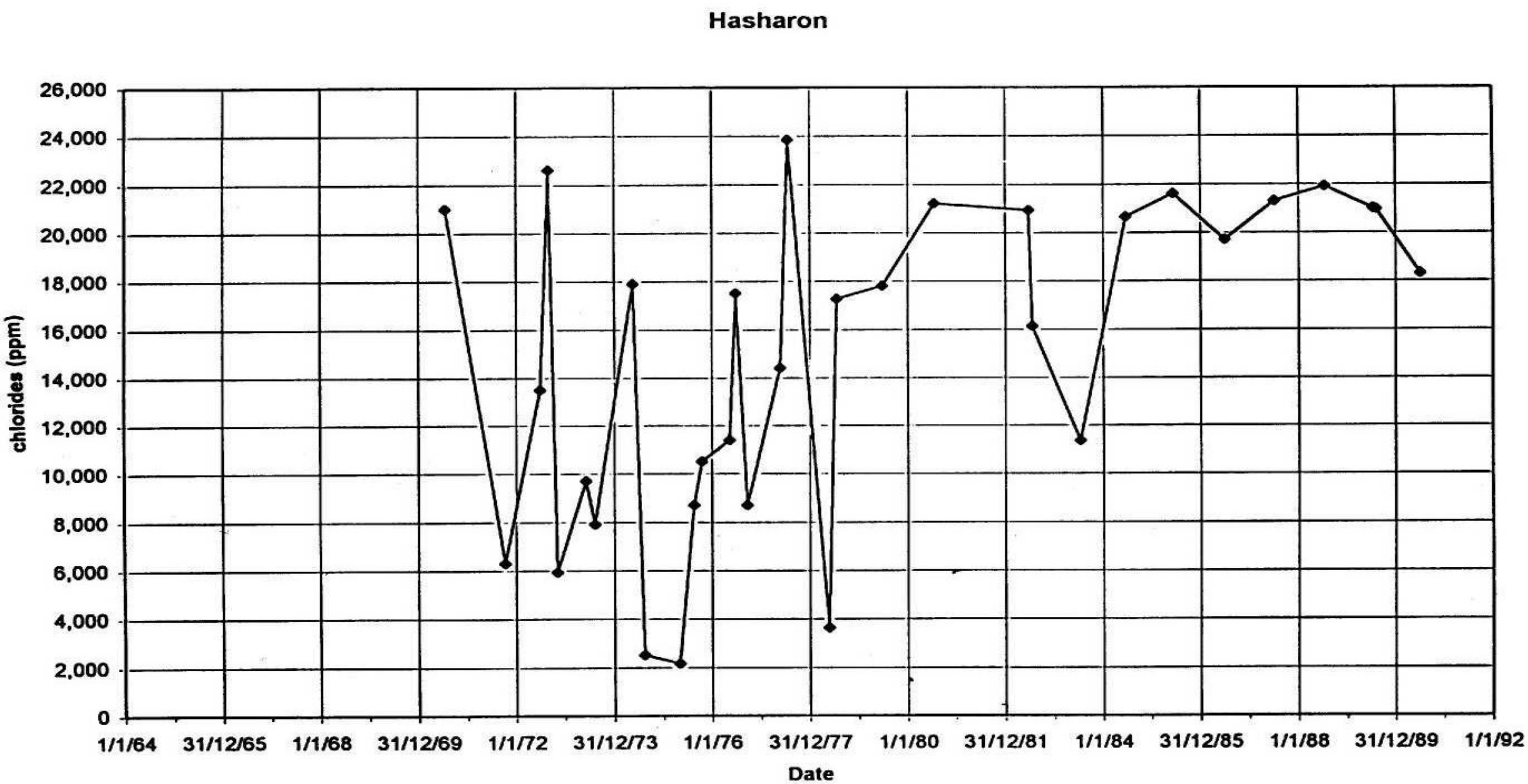


Chart 3.2: Salinity variations in Hasharon well

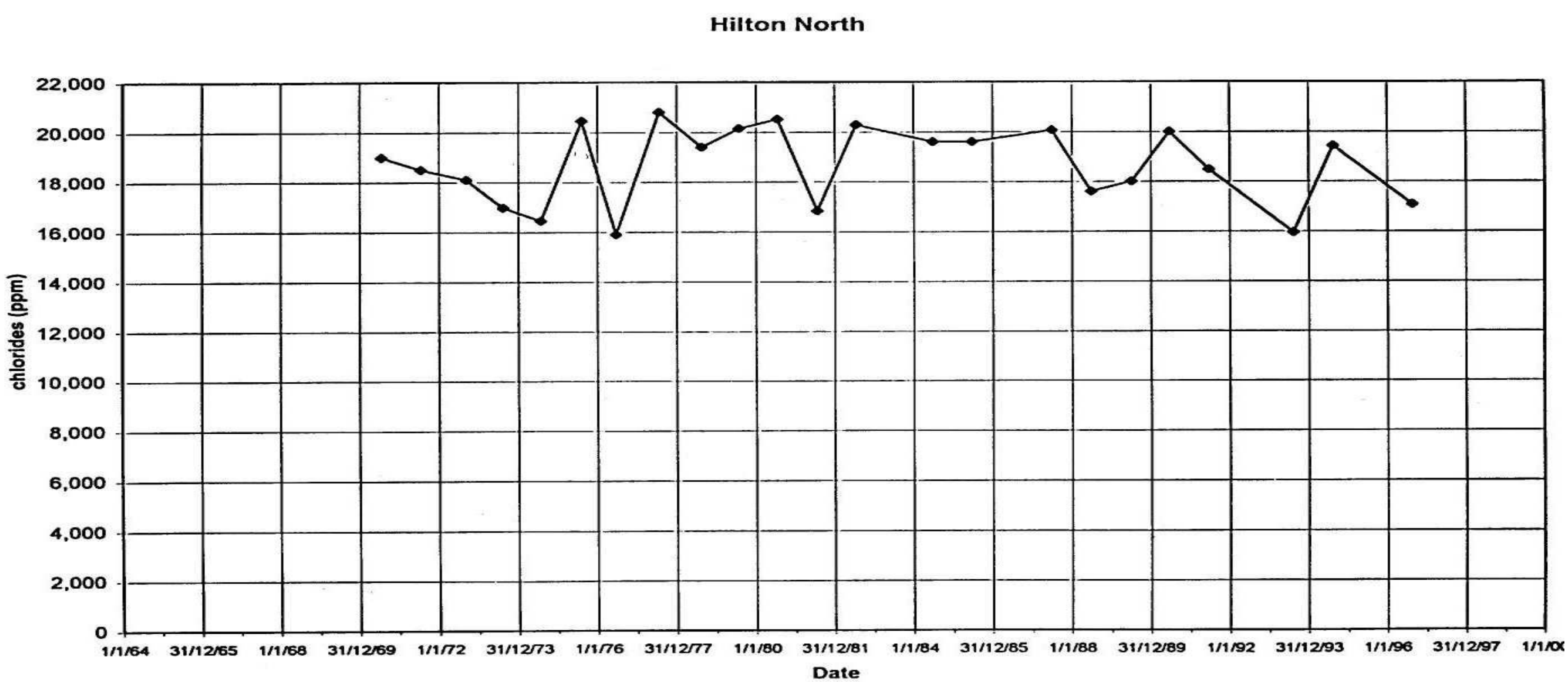


Chart 3.3: Salinity variations in Hilton North well

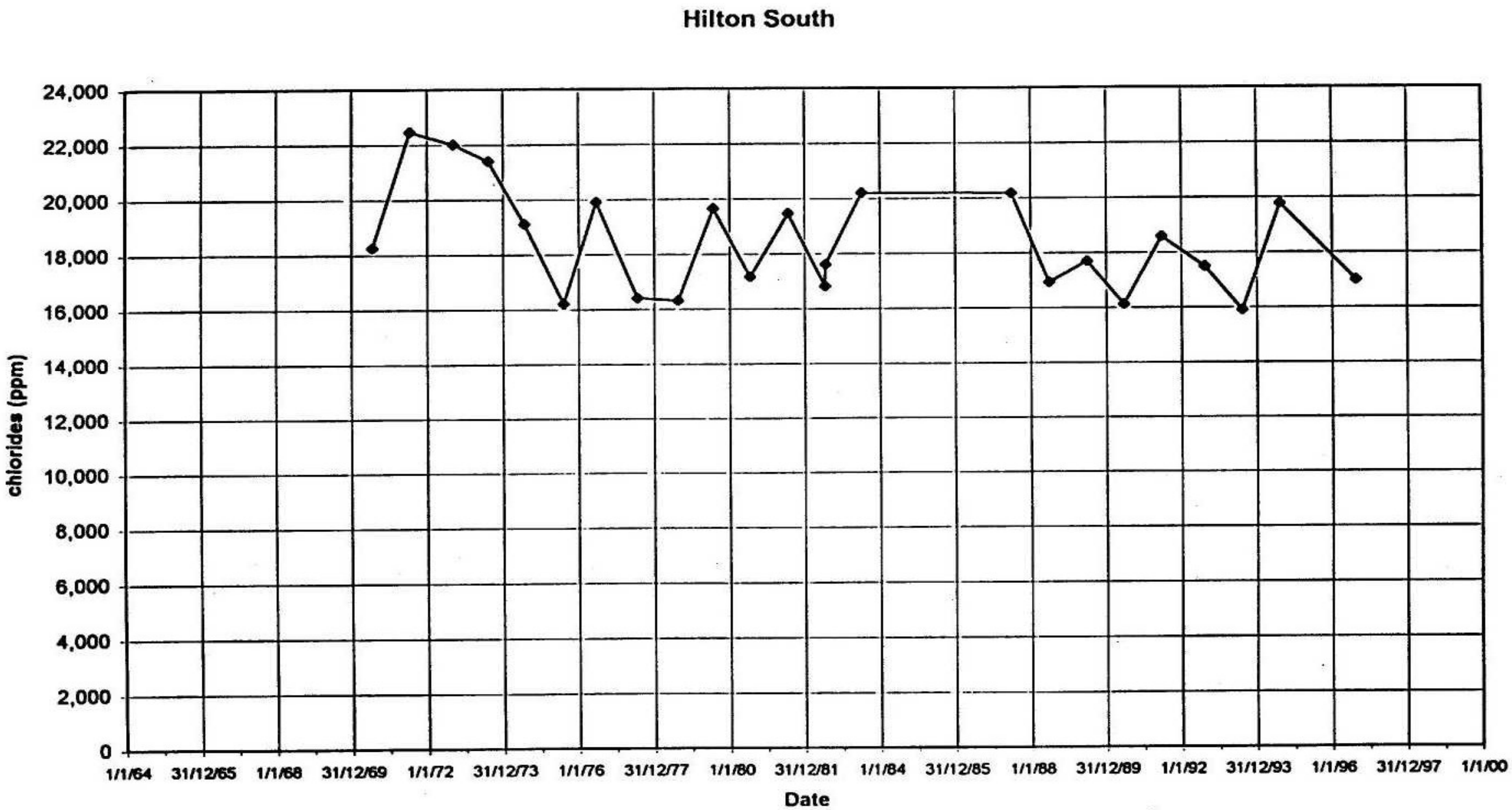


Chart 3.4: Salinity variations in Hilton South well

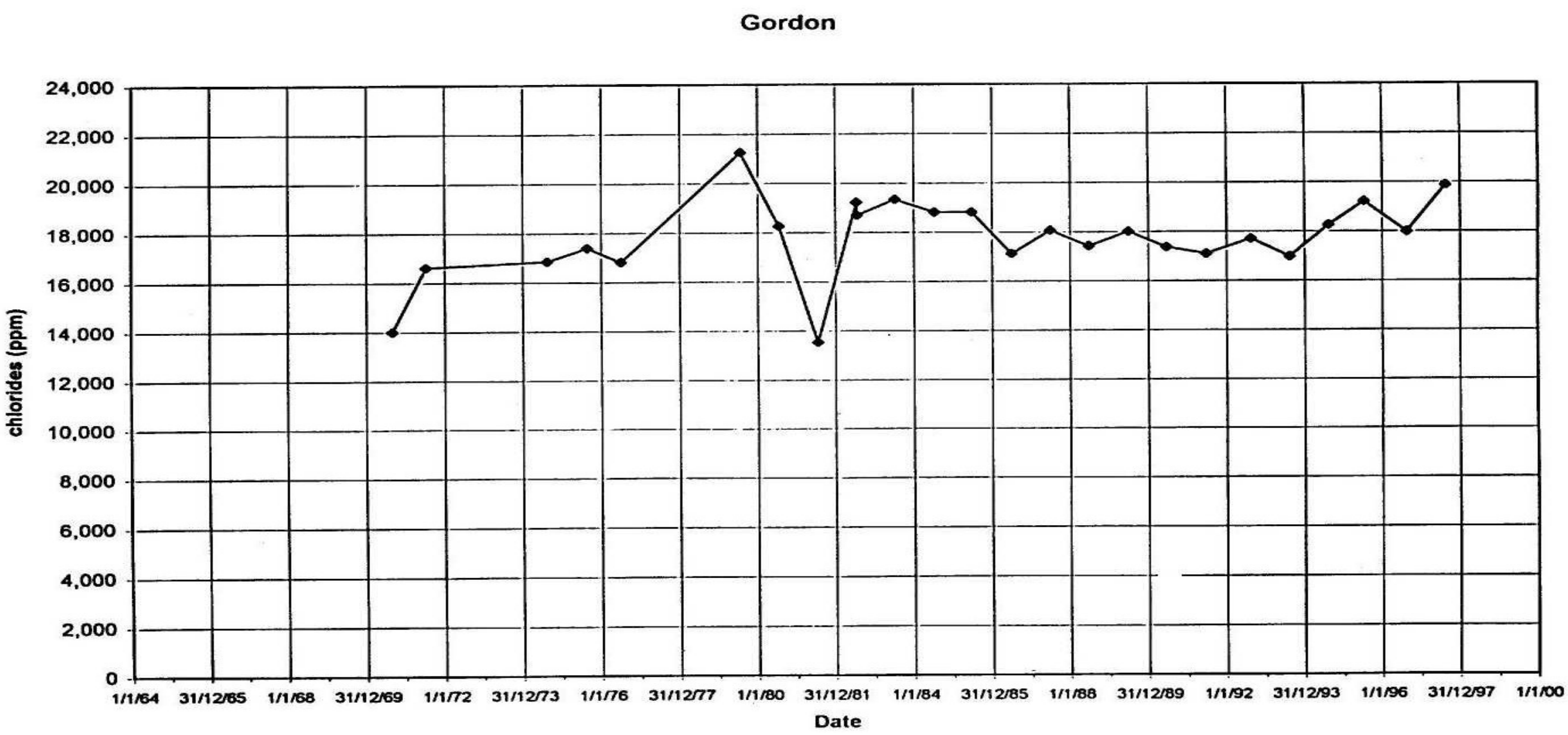


Chart 3.5: Salinity variation in Gordon well

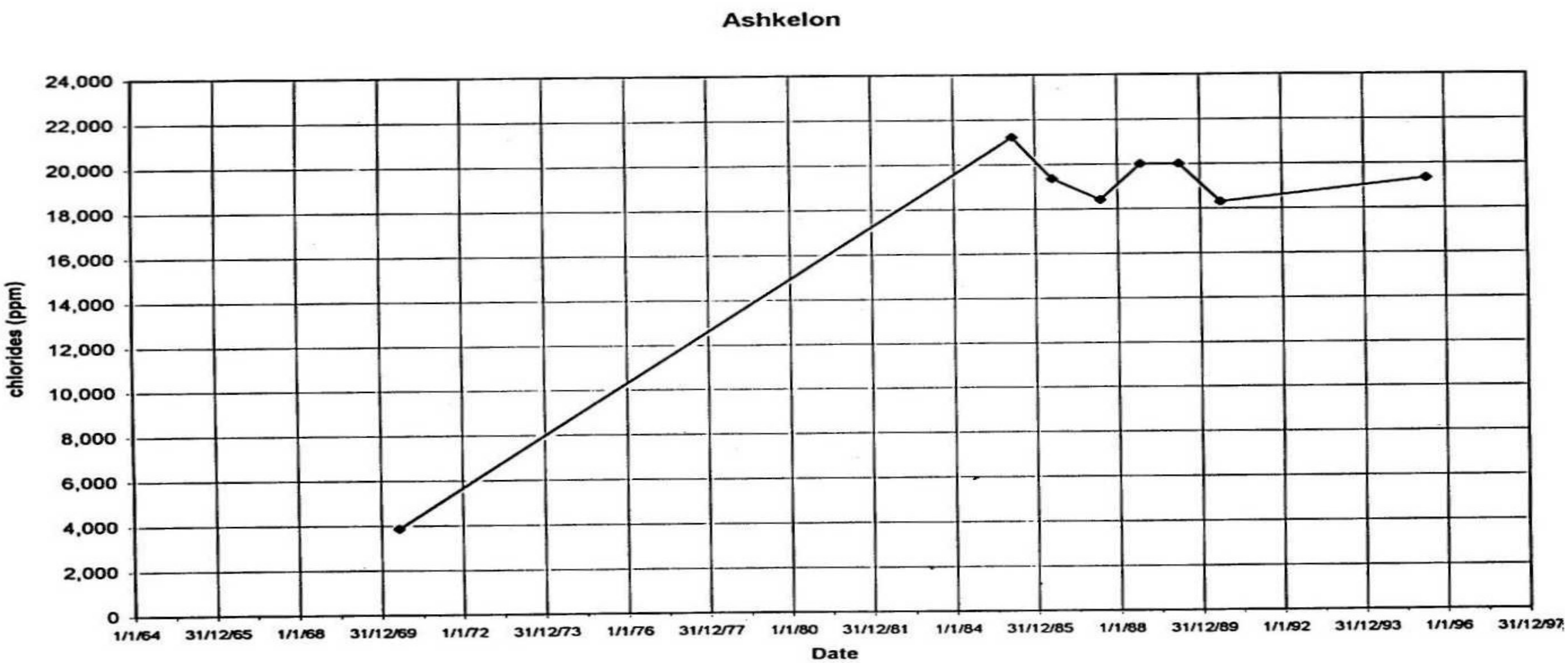


Chart 3.6: Salinity variations in Ashkelton well

Table 3.6: Chemical composition of Red Seawater and pumped saline groundwater

	Cl	SO_4	HCO_3	Ca	Mg	K
Zarchin 1+2	20,525	2,633	103	1,240	1,136	416
Seawater	22,798	3,111	152	480	1,494	516

3.5. Desalination plants with Beach well Intakes

Data on existing SWRO plants using Beach well Intakes were received from Malta, Canary Islands, Ibiza and Greece. The available data of these plants are summarized in the following sections.

3.6. Malta

3.6.1. Scope and capacities

Detailed data on desalination in this island were available for 1991. No reliable up to date documentation was available. Thermal and RO desalination plants supply approximately 65% of the total water needs of the permanent population of approximately 350,000 and for tourism. The current capacity of RO installations is 100,000 $m^3.d^{-1}$. In 1991, the total desalination capacity was estimated to be 64,000 $m^3.d^{-1}$ (Maoz, 1991). Most of the Intakes in 1991 were Beach wells (see photograph), which were operated by the water supply authority while the desalination plants were operated by contractors (Polymetrics Inc.). The first large plant (20,000 $m^3.d^{-1}$) was built at Ghan Lapski with a vertical well Intake and operation started in 1983. Andrews (1986) reported on the performance of the plant in the first 2 years.

3.6.2. Plant design

The components of this plant are:

- Vertical well Intake system (15 wells)
- Feed break tank and boost pumps
- Feed integrated turbo boost pumps
- Acid (reduced consumption of 6.6 ppm in feed water)
- Cartridge filters (5)
- Turbo pumps (10)
- Single stage 10 independent RO trains each 2,000 $m^3.d^{-1}$ capacity
- Post treatment (caustic and chlorine)
- Product storage tank
- Flushing system of the high salinity water from the RO system
- Brine disposal
- Advanced control system (SCADA)

3.6.3. System performance

The performance of the system is summarized as follows:

- The entire facility consumes less than 6.12 $KWh.m^{-3}$ for water delivered to the storage tank
- The entire plant is controlled and adjusted by one operator
- Feed water with a TDS 10% less than design (apparently seawater mixed with groundwater)
- SDI of feed water is 1.1 - 2.7
- High sand loading from the limestone formation resulted in the use of approximately double the design quantity of cartridge filter elements. A study was contemplated on settling out the majority of the sand in the feed break tank
- A higher than expected corrosion rate of the aluminium bronze impellers has been observed in the vertical well pumps
- Reported product water salinity was 180 TDS

3.6.4. Costs

The total investments in the plant including design, supply, installation and commissioning was US$ 12,500,000 in 1983. An additional cost of approximately US$ 5,000,000 was estimated as: government management, drilling of wells, electric power supply, and product distribution pumps. The total capital cost was estimated as US$ 875.$m^{-3}.d^{-1}$ (1983 prices). The production cost was estimated as presented in Table 3.7. Amortization (capital cost), was calculated by assuming 15% interest, 90% plant utilization and a depreciation period of 20 years.

Table 3.7: Production costs of the Ghan Lapski SWRO plant, Malta

US$.m^{-3}		
Electricity	0.43	(40%)
Permeators	0.11	(10%)
Labour and overheads	0.05	(5%)
Spare parts	0.04	(3%)
Chemicals	0.01	(1%)
Filters	0.01	(1%)
Total operating cost	0.65	(60%)
Amortization	0.43	(40%)
Total Cost	**1.08**	(1983 prices)

3.7. Balearic Islands – Ibiza

Eddinger *et al* (1996), reported on the first year of operation of the SWRO plant for the town of Ibiza. The construction of the plant was completed in 1992, but due to delays in the construction of the Beach well Intake system (political reasons); it was in full commercial operation in June 1994. The plant production varies highly between 6,300 - 10,200 m^3.d^{-1}, following the seasonal variations of water consumption in the distribution system of Ibiza. The main features of the plant as detailed by Eddinger are as follows.

3.7.1. Plant design

a. Well Field

At the location of Punta Grossa, a coastal well field was constructed with eight wells of DN 500/400 perforated by percussion to a depth of 50 m into a limestone and dolomite aquifer with good fractures and discontinuities and with a high to medium permeability. The most important concern in the location of the wells was to avoid increasing intrusion of seawater into the aquifers further inland which would cause almost irreversible damage to existing potable water wells in the area.

The individual wells were constructed with a UPVC well casing, screen and a concrete wellhead. The well pumps were made of marine bronze and installed at a level of approximately 40 m. Local metering gear was provided in the well head.

b. Pretreatment

Prior to the pressure sand filters, the raw water quality was measured and dosing points for chlorine and a flocculating agent with a static mixer were provided. For the first mechanical filtration stage, eight units of dual-media filters of 3.5 m diameter with nozzle floor were installed. The filters were made of carbon steel, lined internally with hard rubber to avoid corrosion.

All valves were pneumatic and automatically controlled. Backwashing was performed with air scour and with filtered water. After the filter station, a control valve will adjust the raw water system pressure to a constant value to avoid high pressure fluctuations in the pipeline when the well pumps were started or stopped. The filtered water was collected in a 400 m^3 filtered water tank built of specially coated bolted steel plates. The filtered water pumps (3+2) transfer the filtered water through the cartridge filters to the RO trains. Dosing points for sulphuric acid and sodium bisulphite were provided prior to the cartridge filters (CF). The CF bank with four (3+1) filters provided the second mechanical filtration before the membranes. The filters were also made of carbon steel and internally lined with hard rubber. Standard cotton cartridges of 5 micron were used.

After the CF bank, the filtered feed water quality was metered (temperature, pH and Redox with a special sensor control system). Feed water was dumped through a dump valve if the parameters were outside of the acceptable limits for the RO system. Duplicate dosing systems for pre-chlorination with chlorine gas, sulphuric acid of 96% concentration, flocculating agent and SBS were installed in separate dosing rooms with all required safety gear.

c. Reverse Osmosis section

The desalination of the seawater was performed in three identical RO trains with a nominal capacity of 3000 $m^3.d^{-1}$ each. To overcome the osmotic pressure and to set proper operating conditions, a system pressure of a maximum 69 bar was required.

The pressure was generated by a five-stage segment ring high-pressure pump driven by an 800 kW HV motor. To recover energy from the brine, a Calder PT 3-1 Pelton-type energy recovery turbine was coupled on the same shaft. The energy recovery utilized with this type of turbine was approximately 34%. The hydraulic system parameters were adjusted by use of special high-pressure valves. The materials of the pump, turbine and valves were Duplex (DIN 1.4462) or higher grade. The PN 100 high pressure piping from the pump to the membrane rack and return to the ERT was made of VDM Chronifer 1925 HMO (DIN 1.4529), to avoid corrosion in this area.

The first stage of the RO system consisted of 44 pressure vessels suitable for eight elements. The element type installed at the first stage was TORAY SU 820. The nominal first stage recovery was initially 40%. For increased plant flexibility and reduction of overall membrane replacement costs, a second stage of the system was installed. A booster pump takes permeate from the first stage permeate header and charges the second stage stack. Fifteen pressure vessels suitable for eight brackish water elements were provided in the second stage. The element type, Toray SU 720 L is used. No energy recovery was used in the second stage (uneconomical), but brine was re-circulated to the feed of the first stage for optimization of the flow balance. The second stage recovery was up to 90%.

At present, 42 pressure vessels of the first stage are loaded with seven elements and one dummy which is well suited to produce the nominal permeate quantity while the second stage is not activated. When the second stage is activated, the first stage recovery will be increased to approximately 43% to a maximum of 45% to keep the overall train recovery across both stages at 40% as initial. The number of pressure vessels in service is changed with the production requirements. Flushing pumps are installed to serve for permeate flushing, as well as a separate cleaning system with a 13 m^3 tank, cleaning pump and cartridge filter.

d. Post-treatment

The permeate from the RO trains was transferred to the 120 m^3 concrete surge tank which also acted as permeate storage for product transfer to the water distribution system. The four product water transfer pumps (3+1), transferred the product through a 5 km long pipeline to a large elevated storage tank. A duplicate post-chlorination system and a lime dosing system were installed to turn permeate into good quality drinking water.

Special attention was also given to the problems of water hammer (critical at shut-down of all pumps); hence a sophisticated water hammer protection system with hydraulically dampened non-return valves, air vent valves and over-pressure discharge valves was provided.

e. Brine discharge system

The brine, sand filter backwash water and all other drainage water were collected in a 80 m^3 concrete underground tank which was divided into two compartments. The first compartment (with overflow to the second one) was used to feed the generator cooling pumps (1+1) for the power station lubricating oil and cooling water system. Brine transfer pumps (2+1), transfer the brine through a 3 m long pipeline returning it to the sea at the cliffs of Punta Grossa.

3.7.2. Plant performance

The experience in this plant was summarized by Eddinger summarized as follows:

a. Raw water quality

The following data were reported for the Well Field Data.

Temperature	:	19°C average with a seasonal variation of ± 1°C
SDI	:	0.3 - 1
TDS	:	approximately: 39,300 - 40,500 ppm
pH	:	7.25

One of the biggest advantages of Beach wells, is the virtually stable temperature throughout the year as well as the low SDI factor. The TDS in Ibiza is higher than the usual seawater TDS, probably due to a higher salinity in the groundwater. However, it is reported that almost no potable water is abstracted from the landside aquifers; this is reported to confirm that the location of the well field is correct.

b. Pretreatment data

- Chemical dosing: chlorination stopped, disinfection of complete pretreatment every 6-8 weeks by shock chlorination; SBS dosing stopped; flocculation not required; acid dosing to set a filtered water pH of approximately: 6.5 - 6.8
- Sand filters backwashing: once a week
- Cartridge filters (CF) replacement rates: approximately every 4-6 months

The slow activities of biofouling in the system (long sand filter backwashing intervals and long CF replacement rates) confirm that the pretreatment can be operated without chlorination with close monitoring and continued bimonthly disinfection.

c. Operating data of RO trains without second stage

The data in Table 3.8, are readings taken at the plant in 1995. The number of pressure vessels on stream was different for the trains (ROT - Reverse Osmosis Train). With the second stage in operation, the permeate conductivities dropped to 580/680 for ROT 2 and ROT 3, respectively.

d. Operating data of post - treatment

- Total conductivity of product: approximately: 760 µS
- Post-chlorination: 0.5 - 1 ppm
- pH adjustment: approximately: 15-10 ppm lime dosing to set the pH to about 8.5

Table 3.8: Performance of the Ibiza SWRO plant

Actual plant's reading on 12.5.1995			
Parameter	**ROT 1**	**ROT 2**	**ROT 3**
Flow feed, m^3	337	363	382
Flow permeate, $m^3.h^{-1}$	134	144	153
Pressure feed, bar	66.5	66.7	65.5
Diff: pressure RO, bar	1.9	1.1	1.2
Cond: 1st stage, µS	320*	900	990
PVs' on stream, nos.	36	41	42
Specific power consumption HPP $kWh.m^{-3}$	4.2	3.95	3.85

* With new elements installed.

3.8. Canary Islands - Lanzarote

Gotor (1995) summarized the experience of desalination in the Canary Islands where some plants are based on Beach well Intakes. The Canary Islands Archipelago consists of seven islands; Tenerife, La Plama, La Gomera, El Hiero, Gran Canaria, Lanzarote and Fuerteventura.

Lanzarote is the most eastern island of the Canarian Archipelago. It occupies a surface area of 796 km^2 and has a population of nearly 80,000. Tourist population of the island reaches in excess of 100,000 per month. Most of the water in the island is supplied from seawater desalination plants. Some data of these plants are shown in the Table 3.9.

The largest plant on the island is Lanzarote III. The first of two units has a capacity of 10,000 $m^3.d^{-1}$ using the SWRO process. Feed water is taken from a well in basaltic rock at the plant site. TDS of the feed water is 37,011 $mg.l^{-1}$ at 20 °C. A filtration step is included with three sand filters allowing one to be out of service for washing. Due to the high quality of the feed, pretreatment includes only hypochlorite, sodium bisulphite and antiscalant dosing. Polishing filtration follows with polypropylene cartridges.

Table 3.9: Lanzarote Island desalination plants

Plant	Type	Capacity $m^3.d^{-1}$	Start-Up Year	Capital Cost	
				Million US$	$US\$.m^{-3.}d^{-1}$
Lanzarote II	SWRO	7,500	1986	9.23	1,231
Lanzarote III	SWRO	10,000	1994	10.31	1,031
INALSA I	SWRO	5,000	1990	4.85	970
INALSA II	VC	1,100	1983-90	1.23	1,119
INALSA SUR	VC	4,800	1990-94	4.00	833
Lanzarote III (Extension)	SWRO	5,000	1994	4.05	809

Two high pressure pumps are provided, Ingersoll Rand type 6 x 46 DA-5, operating at 2980 rpm, constructed in stainless steel, AISI 904L. Energy recovery is by two hydraulic turbines, Ingersoll Rand, type 4 x 11DAT-S, operating at 2980 rpm, and constructed of stainless steel, AISI 316L. The membrane blocks consist of 1674 Dow-Filmtec membranes type SW-HR30-8040, in 279 vessels, and arranged in 2 stage array. Operating pressure is 68.2 bar (990 psi), and overall conversion is 45%. Post-treatment consists of sodium carbonate dosing to increase the pH. Gotor, as shown in Table 3.10 summarized some salient cost data of this plant.

3.9. SWRO plants with Beach wells Intakes in Greece

3.9.1. Existing plants

Beach wells abstracting groundwater of seawater quality exist in Greece to cover freshwater needs in the islands and for industrial cooling purposes. Experience in the application of Beach wells as Intakes for Seawater Reverse Osmosis (SWRO) is still limited. Unfortunately, the number of Beach wells in Greece is limited because of morphological, geological, and economical reasons. Nineteen Reverse Osmosis desalination plants exist in Greece and five of them use the Beach well method for seawater Intake (Table 3.11).

Some data were collected from these existing Beach wells that are already used as SWRO Intakes. The data were collected in line with the cost estimate model developed for the present study (Appendix C), and are summarized in Table 3.12. More information on these plants was collected and presented as follows:

Mykonos is an island that belongs to the Prefecturate of Cyclades and has a Reverse Osmosis desalination plant that utilizes Beach wells. The plant has a production capacity of 1,200 $m^3.d^{-1}$ and the salinity of the Intake seawater is approximately 32,000 $mg.l^{-1}$. There are 7 wells located a few metres away from the coast with a maximum potential yield of 500 $m^3.d^{-1}$. The depth to water table is approximately 4 metres and the depth to the dynamic water level is about 5 metres. The fresh water fraction pumped is negligible.

Table 3.10: Salient cost data of the expanded Lanzarote III SWRO plant

Total plant capacity	$m^3.d^{-1}$	15,000
1994 Production	Million m^3	4.1
Recovery ratio		0.45
1994 Energy consumption	Million kWh per year	22.7
Total direct capital cost (TDC)	Million US$	14.35
Fixed charge rate		10.04
Plant maintenance personnel	Number, Total	27
Chemical costs (1994)	Million US$	0.27
Indirect capital costs		37% TDC
Electric power cost		0.12 per kWh
Membrane replacement		20% per Year
Maintenance & Parts replacement		0.8% TCC
Insurance		0.5% TCC
Labour		$460,000per year
Total Direct Installed Cost (TDC)	Million US$	10.48
Total Indirect Cost	Million US$	3.88
Total Capital Costs (TCC)	**Million US$**	**14.26**
Annual fixed charge cost	**Million US$**	**1.44**
Operation & maintenance costs	Million US$	
Electric Power, Million. 22.7 kWh per year @ $0.12 per kWh		2.72
Chemicals		0.29
Labour, operation and maintenance		0.46
Maintenance and parts replacement		0.11
Membrane replacement		0.17
Insurance		0.07
Total Annual Costs	**Million US$**	**5.27**
Total Cost of Water	**US$.$m^{-3}$**	**1.29**

With regard to the lithology of the area, it comprises a relatively thick layer of medium to coarse sand that overlies a formation of metamorphic rocks. The extent of the island is limited and the impermeable geologic formations that dominate do not facilitate the existence of a significant underground water table and therefore the possibility for great impacts from the use of Beach wells is small. The transmissivity of the sandy soil that Beach wells intersect is approximately 110,000 $m^3.y^{-1}.m^{-1}$, which indicates a soil with relatively high transmissivity and good potential filtration. Furthermore, the sea bottom gradient is very low and because of the aforementioned unfavourable conditions for the formation of a significant amount of underground water in the specific island, the contribution of freshwater is extremely low. Thus, it may be argued, that in the particular plant; Beach wells are an effective approach since it provides adequate yield and high quality (natural filtration) of seawater without causing pollution problems to the freshwater storage locations.

Table 3.11: R O installations in Greece

Name	Location	Type	Intake
Ithaki plant	Ithaki island, Ionion Sea	R.O.	Open sea
Ermoupolis plant	Syros island, Aegean Sea	R.O.	Open sea
Ermoupolis plant	Syros island, Aegean Sea	R.O.	Beach wells
Kini plant	Syros island, Aegean Sea	R.O.	Beach wells
Mykonos plant	Mykonos island, Aegean Sea	R.O.	Beach wells
TEMAK	Mykonos island, Aegean Sea	R.O.	Open sea
Oia plant	Santorini island, Agean Sea	R.O.	Open sea
Fyra plant	Santorini island, Agean Sea	R.O.	Open sea
Nisyros plant	Nisyros island, Agean Sea	R.O.	Open sea
Kasteloriza plant	Kastelorizo island, Agean Sea	R.O.	Open sea
Spetsopoula plant	Spetsopoula island, Agean Sea	R.O.	Open sea
EKO	Thessaloniki, Macedonia	R.O.	Open sea
LEVER	Attiki, Central Greece	R.O.	Open sea
PETROLA	Attiki, Central Greece	R.O.	Beach wells
HALY	Attiki, Central Greece	R.O.	Beach wells
HOECHST	Attiki, Central Greece	R.O.	Open sea
ROLCO	Attiki, Central Greece	R.O.	Open sea
ETMA	Attiki, Central Greece	R.O.	Open sea
ETMA	Attiki, Central Greece	R.O.	Open sea

Table 3.12: Greece desalination and Beach well Intake cost estimates

	Reverse Osmosis Desalination Plants		
A. TITLES			
Plant name	Mykonos	Syros, Kini	Neorio
Type of plant	SWRO	SWRO	SWRO
Type of Intake	Beach wells	Beach wells	Beach wells
B. DESALINATION PLANT			
Plant capacity (m^3 per day)	1200	144	150
Annual Production (m^3)	120000	25920	22500
Feed water salinity (ppm)	32000	43000	40000
Recovery (%)	30	39	35
Total site area (m^2)	500		60
Distance from Intake (m)	200		20
Distance to brine disposal (m)	200		30

Table 3.12 (continued)

C. HYDROGEOLOGY			
Single well capacity ($m^3. d^{-1}$)	170	144	150
Depth of well (m)	5.5		10
Depth to dynamic water level (m)	4		-
D. FINANCE			
Symbol of Foreign currency (F.C.)	Drs	Drs	Drs
Exchange rate	1dr/ 338 ECU	1dr/ 338 ECU	1dr/ 338 ECU
Date of cost estimate	1989	1993	1992
Interest rate (%) (average from date of cost estimate up to date) (3/2000)	19.5% 9%	18.2% 9%	19% 9%
Life time and lead (years)	10		
E. CHEMICALS			
Type of Chemical	Sodium Bisulfite, Chlorine, Sulphuric acid	Sodium hypoclorite, Ferric chloride, Sulphuric acid	Chlorine, Sulphuric acid
Amount of chemicals used in a daily basis (kg)	25 $kg.d^{-1}$ 20 $L.d^{-1}$	250 $kg.d^{-1}$	~0.15 $kg.d^{-1}$ of each type
F. ENERGY			
Energy consumption ($kWh.m^{-3}$)	5	6.5	
	REVERSE OSMOSIS DESALINATION PLANTS		
G. DESALINATION PLANT COMPONENTS AND INPUTS			
Pre-treatment	Sodium Bisulfite for disinfection & dechlorination,	Sodium hypochlorite for sterilization,	Chlorination for sterilization,
	Sulphuric acid for pH stabilization and alkaline scale prevention	Ferric chloride to prevent the colloidals,	Sulphuric acid for pH stabilization and alkaline scale prevention
		Sulphuric acid for pH stabilization and alkaline scaling prevention	

Table 3.12 (continued)

Post treatment	Passage through calcite column for product water enrichment,	Sulphuric acid as a dissolver of the calcium used,	
		Passage through calcite column	Chlorination for sterilization
	Chlorination for sterilization	Caustic soda for product water enrichment,	Passage through calcite column
		Dechlorination	
Brine disposal	Sea	Sea	Sea
Investments			
Fixed cost (drs)	1000 Mdrs (1999)	66 Mdrs (1993)	70 Mdrs (1992)
Maintenance costs ($drs.m^{-3}$)	750 (1999)	30 (1993)	
Chemical costs ($drs.m^{-3}$)		17.8 (1993)	
Energy costs ($drs.m^{-3}$)		132 (1993)	150 (1999)
Total Operational & Maintenance costs ($drs.m^{-3}$)	1040 (1999)	250 (1993)	350 (1999)
* There is no relevant legislation or monitoring processes concerning underground water protection.			

Another Reverse Osmosis desalination plant that uses Beach wells in Greece is located at the Neorio of Syros Island. It is a plant that produces water for the needs of Neorion shipyard and therefore its capacity is limited. It incorporates a single well that provides 150 $m^3.d^{-1}$ of desalinated water and the electricity conductance of the uptake seawater is approximately 27 $iS.ml^{-1}$. The well is constructed 15 metres away from the coast and its depth is 15 metres. The aquifer is encountered at about 10 metres depth while the depth to the dynamic water level is approximately 12 metres. The sea contains medium to coarse sand and fine to coarse gravel in the surface soil layer while relatively impermeable metamorphic rocks underlie the above formation. Again, the permeability of the soil used by the well is relatively high, since the soil particles are coarse in this case, and thus the infiltration rate is increased in relation to Mykonos plant. The transmissivity of the sandy formation is approximately 200,000 $m^3.y^{-1}.m^{-1}$, which offers an efficient yield but it can be used mainly for industrial applications since the potential for natural filtration is not as high as that required for domestic uses. Consequently, it can be stated, that for the purpose of the specific shipyard, Beach well is a very effective approach with low operational and maintenance costs and it provides an adequate amount of desalinated water with relatively good quality.

The third Reverse Osmosis desalination plant with Beach wells is located on the island of Syros, in the municipality of Ano Syros. It is used mainly to cover domestic needs of the island together with other two desalination plants with open sea Intakes. Thus, the availability of both, Beach wells method and open sea Intake method, in the same area with similar hydro geologic properties facilitates the comparison of these two methods for acquiring seawater in desalination plants. Particularly, the average daily production of Ano Syros plant is 136 $m^3.d^{-1}$ and the maximum Beach well yield is approximately 250 m^3. There are two Beach wells in the specific plant but currently only one of them is used and the other has been constructed for backup and to meet any possible increase of future needs. The Beach wells are constructed only a few metres (~10m) away from the coast and their depth is 4 m. The existing aquifer is located at a depth of 1.5 metres while the depth to the dynamic water table is only 3 m. The low depth of the aquifer provides an indication of high permeability for the surface soil layer, which will be certified by the measured infiltration rate of the formation. The fraction of freshwater pumped by the Beach wells is negligible in this case also because the dominant hydrogeologic conditions do not allow the existence of an aquifer with significant yields. This is illustrated by taking into account the salinity of the uptake water, which is 43,000 ppm, and by comparing it with the salinity of the other plant in the island that acquires the seawater directly from the sea and presents similar values (42,000 – 43,000 ppm).

The upper soil layer in the area of pumping consists of medium to coarse sand, which overlies metamorphic formation that presents low permeability. Thus, the surface geologic formation has a relatively high transmissivity, which is estimated at 125,000 $m^3.y^{-1}.m^{-1}$. Therefore, the hydrogeologic conditions offer the potential for significant productivity and good quality of the acquired water and since the Beach wells approach does not have serious impacts on any water body in the specific region, it constitutes an efficient technique for the desalination of seawater. Additionally, using this method provides important economic benefits as concerns the operational and maintenance costs of the plant that is illustrated by comparing the relevant figures of the Beach well desalination plant with the respective figures of the desalination plants that use alternative methods in Syros Island.

Table 3.13: Significant characteristics of the Greek desalination plant

Desalination plants	Transmissivity ($m^3.y^{-1}.m^{-1}$)	Single well capacity ($m^3.d^{-1}$)	Depth of wells (m)	Depth of water table	Freshwater fraction pumped (%)
Mykonos	110,000	500	6	4	~0
Neorio, Syros	200,00	150	15	10	~0
Ano Syros, Syros	125,000	250	4	1.5	~0

3.9.2. Conclusions of Greek experience

Conclusively, using the Geek experience from all the aforementioned desalination applications it can be stated that Beach wells constitute an efficient technique if the decision for its implementation in a desalination process is based on some credible scientific criteria. In particular, transmissivity plays an important role for the adoption of a specific seawater Intake method since it determines the pumping capacity, the characteristics of the existing aquifer and the possible intrusion of the seawater to the freshwater. Thus, in the Greek applications, Beach wells are selected to use in areas with relatively high transmissivity (>100,000 $m^3.y^{-1}.m^{-1}$) in the upper soil layer while the lower formations are impermeable. These conditions secure high productivity and good initial quality of the seawater, which reduces the costs of the plant significantly.

Further, the depth of the wells and their distance from the coast should be small (>10m and >15m respectively) in order to eliminate the possibility of abstracting significant amounts of freshwater from the aquifer. Further, if the Beach wells are adjacent to the coast and their depth from the ground surface is less than the existing aquifer's depth, then a very useful filtration will take place without significantly affecting the freshwater storage. In the Greek installations, the choice for implementing Beach wells has been made in areas with insignificant aquifers due to the small extent (small islands) and the hydrogeologic properties of the area. Therefore, in these cases; there are no potential environmental problems concerning impact of the desalination plants on the aquifers.

Moreover, the lithology of the area also plays an important role in the selection of the Intake method. More specifically, in all Greek plants the Beach wells are based on sandy soils, which provide relatively efficient pumping capacity and convenience during construction with good natural filtration. Nevertheless, many coasts comprise gravels and boulders, which present even greater transmissivity and thus increased pumping capacity but they provide very limited filtration and difficulty in construction. Therefore, sandy soils with moderate to high transmissivity (100,000 – 150,000 $m^3.y^{-1}.m^{-1}$) should be preferred for the Beach wells method.

Finally, the fresh water fraction pumped has to be under serious consideration and thus continuous monitoring of the Intake in desalination plants should be planned which will be followed by strict penalties if found to interfere with freshwater abstraction. Unfortunately, in Greece there is no relevant legislation to protect the underground water storage or to impose limits of fresh water abstraction. However, it is unlikely that the existing desalination plants impose any adverse impact on the underground water up to the present time.

More considerations in the design of Beach wells based on the combined Greek and worldwide practice and experience are summarized in Appendix H.

3.10. Saudi Arabia

Some difficulties were encountered in the data collection in this part of the world. The discussion is based on the publications and not on the field data.

Hyden (1985) described the advantages and disadvantages of using shallow Beach wells in comparison to open seawater Intake in the Tanajib plant located on the Arabian Gulf Coast with a net capacity of 600,000 gpd (2,250 $m^3. d^{-1}$)

The cited advantages of Beach wells are:

- No expensive off-shore installations
- excellent mechanical cleaning of the seawater by submarine filtration
- reduction of marine growth and microorganism problems
- avoidance of any flocculation, sedimentation and media-filtration
- good balance of seawater temperature and salinity

The disadvantages cited are:

- complex pumping installations
- insufficient knowledge of the subterranean flow regime
- its influence on raw water quality

However, the above disadvantages may be explained by drilling test wells. Such test wells will determine the radius of influence of a single well and therefore the necessary well spacing required.

3.11. Performance of Beach wells

Beach wells face following two types of problems:

1. Operation and maintenance of wells in general
2. Corrosion and other problems resulting from the high ion concentration of seawater

Many major problems at operating plants have been caused by improper or poor well construction. In many cases, well failure causes a sudden change in the quality of water being pumped into the treatment plant. The most common causes of well failure are:

a) Borehole collapse
b) Corrosion of casing
c) Improper or defective construction techniques
d) Growth of organisms within the well bore
e) Water intrusion from another source
f) Formation of mineral concentrations
g) Crusts in the open hole or screened section of the well bore

A recent publication (Abo'abat *et al*, 1998) summarizes typical problems of operation and maintenance of aged deep wells.

The following problems were identified in the Riyadh Water Supply System, which included about 160 wells:

a) Damage to the basic structure mainly occurs due to the old age of the well. The chemical reactions by dissolved salts cause corrosion in the casing pipes and as a result, the cement structure fails
b) Corrosion of the casing pipes and screens brings sand in the pumped water
c) Bending of well assembly due to geological reasons, which was not known at the time of designing and construction of the well
d) Pollution, due to nearby sewage lines or mainly use of cement which cannot resist sulphur
e) Installation problems, mainly of the equipment inside the well or the casing pipes due to corrosion and resulting water leakages

In view of the above points (a-e), and other common problems, the recommendations of this Riyadh study were summarized as follows:

a) Filters and casing pipes should conform to international specifications
b) Cement should be applied under pressure between the well hole and the casing pipes
c) A specialized company should test the casing and installations
d) The cement grout-drying time of 72 hours must be adhered to with no operation permitted during this period
e) Four sealant ‘O’ rings should be used to insulate the threads in all connections/couplings

f) Non-return valves should be used whenever recommended
g) Pumping to be discontinued if sand appears in the pumped water
h) The deep well straightness should always be taken care of. Submersible pumps can solve the problem if the well is not straight, instead of drilling a new well
i) The liner material and valves used at the water layer should be of stainless steel and the openings should be as per specification
j) Well locations should be as far away as possible from sewer and sewer plants. If pollution occurs, the well should be filled up and replaced by a new one farther away from the pollution source
k) Pipes and all equipment to be painted as per specification
l) Care to be taken when installing the equipment, *i.e.* not to remove the paint
m) Stainless steel pipes should be used where steel bacteria is present
n) 'Packing' and 'bushings' used, should be of specified materials
o) A maintenance plan should be followed for replacement of all equipment as per specification to avoid sudden failure or major problems
p) Periodical and regular maintenance should be adhered to

Some more considerations are summarized in Appendix H.

3.12. Other non-surface Intakes

Two other types of non-surface Intakes are considered in addition to Beach wells *i.e.* beach galleries and seabed filtration. These are summarized in the Chapter 4. An interesting development by the Danish Company: (BMS, see 4.7), is of galleries for beach management with the objective of controlling beach sand erosion. The origin of this development was a gallery for extracting seawater for regular use.

3.13. Surface water Intakes and their disadvantages

Recently, Don Hornburg summarized the design of surface seawater Intake systems for thermal desalination plants and the problems encountered (in press). The main problems result in decreased production rate, efficiency, increased maintenance and downtime. The main problems observed are summarized in Table 3.14.

Most of these problems and their causes are related to the effects of the open sea on the Intakes. Many other authors summarized these problems of surface Intakes: Iso, (1977), Japan; Gabrielli and Ripasatvi, (1979) Italy, El-Sai *et al*, (1981) Kuwait, Kreshman, (1985) Libya and Elarbash, (1991) Malta.

Table 3.14: Problems in desalination plants resulting from poor surface Intake design or operation

Observations	Problem/Results	Possible Causes
Increasing and higher than normal seawater supply temperature	Decrease in plant production rate and performance	Re-circulation from discharge to Intake
		Shallow flow to Intake point
Heat reject tube erosion	Tube leaks/replacement	Sand in cooling water supply
	High TDS product water	Shells lodging in tubes
Heat reject tube corrosion/pitting	Tube leaks/replacement	Deposits in tubes
	High TDS product water	Ammonia or sulphides in cooling water
Plugging of tubes	Decreased cooling water flow	Trash, shells, mussels from Intake
	Decreased plant production rate	
Decreased cooling water flow	Decreased Production rate	Low level in pump basin
		Intake pipes fouled
		Fouled supply pipe
Fouling of reject tubes	Decreased production rate	Lack of chlorine residual
		Oil from Intake

The main disadvantages of surface Intakes may be summarized in the following (main problems). Most of them are focused on high risks of fouling and clogging of membranes and filters and structural damage to the open sea Intakes.

1. Suspended matter in seawater may be considerable. The SDI of seawater may reach 4.6 – 6.7, while in Beach wells, it is approximately 2.0. A high SDI requires expensive pretreatment. The greater part of investment, chemicals and manpower costs can be saved with the lower SDI of Beach wells
2. Seawater quality may be affected by industrial sewage disposal resulting in high concentrations of organic, inorganic and toxic materials
3. Pollution from oil tankers and other vessels often results in the shutdown of desalination plants
4. Change in marine flows resulting from winds and storms may result in pollution of the Intake from unpredictable sources
5. The Intake structures may be accidentally damaged by exceptional marine currents, sand blocking, and ship accidents. Two pipelines may provide more reliability but a great part of the plants capacity may become idle after an accident, while in Beach wells, failure of one well is usually only a small fraction of the total capacity

6. Laying of marine pipelines may have an adverse impact on ecosystems and other marine values *e.g.* archaeological sites
7. Operation and maintenance costs of marine pipelines may be very high
8. The RO process is substantially affected by the temperature of feed water and the membrane life span is adversely affected by temperature variations. The temperature variations of feed water are much higher in surface Intakes than in subsurface Intakes
9. There is competition between coastal land uses and other purposes *i.e.* recreation, tourism, power plants and harbours. This may be reflected in high costs of land

4. TYPES OF NON-SURFACE SEAWATER INTAKES

Non-surface Intakes are installed in boreholes or excavations reaching water bearing formations below the seabed or adjacent to the coastline. A major part of such an installation usually consists of screened pipes, sometimes surrounded by gravel pack. This part collects water (mostly originating in the sea) by lowering the inside pressure below seawater pressure.

Three main types of non-surface seawater Intakes are considered in the present study:

1. Beach wells
2. Beach galleries
3. Seabed filtration

In addition, some variations on these types are also examined:

- Seabed offshore wells
- Seabed tunnelling
- Ranney type wells with large diameter shafts
- Inclines/Slanting wells

A short description of the first types compared to surface Intakes (Figure 4.1), was presented by Wright and Missier, (1997), and is summarized in the following sections.

4.1. Beach wells

Beach wells (Figure 4.2) consist of casing pipes (preferably non-metallic) with diameters of 15.24 – 60.96 cm (6" - 24"), introduced into boreholes with screened sections usually at their lower parts. The diameters of the well screens are usually 15.24 – 30.48 cm (6" - 12"). A turbine pump of stainless steel is lowered into the casing below the water table in the well. The pump motor may be integrated with the pump and submerged (submersible pump) or be installed at the top level, driving the pump by a long shaft (Turbine pump). More information on some aspects of Beach well structures and sizing is given in Appendix H.

4.2. Horizontal beach gallery

Horizontal beach gallery, see Figure 4.4, is an alternative to Beach wells when the thickness of the water bearing formation is small or when its permeability is low. It is constructed by digging a ditch on the beach parallel to the coast to a depth of up to 7 m. The ditch is filled with a graded gravel pack in which a screened pipeline collector is installed in the majority of cases. Beach sand is used to fill the ditch above the gravel pack.

Water is pumped 'up' from the gallery by well points, submersible or other pumps at intervals of 30 - 500 m. Beach management galleries (Section 4.8) are a variation of this concept.

4.3. Seabed filtration

Seabed Filtration (Figure 4.3) is constructed by dredging or trenching the seabed bottom in parallel to the coastline at a distance of approximately 100 - 500 m. The gallery is 10 - 20 m wide to a depth of 2 m below the seabed. Pipelines connect the gallery to the coast. The trench is filled with a graded gravel and sand pack. The system operates similarly to a slow sand filter. For regenerating capacity after clogging, the upper sand layer of 3 - 5 cm is removed and replaced every 0.5 - 3 years.

No seabed filtration is known as a desalination Intake but it is used in riverbeds in conventional water supplies (Hartung and Tuepke, (1963).

4.4. Offshore seabed wells

Offshore seabed wells are mounted on derricks installed in the sea. This offshore oil-well type is very expensive and may be justified for high discharge rates that would endanger a fresh water aquifer on land. A recent study (TAHAL, 2000), shows that such a solution may be economically viable if the seabed wells are drilled from existing harbour jetties. The oil-unloading harbour jetties may serve for this purpose near Ashkelon in Israel.

4.5. Seabed tunnelling

Seabed tunnelling with radial wells was investigated by TAHAL for the Israel Water Commission (TAHAL, 2000). This study was concerned with a large desalination plant with a daily capacity of approximately 290,000 m^3 per day. A tunnel was designed with the following dimensions:

Length of tunnel:	800 m (active), 1300 m (total)
Depth of tunnel (below sea bottom):	20 m
Diameter (Inner):	2.7 m
Material:	Concrete
Intake:	120 Radial boreholes (to be bored from the tunnel) 3 each at intervals of 20 m. Two horizontal boreholes and one vertical borehole
Estimated cost:	US$ 18,000,000

The cost of an alternative of seabed wells near existing jetties (Section 4.4) was estimated at US$ 3,300,000. This means that seabed tunnelling is expected to be an inferior concept, even for large plants.

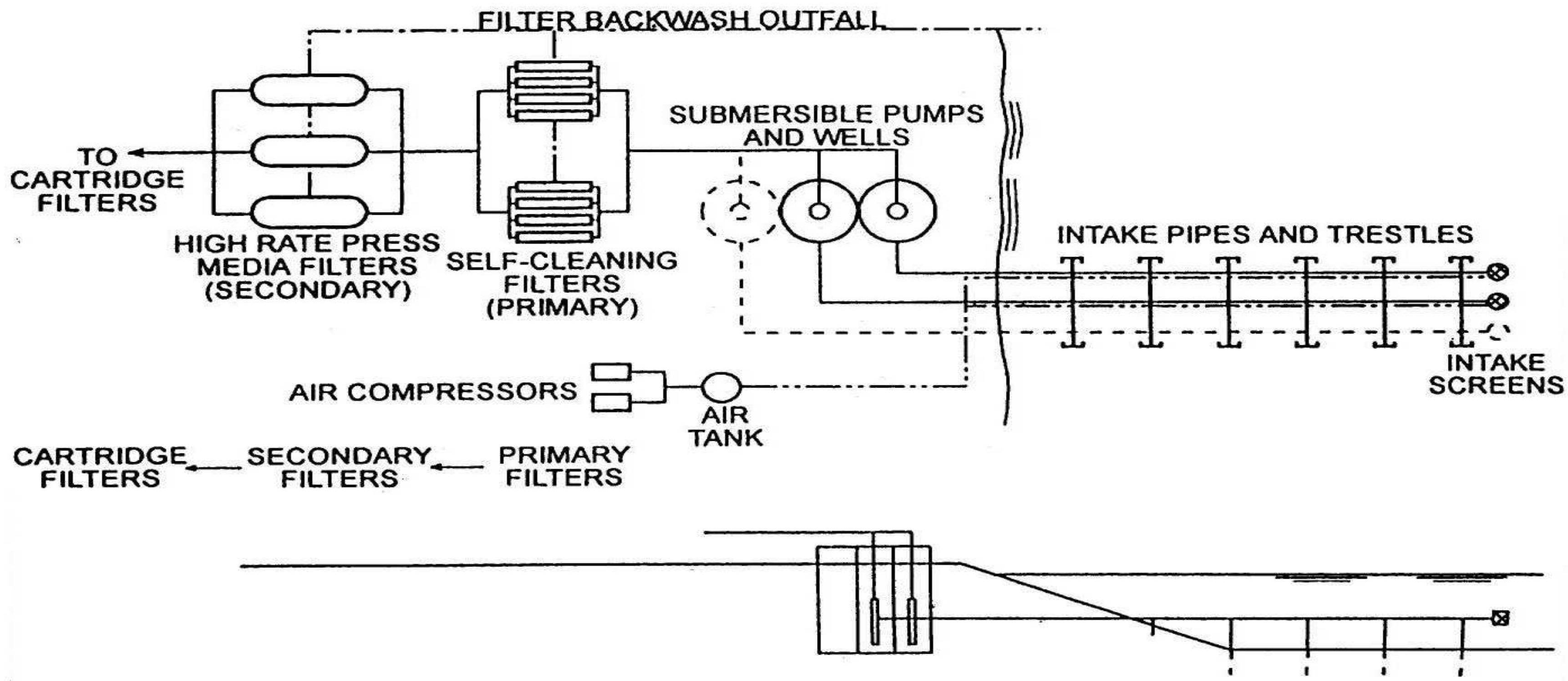


Figure 4.1: Typical surface water Intake & pretreatment system

Source: Wright R. R. and Missier T.M. (1997)

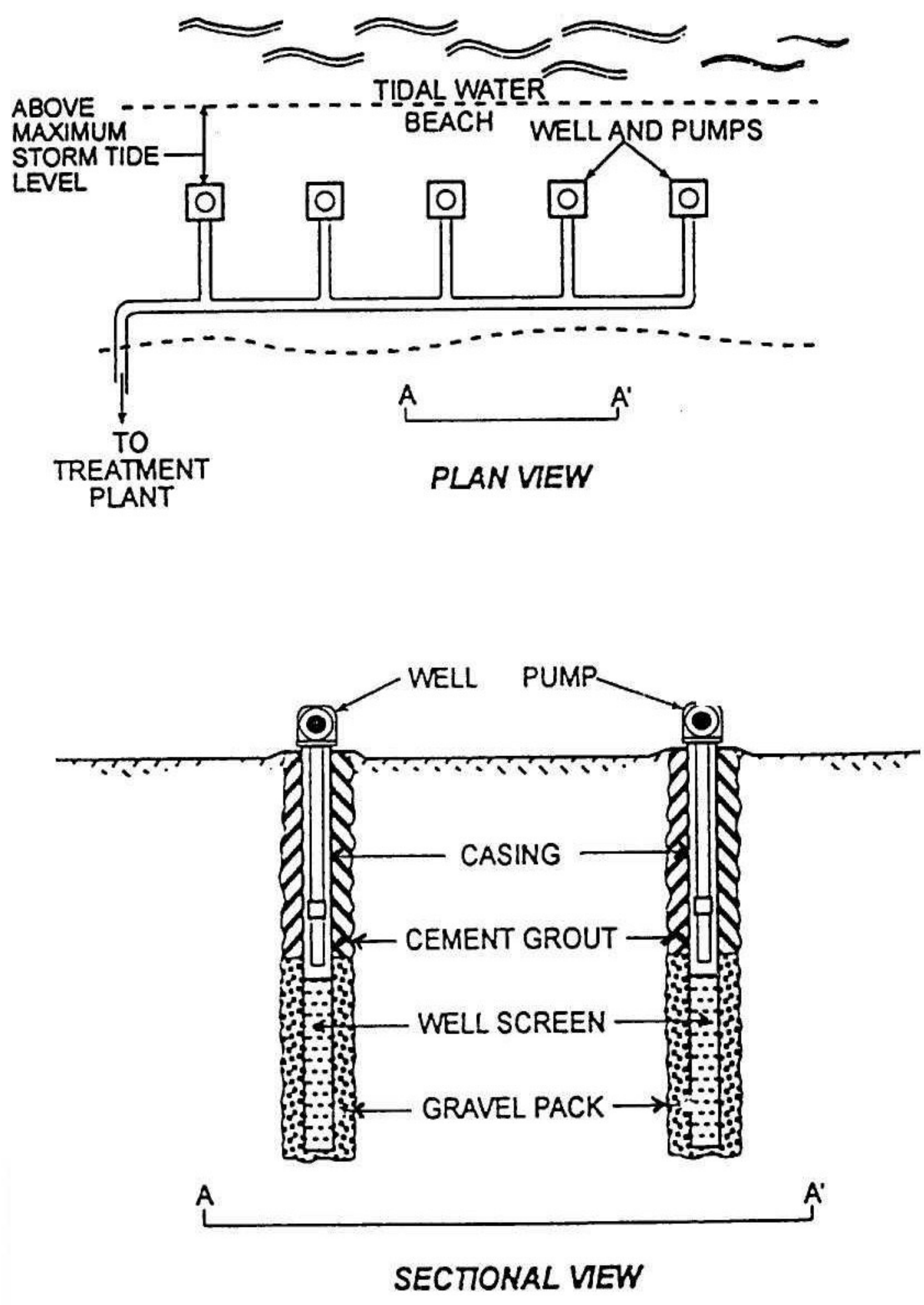


Figure 4.2: Beach well system
Source: Wright R. R. and Missier T.M. (1997)

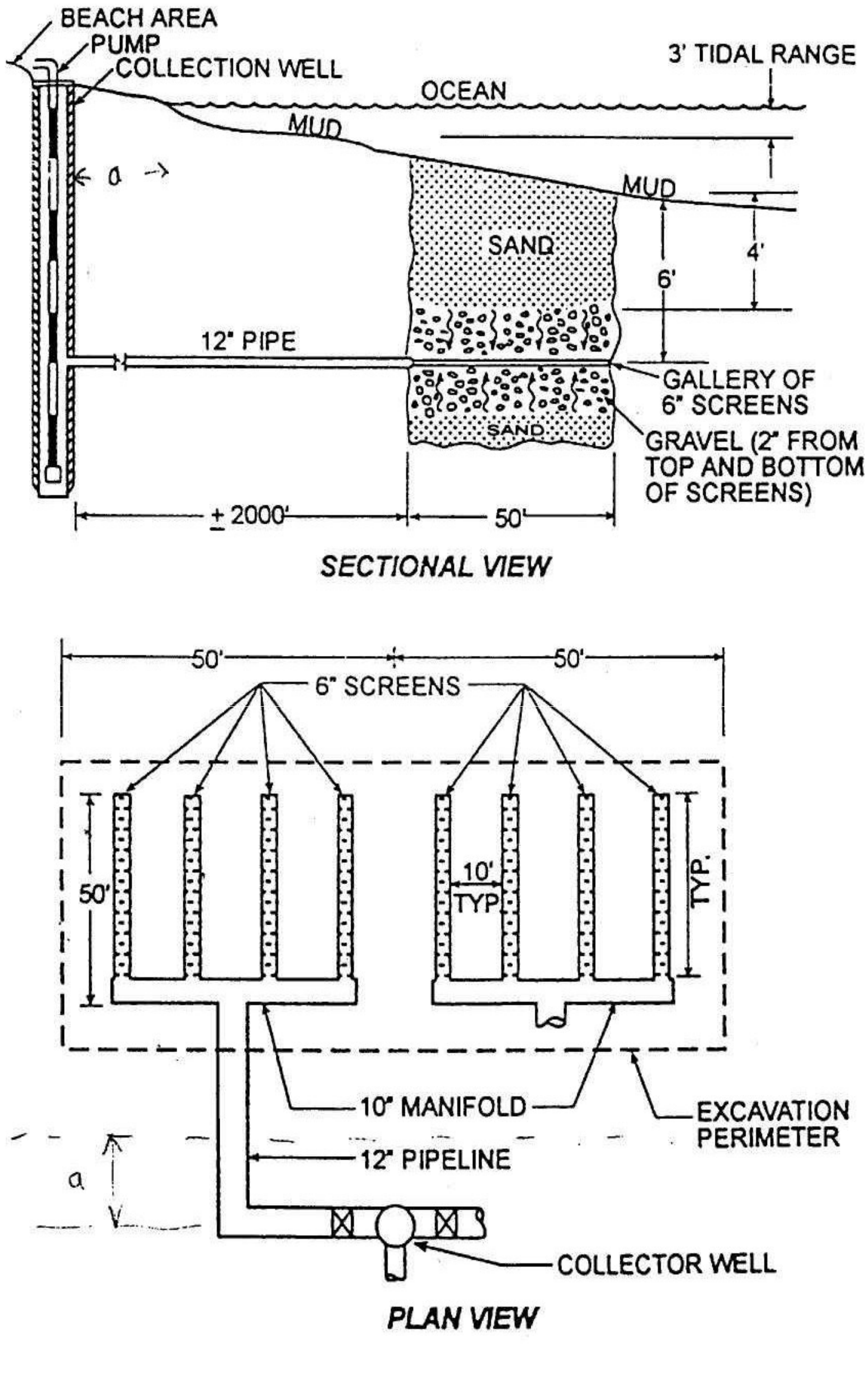


Figure 4.3: Seabed filtration system
Source: Wright R. R. and Missier T.M. (1997)

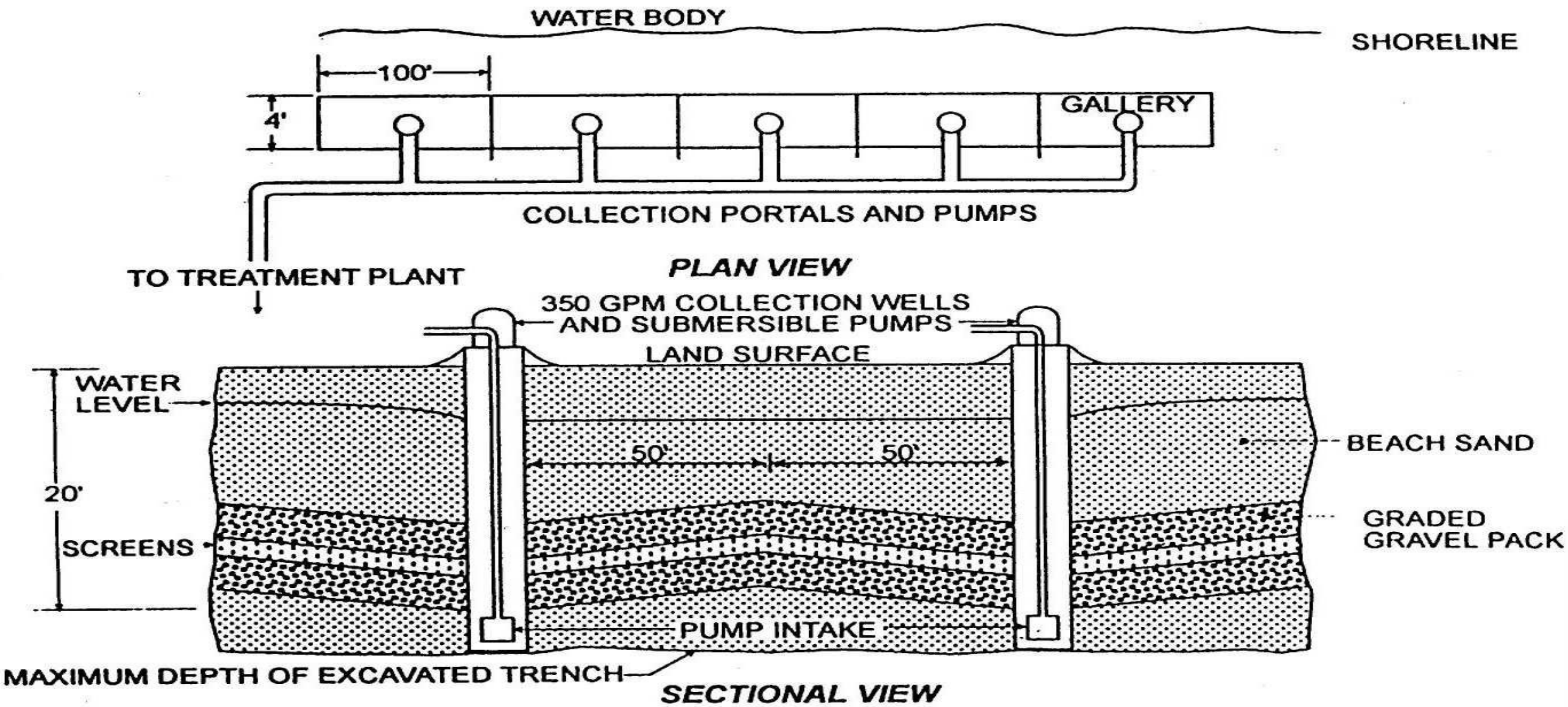


Figure 4.4: Horizontal gallery collection system

Source: Wright R. R. and Missier T.M. (1997)

4.6. Ranney type wells

Ranney type wells are an extension of the horizontal beach gallery principle when the aquifer is thin, transmissivity of the formations is low, and longer gallery sections are required. The horizontal gallery sections are radial and drilled horizontally from a large central shaft (3 – 5 m) diameter. Water supplied to the shaft is by gravity and pumped away using a low-pressure pump. This method is popular for pumping high quality water from alluvial river envelops *e.g.* along the Danube River. This method is less attractive than SWRO Intake since inland radial galleries would be eliminated to avoid adverse influencing of the fresh groundwater inland.

4.7. Inclined/Slanting wells

This technique applies drilling machines in an inclined position. Principally, it may reach underground Intake locations offshore as required by stationing the drilling machine on land and not on expensive marine devices.

The experience gained is with oil wells only (private communication) and no experience in water wells is available at present.

4.8. Beach drains management system

A Danish company as a new approach to coastal restoration has developed Beach Management Systems (BMS), where the loss of beaches threatens recreational revenue and stability. The concept and experience is summarized in Vesterby H. (1991), Vesterby *et al* (1999) also, unpublished reports and a videocassette by BMS. A coastal drain is claimed to be an effective engineering approach to coastal erosion control and beach restoration. By reducing the hydraulic pressure, the system induces a gradient towards its drain, reduces the hydraulic pressure, and creates an unsaturated zone in the beach face (Figure 4.5). This zone makes downward percolation of wave run-up possible throughout the year and cuts off the subterranean flow of water to the ocean at the seepage face. As a result, the volume of the backwash is less, the erosion process is reduced and additional sand is deposited on the beach.

Hydrodynamic models developed by the Danish Hydraulic Institute, demonstrate the process of flow direction reversal at the seepage face of groundwater and how this range is transformed from saturated to unsaturated (Figures 4.6 and 4.7).

Typical dimension of these drains are (Appendix G):

Length of drain:	200 - 800 m
Total capacity:	100 - 1,400 m^3 per hour
Distance from shoreline:	5 - 10 m
Invert elevation:	-1 -2.5 m (a.m.s.l.)
Drain diameter:	113 - 450 mm

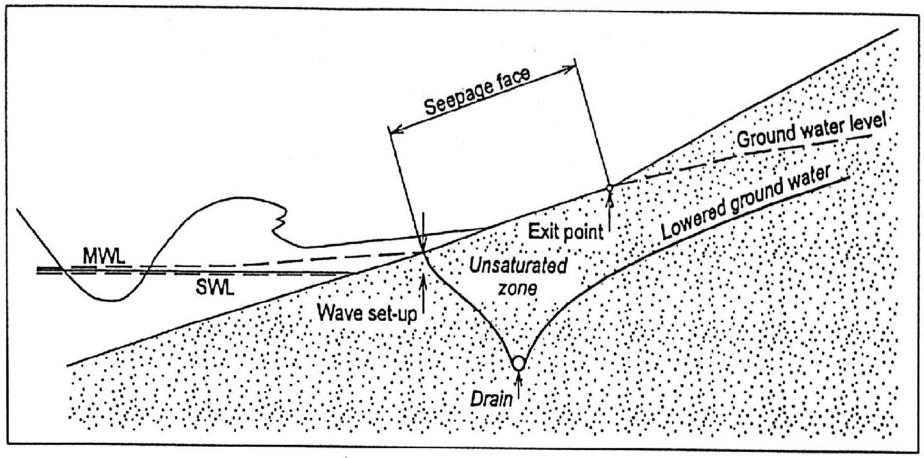


Figure 4.5: Cross-Section. Water table lowered by beach drain (not to scale)

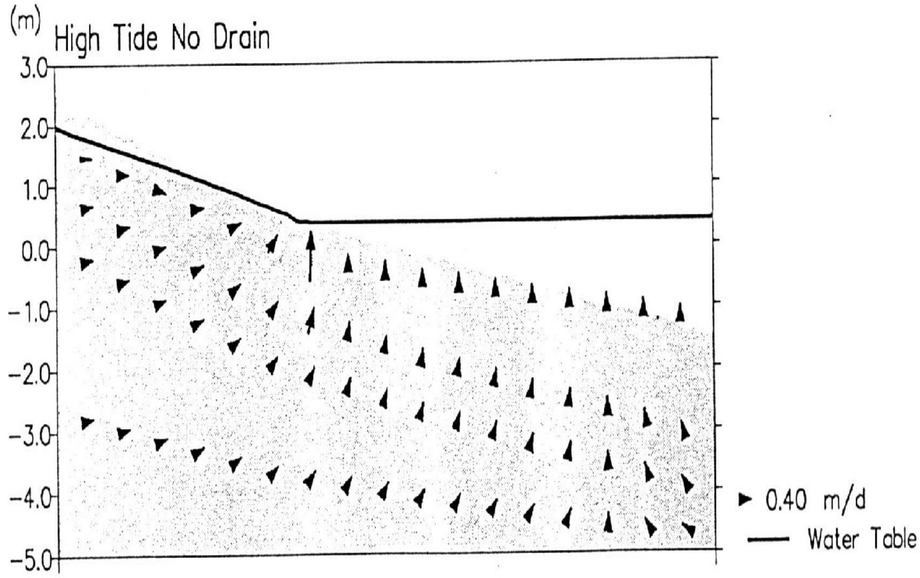


Figure 4.6: Groundwater table and flow patterns during high tide without drain

In most of the systems reported by BMS (Appendix G), the water pumped out of the drain is disposed of in the sea. In a few cases, this water is used for swimming pools or aquaculture. The history of BMS started however, with a beach gallery intended for water use, which failed due to the regression of the beach line resulting in lowering the hydraulic capacity. A multi purpose BMS using the pumped water for feeding desalination plants would have an additional benefit. In addition, the experience gained in the design and operation of beach drains is an important contribution to the design criteria used in the present study.

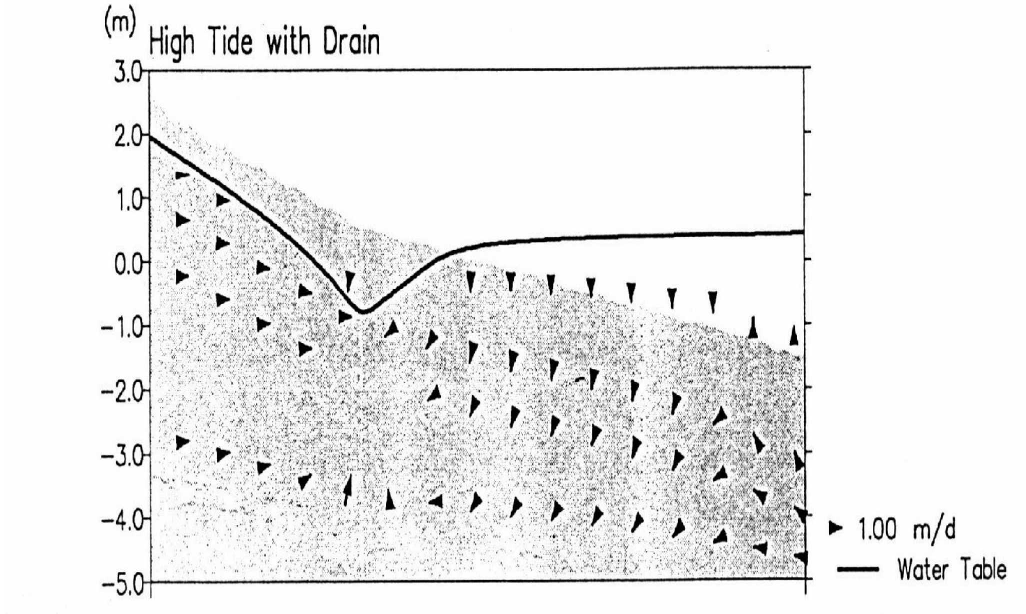


Figure 4.7: Groundwater table and flow patterns during high tide with drain

5. SUITABILITY OF INTAKE TYPES TO PROCESS AND SITE CONDITIONS

5.1. Introduction

The main purpose of the Intake system is to provide a reliable source of feed water of proper quality and nominal quantity to the desalination plant.

The problem of suspended matter in a seawater Intake used for SWRO plants, is one of the major problems for plants installed along seacoasts. The clogging of feed water flow by fishes, shells, weeds, algae, sewage, oil residue *etc*., causes a decline in the lifetime of membranes, lower feed and product water flows, and increase in pre-treatment equipment investments, higher energy consumption, and thus higher costs of the desalted water. Table 3.14 summarizes the problems threatening the performance of the SWRO.

The main Intake types are divided into two categories:

1. surface systems are based on installing one or two pipes on the sea bottom and pumping the feed seawater into the desalting plant
2. non-surface systems are based on pumping feed water from wells or other subsurface structures located close to the seacoast, where the dissolved salts concentrations are similar to seawater concentrations

Beach wells and other non-surface seawater Intakes, have a proven technique to replace the 'surface seawater Intake system'. The Beach well system has been used successfully with the minimum of pretreatment for SWRO plants in several locations around the world.

A Beach well system comprises of wells, well casing, screen, submersible pump, pump starters and interconnecting piping. A surface system usually includes one or two pipelines, or a channel to convey seawater to a pump pit, a coarse trash removal system to prevent the intrusion of fish and large objects and seawater transfer pumps. The main under-water works that are required for the surface system include, excavation, embedment, dredging, pipe laying, pipe anchoring, pipe joining and pit assembling. This report presents the considerations in analyzing the feasibility of the different types of Intakes, their advantages and disadvantages, and presents economical analysis of the types based on a model developed for this study.

5.2. Consideration for the comparison of non-surface Intakes to surface Intakes

The following considerations in selecting the type of Intake whether surface or non-surface can be summarized from the literature and experience collected for the present study.

a) The effects of sediments and suspended solids transport and settling may result in serious problems during pumping surface water. Settling can occur near the Intake point, changing the designed hydraulic conditions. This problem does not exist in non-surface Intakes

b) Various forms of pollution of surface seawater feed can cause severe damage in desalination plants. Chemical compounds discharged from chemical plants and refineries may cause contamination of product water as well as corrosion problems on the metallic equipment. Organic pollution may be present in seawater arriving from wastewater treatment plants or from raw sewage. This pollution may clog the filters in the pretreatment area. Therefore, seawater currents regime investigation should be carried out before a final decision is made on the plant's site. The need of this investigation is eliminated in the non-surface system

c) 'Thermal pollution' of seawater may be harmful to the membranes in the SWRO plant. Power stations, chemical plants, and industrial processing plants may use large volumes of seawater for cooling water. The cooling water is returned to the sea and may be 8^0C to 15^0C higher than the ambient seawater. Because of the large volumes of cooling water, the discharge plume can reach long distances before the warm discharge stream is mixed and reduced to the ambient temperature of the sea. This problem is mainly relevant to the SWRO process because the membranes` stabilization is limited to higher temperatures (up to about 45^0C). This problem is not relevant to distillation processes, as preheating of feed water is an advantage. However, in SWRO plants, warmed feed water, up to the allowed temperature increases the flux through the membrane and may decrease energy consumption

d) The temperature of seawater changes between summer and winter and between day and night. In the Mediterranean Sea near the Israel coast, the temperature of seawater can vary between 18°C in winter up to 30°C in the summer. For the RO process, the temperature of feed water plays an important role in the performance of the plant, as well as on the lifetime of the membranes. Commercial RO systems are usually designed to operate at a constant flux rate, and therefore fluctuations of the feed water temperature are compensated for by adjustment of the feed pressure. Higher temperature of feed water may require a reduction of the operating pressure when in a constant flux and thus causing a lower compaction of the membranes. Each type of RO membrane has a certain temperature above which the production cannot be in a single stage desalination process. The rate of change of water flux through the membrane with temperature is approximately 3% per 1^oC. The changes of the temperature in non-surface Intakes will probably be significantly smaller than in the open sea. When pumping from the sea, the submerged inlet of the suction pipe should be located at depth so that changes in temperature will be minimized

e) In a surface system, breaking waves and currents create heavy mechanical loads and forces on the under water pipes and accessories. The structure should therefore be located at a sufficient depth below the water surface so that the disturbances resulting from wave forces can be minimized. A similar problem can exist due to tides. This situation is eliminated in non-surface systems

f) The chemical composition of seawater is usually stable. Under steady-state conditions, groundwater provides a chemically stable source of water over a long time. However, in pumping wells adjacent to the coast, groundwater quality can vary over a long time (Section 3.1.1 and 6.1.5). The SWRO plant should therefore be designed to accommodate the changes in the chemical composition of the feed water

g) The seabed and coastal structure and conditions are primary factors in determining the type of surface Intake structure. The Intake can draw the feed water from the open sea or from a sheltered lagoon using either submerged pipe(s) or channel. For a non-surface system based on pumping wells, the characteristics and the permeability of the underground formations are the important factors in designing the pumping system

h) The electrical system and controls are more complicated in the non-surface well pumping arrangement as it serves several pumps, whereas, in the surface pumping method, only one or two pumps are in service

i) The onshore piping between the wells supplying feed water to the desalination plant is composed of many pipes, joints, pumps, headers *etc*. The piping of the surface system is simpler, as the RO plant is connected directly to the seawater feed pipe

j) In case of failure of one or more non-surface wells, the desalination plant can continue to produce most of the water required because most of the other wells remain operating. In surface systems, however, shut-down of the whole plant may result when the single Intake pipe fails

k) The volume of seawater pumped for large distillation plants is approximately 2.5 times more than the nominal product volumes. Most of the water is required for cooling/preheating of feed/product water. Therefore, large distillation plants are often based on surface Intake systems because too many wells might be needed for the non-surface option

l) Untreated surface seawater may have a SDI value of approximately 4.6 – 6.7 (Hasan 1991). The conventional pretreatment for SWRO plants reduces the SDI value to about 2. (Wilf 1997, Hasan 1997). SDI values of 1 to 2 can be achieved by using Beach well Intakes with a reduced pretreatment compared to surface Intakes. Low values of SDI in the feed water will increase the flux through the membranes, improve the recovery ratio of the process, and therefore save the total area of membranes required

m) In spite of the advantages, non-surface Intakes may be rejected if the local hydrogeological conditions do not enable sufficient groundwater flows (transmissivity below 1,000 m^3 per day) or do not enable natural filtration (fissured or cavernous rock)

n) The main advantage of using a non-surface Intake is the ‘natural’ filtration of the feed water to SWRO plants in sand formations, frequently existing near the seacoast. However, non-surface Intakes have advantages with a coarse granular ground formation with poor filtration. These advantages include: avoiding under-water piping installations, reduction of marine growth effects, temperature stabilization, prevention of sedimentation in inlet piping and the flexibility of operating several wells in comparison to being dependent on one Intake pipe, that might fail

5.3. Investment and estimated costs savings using non-surface systems

The feed water that is supplied in a non-surface system from underground abstraction facilities such as pumping wells is pre-filtered naturally in the small pores of the underground formation and in the gravel and sand pack, which surrounds the borehole. The reduced biological activity results in saving by investmenting in pretreatment plant and operation costs of chemicals, and minimizes the potential risk of bio fouling in the whole system. The lower bio-fouling activity may eliminate the need for pre-chlorination or using small quantities of chlorine for disinfection. This situation does not exist in pumping surface seawater.

The usage of a Beach well seawater Intake system probably reduces the capital costs of the Intake and pretreatment systems, and adds to the lifetime of the membranes, lower operation expenses and manpower, and reduces the amount of consumed chemicals. The saving in pretreatment filtration equipment can also save the total area needed for the plant, thus saving in the total investment.

The following figures are estimated for SWRO plants, which produce 10 million m^3 per year and 30,000 m^3 per day:

a) Estimated saving in investment of the pretreatment equipment section is 10 – 12% of the estimated investment for the pre-treatment system, which is 150 \$ per m^3 per day (for a 30,000 m^3 per day plant) *i.e.* the estimated saving is 15 \$ per m^3 per day

b) Estimated saving in manpower is 1 – 3 operators, *i.e.* 40,000-120,000 \$ per year

c) Estimated saving in area required for pretreatment plant is approximately 5,000 m^2. In Israel's densely populated coastal areas, land values are approximately US\$ 100 per m^2

d) Saving in chemical consumption in SWRO plants is based on the specific concentration of the suspended matter and dissolved salts in the feed water on the site. For estimation purposes, the saving in chemical cost between surface and non-surface systems can be in the order of 3 to 5 times, see Tables 7.1 and 7.2

e) When feed water is pumped from coastal wells, the total salinity of the water might be lower than the salinity of nearby seawater. Because energy consumption depends on the salinity of feed water, it was found that, generally, for each 1% reduction in concentration of dissolved salts in feed water the consumed energy is reduced by 1%

5.4. Comparison between distillation and SWRO processes in regard to feed water quality

a) The pretreatment filtration size of feed water for the SWRO process is 5 microns, whereas, the filtration size for distillation process is about 25 microns. Thus, distillation processes are less sensitive to the filtration level of the feed water, and the advantage of non-surface Intakes diminishes

b) A similar situation refers to values of SDI (Silt Density Index) of feed water. Distillation processes are less sensitive to SDI high values than SWRO process and thus require less stringent pre-treatment, lower investment and operation costs, see Paragraph 5.2. l (above)

c) Distillation processes, as indicated above, are not sensitive to thermal variations and thermal pollution of the feed water

5.5. Economic comparison of surface and non-surface Intakes

Designs of non-surface Intakes are detailed in two recent papers *i.e.* TAHAL (1997) and Wright and Missier (1997), which include preliminary designs and cost evaluation of Beach wells, galleries and seabed filtration.

The second study shows, see Table 5.1, that the cost of Beach wells is significantly smaller than surface Intakes for plants in the range of 2,000 - 30,000 m^3 per day. The second best are horizontal galleries. Seabed filters are more expensive than surface Intakes for some of the lower capacities and they have the highest overhead and management costs. Mercado and Kaly, 1996, recently estimated the reduction of desalination costs in Beach well Intakes compared to surface Intakes for a 50 MCM per year plant. This estimate was based on the assumptions and calculations shown in Table 5.2. The reduction of costs is estimated at 13%.

Table 5.1: Cost comparisons of Intakes serving R. O. desalination plants (Wright and Missier, 1997)

Intake System	Water Supply System Capacity, m^3 per day				
	2,000	**4,000**	**7,500**	**15,000**	**30,000**
Beach wells					
Capital cost unit	1.00	1.00	1.00	1.00	1.00
O&M cost unit	1.00	1.00	1.00	1.00	1.00
Surface Water					
Capital cost ratio	1.99	1.92	1.81	1.67	1.68
O&M cost ratio	2.00	1.29	1.14	1.27	1.21
Seabed Filter					
Capital cost ratio	2.30	1.99	1.74	1.35	1.17
O&M cost ratio	2.13	1.33	1.19	1.31	1.28
Horizontal Beach Gallery					
Capital cost ratio	1.14	1.16	1.18	1.18	1.19
O&M cost ratio	1.00	1.00	1.00	1.00	1.00

Abdel Jawad and S. Ebrahim (1994), describe an experimental plant of two 20 $l.min^{-1}$ RO installations operated for one and a half years. One with a conventional seawater Intake and pretreatment and the other with a Beach well Intake filtered through 20 microns cartridge filters without any further pretreatment. Comparisons are shown of feed water quality (SDI, temperature, pH, conductivity, dissolved organics), performance (recovery percentage, salt rejection, product TDS) and cost. The authors concluded that the product water cost of the RO system fed with Beach well water is 30% less than that of an identical system fed with conventionally pretreated surface water. The RO system fed with Beach well water had higher recovery and salt rejection. Heyden (1985) describes the design and operation of an RO plant with Beach wells and summarizes their advantages and disadvantages (Chapter 3.10).

The suitability of Intake types of process and site conditions is to be decided by an economic comparison. A spreadsheet for such a comparison was developed in the present study (see Chapters 6.2, 7.3 and Appendices C and D).

Table 5.2: SWRO cost estimates – Plant only
(Mercado and Kally, 1996)

	Item	**Surface Intake**		**Beach wells**	
		7% Interest	**12% Interest**	**7% Interest**	**12% Interest**
Investments (10^6 US$)	Desalination	30.0		28.8	
	Ancillaries	75.0		72.0	
	Site Development	0.5		0.4	
	Intake	20.0			
	Brine Disposal	12.5		12.0	
	Interest during Construction	8.7	14.9	6.2	10.6
	Administration and Design	33.7		27.5	
	Unaccounted for 10%	18.0		20.5	
	Total	198.4	225.1	161.6	182.6
Annual Costs (10^6 US$)	Capital Costs (CRF, 25 years)	17.0	28.0	13.9	23.3
	Electricity 6 cents per kWh 5.5 $kWh.m^{-3}$ 5.3 $kWh.m^{-3}$	16.5		15.9	
	Chemicals	8.9		7.2	
	Membrane Replacement	3.1		3.0	
	O&M (not including Bench Wells)	6.7		5.4	
	Miscellaneous	3.0		2.8	
	Total Annual Costs	55.2	66.2	48.2	57.6
Cost of Water (50 MCM per year)		**1.10**	**1.32**	**0.96**	**1.15**

5.6. Environmental aspects

Environmental aspects of Beach wells are mainly related to their impact on the inland groundwater. The use of Beach wells for desalination may act as a barrier for seawater intrusion, (compare Figures 6.6 and 6.7). Pumping in deep wells penetrating beneath the interface between seawater intruding into a fresh water aquifer may contribute to the management of the aquifer. This activity reduces pressures in the underground saline water and controls its intrusion. This may enable an increase of fresh water abstraction. This process of aquifer management was rejected in the past due to high pumping cost and loss of the fresh water fraction that is inevitably mixed in the pumped seawater (Mercado and Kally, 1996).

Mercado and Kally (1996) indicate that the fraction of fresh water pumped in Beach wells may be the main disadvantage of implementing the Beach well approach.

To minimize the freshwater fraction in the pumped water, the following design criteria should be followed:

- Land wells should be located as near as possible to the coast line and seabed wells should be located as far as possible from the coast line
- Wells should penetrate the full depth of the aquifer and reach its base
- The well screen should be as short as possible with the maximum percentage of perforated area

A further study of this issue is presented in section 6.1.

5.7. Selection of Intake type and process

In summing up the issues discussed above, the main factors that may affect the selection of an Intake type and desalination process were analyzed and are shown in Table 5.3. This table indicates by + sign the types of Intakes (and types of desalination process) that become more attractive when the factor in the first column increases. The opposite situation exists for the - sign. Blank means non-sensitivity to the factor. These are preliminary considerations to enable preliminary rejection of alternatives. The final decision is by an economic comparison.

Beach wells will be generally preferred under the following conditions:

1. Water abstraction takes place in a sea area that may be polluted by micro-organics, soil particles (high turbidity) or other pollutants that can be trapped (cleaned up) by the soil layer surrounding the Beach well
2. The marine conditions are unstable due to high currents, storms and extreme tides that may endanger marine feed water pipelines, or cause sand accumulation at the inlet
3. Other structures such as jetties, pipelines and buoys, may interfere with the feed water pipeline
4. Adverse impacts can be expected on sensitive marine ecosystems, on recreation and tourist sites or on cultural/archaeological sites
5. Hydraulic constraints of groundwater flow are within plant capacity
6. Salinity in Beach wells is lower than in seawater

7. Total cost is higher than the Beach well option

Beach wells will be rejected under the following conditions:

1. The hydrogeologic conditions do not facilitate or allow the implementation of this approach. If the transmissivity is low (below 100,000 $m^3.y^{-1}.m^{-1}$ or below 300 $m^3.d^{-1}$ m^{-1}), then the productivity of the well will be very low
2. If the depth of the aquifer is greater than 15 m, the construction and operation costs may be high and thus the economic viability of the well will decrease
3. Lithology of the area determines the Beach wells efficiency since if the soil layer does not consist of well-graded material the pre-filtration will be limited (sandy soils are preferred than rock or cobble formations or boulders)
4. Groundwater is contaminated by fuels, chemicals, *etc.*
5. Large distance between well site and desalination plant
6. Well sites are in conflict with other land uses such as: dwellings, conserved landscape, archaeological sites, *etc.*
7. Total cost is higher than other alternatives

Table 5.3: Considerations in selecting the type of Intake for RO seawater desalination and in selecting the desalination process

Increasing factor	Priority of Intake types increases as factor increase					Priority to selecting RO process increases
	Surface	Beach wells	Horizontal gallery	Seabed filtration	Off-Shore non-surface	
Total discharge	+	-			+	-
Sea waves and currents	-	+			-	
Sea impurities	-	+	+	+	+	
Sea thermal variations or pollution	-	+	+	-	-	-
High yielding aquifer inland	+	-	-	+	+	
High salinity of inland GW	+	-	-	-	+	
Thin aquifer	+	-	+	+		
Low hydraulic permeability of the subsurface formations	+	-	-		-	
Fissured and cavernous rock	+	-	-		-	
Deep sea bottom	+	+	+	-	-	
Abstraction of seawater reduces interface intrusion	-	+				
Lower desalination cost with lower feed water salinity	-	+	+	-	-	+

+ Priority increased when factor increases
- Priority decreases when factor decreases
[Blank] Insensitive to change of factor

6. SIMULATION AND DESIGN METHODS

Three types of models have been used in the present study:

1. A geohydrological model to represent the performance of Intakes in terms of quantity and salinity of the pumped water and their variations over time as well as the impact of the Intakes on the groundwater inland and the reduction of inland availability of groundwater
2. A techno-economic model, which represents the cost elements of the total system and delivery/disposal of product water and of the brine produced. This model also includes an assessment of the indirect costs, based on the geohydrological model
3. Models for estimating energy requirements of desalination based on the TDS and chemical composition of the feed water and the recovery ratio

The first model is discussed in the present chapter with the background in Appendix B. The second is mainly discussed in Chapter 7 and Appendix C. The third is usually part of the information furnished by membrane producers and suppliers, and is shown in Appendix E.

6.1. Modelling the dynamics of seawater intrusion into coastal aquifers

6.1.1. The Model

The principles of modelling and the selection of the SUTRA model for the present study are discussed and summarized in Appendix B. The model simulates flows and concentrations with varying salinity and density, and their variation over time in coastal Aquifers.

This model was applied with the assistance of KTH in Stockholm, to a simplified model of the Nitzanim area in Israel.

6.1.2. The SUTRA Model

SUTRA is a finite-element simulation model for saturated-unsaturated, fluid-density-dependent ground-water flow with chemically reactive single-species solute transport. SUTRA may be employed for a real and cross-sectional modelling of saturated ground-water flow systems, and for cross-sectional modelling of unsaturated zone flow. Solute transport simulation using SUTRA may be employed to model natural or man-induced chemical species transport including processes of solute sorption, production and decay, and may be applied to analyze ground-water contaminant transport problems and aquifer restoration designs. In addition, solute transport simulation with SUTRA may be used for the modelling of variable density leachate movement, and for cross-sectional modelling of saltwater intrusion in aquifers in near-well or regional scales, with either dispersed or relatively sharp transition zones between freshwater and saltwater.

The model employs a two-dimensional hybrid finite-element and integrated-finite-difference method to approximate the governing equations that describe the two interdependent processes that are simulated:

(1) fluid density-dependent ground-water flow
(2) transport of a solute in the ground water in which the solute may be subject to equilibrium adsorption on the porous matrix, and both first-order and zero-order production or decay

The model was developed by the USGS and documented in Voss, C.I., 1984. A finite-element simulation model for saturated-unsaturated, fluid-density-dependent ground-water flow with energy transport or chemically reactive single-species solute transport: U.S. Geological Survey Water-Resources Investigations Report 84-4369, p 409.

Souza, W.R., (1987), Documentation of a graphical display program for the saturated-unsaturated transport (SUTRA) finite-element simulation model: U.S. Geological Survey Water-Resources Investigations Report 87-4245, p 122.

A summary of SUTRA may be found on web page:
http://water.usgs.gov/cgi-bin/man.wrdapp?sutra.

An official version of U.S. Geological Survey software is available for electronic retrieval *via* the World Wide Web (WWW) at:
http://waterusgs.gov/software/sutra.html

6.1.3. Aquifer data

The data for constructing the model representing the conditions in the Nitzanim area of the Israel coastal plain, are as follows:

Length of modelled section perpendicular to shoreline	:	3,500 m
Width of section	:	1 m
Depth of bottom, amsl	:	- 120 m
Net natural recharge	:	0.2 $m.yr^{-1}$
Constant head at inland boundary	:	h_o = 3.5 m
Slope of sea bottom	:	1:25
Hydraulic conductivity	:	1.75×10^{-4} $m.s^{-1}$
Specific yield (storativity)	:	0.36
Longitudinal dispersivity	:	10 m
Transverse dispersivity	:	1.5 m
Molecular diffusivity of solute in water	:	1×10^{-9} $m^2.s^{-1}$
Freshwater density	:	998.2 $kg.m^{-3}$
Seawater density	:	1,024.4 $kg.m^{-3}$
Seawater concentration	:	35,000 ppm TDS
Freshwater concentration	:	100 ppm TDS

6.1.4. Tests performed

Long term constant pumping in a single well was tested in the above-defined coastal section. The variables tested were: distance from shoreline, rate of pumping and depth of well. A reference test was performed without pumping to present initial steady state conditions. The results are presented by the following dependent variables:

- Streamline pattern
- Iso-concentration lines
- Evolution of salinity in the pumped water
- Evolution of flows across boundaries

The design combinations are summarized in Table 6.1.

Table 6.1: Design combinations tested with the SUTRA model (no. of test)

Rate of Pumping m^2 per year	**Distance from coastline (m)**				
	0	**25**	**50**	**100**	**250**
500	32*	27*	22*	17*	12*
1,000	33*	28*	23*	18*	13*
2,000	34*	29*	24*	19*	14*
5,000	35*	30*	25*	20*	15*
10,000	36*	31*	26*	21*	16*
0	37				

* Depth of pumping -100 to -120 m.
Note: Discharges are given for 1 m width, *i.e.* $m^3.yr^{-1}.m^{-1}$

6.1.5. Results

A summary of typical results of the above tests are shown in Table 6.2 and the following charts.

Figure 6.1 shows the streamlines in the initial conditions.
Figure 6.2 shows the initial TDS concentration lines.

Figure 6.3 shows the streamlines after 100 years for a pumping rate of 500 m^2 at a distance of 250 m from the coastline.

Figures 6.4, 6.5, show similarly the streamlines for the distance 0 and discharge rates of 500 and 10,000 m^2 per year.

Figures 6.6, 6.7, show the TDS concentration lines for the same tests namely distance 0 and discharge rates of 50 and 10,000 m^2 per year.

The results are summarized in Tables 6.2, 6.3, 6.4 and Figures 6.10, 6.11, 6.12, 6.13, 6.14 and are analyzed in the next section.

Table 6.2: Results of SUTRA simulation

Input					Graphic results			Inflows	Flow across the inland boundary after 140 y		
Test	Qp ($m^2.y^{-1}$)	X (m)	Y (m)		Approx: fraction of fresh water streamlines entering the well (%)	Salinity of pumped water – TDS (ppm)	Fraction of fresh water in the pumped water (%)	NR–QP ($m^2.y^{-1}$)	Freshwater outflow (TDS = 100 ppm) across the border ($m^2.y^{-1}$)	Freshwater inflow (TDS = 100 ppm) across the border ($m^2.y^{-1}$)	Net freshwater inflow (TDS = 100 ppm) across the border ($m^2.y^{-1}$)
12	500	250	–100	–120	0	33,000	5.7	206.6	2.5	81.1	78.6
13	1,000	250	–100	–120	10	30,800	12.0	–293.2	0.7	122.4	121.7
14	2,000	250	–100	–120	62	25,800	26.4	–1,293.0	0.0	203.8	203.8
15	5,000	250	–100	–120	100	25,800	26.4	–4,292.0	0.0	425.6	425.6
16	10,000	250	–100	–120	100	29,300	16.3	–9,293.7	0.0	835.3	835.3
17	500	100	–100	–120	0	34,800	0.6	206.6	3.3	62.2	58.9
18	1,000	100	–100	–120	0	34,200	2.3	–293.2	2.3	85.9	83.6
19	2,000	100	–100	–120	0	33,500	4.3	–1,293.0	0.3	132.1	131.8
20	5,000	100	–100	–120	70	30,900	11.7	–4,292.0	0.0	263.8	263.8
21	10,000	100	–100	–120	100	28,050	19.9	–9,293.7	0.0	431.9	431.9
22	500	50	–100	–120	0	35,000	0.0	206.6	3.5	55.9	52.4
23	1,000	50	–100	–120	0	35,000	0.0	–293.2	2.8	73.3	70.5
24	2,000	50	–100	–120	0	34,600	1.1	–1,293.0	1.3	107.9	106.6
25	5,000	50	–100	–120	57	32,050	8.5	–4,292.0	0.0	210.7	210.7
26	10,000	50	–100	–120	100	31,200	10.9	–9,293.7	0.0	321.5	321.5
27	500	25	–100	–120	0	35,000	0.0	206.6	3.7	52.9	49.2
28	1,000	25	–100	–120	0	35,000	0.0	–293.2	3.0	67.3	64.3
29	2,000	25	–100	–120	0	34,700	0.9	–1,293.0	1.8	96.1	94.3

Table 6.2 (continued)

30	5,000	25	–100	–120	40	33,300	4.9	–4,292.0	0.0	182.7	182.7
31	10,000	25	–100	–120		31,600	9.7	–9,293.7	0.0	285.1	285.1
32	500	0	–100	–120	0	35,000	0.0	206.6	4.0	50.2	46.2
33	1,000	0	–100	–120	0	35,000	0.0	–293.2	3.3	61.7	58.4
34	2,000	0	–100	–120	0	34,800	0.6	–1,293.0	2.3	85.0	82.7
35	5,000	0	–100	–120	40	34,000	2.9	–4,292.0	0.0	153.9	153.9
36	10,000	0	–100	–120	95	31,800	9.2	–9,293.7	0.0	264.0	264.0
37								706.4	5.5	39.5	34.0

Table 6.2 (continued)

Flow across the seawater boundary after 140 y					Outflows		
Test	Total discharge to the sea ($m^2.y^{-1}$)	Total discharge of freshwater (TDS<1000 ppm) to the sea ($m^2.y^{-1}$)	Discharged water with salinity different from 35000 ppm ($m^2.y^{-1}$)	Salinity (ppm) of the discharged water different from 35000 ppm	Seawater inflow across the sea boundary ($m^2.y^{-1}$)	Net flow across the seawater boundary ($m^2.y^{-1}$) Negative = outflow	Balance between flows across boundaries ($m^2.y^{-1}$)
12	1,243.5	0.0	1,220.5	13,080	958.2	–285.2	–206.6
13	1,116.1	0.0	1,093.6	11,693	1,287.7	171.6	293.2
14	718.8	0.0	696.7	11,000	1,808.0	1,089.2	1,293.0
15	21.8	0.0	0.0	–	3,888.3	3,866.5	4,292.0
16	20.5	0.0	0.0	–	8,478.9	8,458.4	9,293.7
17	1,258.1	0.0	1,235.2	13,350	992.6	–265.5	–206.5
18	1,203.0	0.0	1,180.7	11,825	1,412.7	209.7	293.3
19	1,066.3	0.0	1,044.5	9,236	2,227.5	1,161.2	1,293.0
20	200.1	0.0	179.5	4,919	4,228.4	4,028.3	4,292.0
21	20.0	0.0	0.0	–	8,881.8	8,861.8	9,293.7

Table 6.2 (continued)

22	1,259.9	0.0	1,237.0	13,546	1,001.0	–258.9	–206.5
23	1,210.0	0.0	1,187.7	12,189	1,432.8	222.8	293.3
24	1,112.2	0.0	1,090.5	9,770	2,298.6	1,186.4	1,293.0
25	684.0	0.0	663.8	4,906	4,765.4	4,081.4	4,292.0
26	19.1	0.0	0.0	–	8,991.2	8,972.1	9,293.7
27	1,262.4	0.0	1,239.5	13,676	1,006.6	–255.7	–206.5
28	1,216.1	0.0	1,193.8	12,445	1,445.1	229.0	293.3
29	1,127.1	0.0	1,105.4	10,263	2,325.8	1,198.7	1,293.0
30	813.6	0.0	793.6	5,123	4,922.9	4,109.3	4,292.0
31	18.1	0.0	0.0	–	9,026.7	9,008.6	9,293.7
32	1,266.6	0.0	1,243.7	13,832	1,013.9	–252.7	–206.5
33	1,224.9	0.0	1,202.7	12,758	1,459.8	234.9	293.3
34	1,149.3	0.0	1,127.7	10,831	2,359.5	1,210.3	1,293.0
35	869.9	0.0	850.2	6,235	5,008.0	4,138.1	4,292.0
36	71.7	0.0	55.2	2,508	9,101.4	9,029.7	9,293.7
37	1,319.2	0.0	1,295.4	15,002	578.8	740.4	706.4
Fresh water salinity 100							
Seawater salinity 35,000							
NR ($m^2.y^{-1}$) 714							

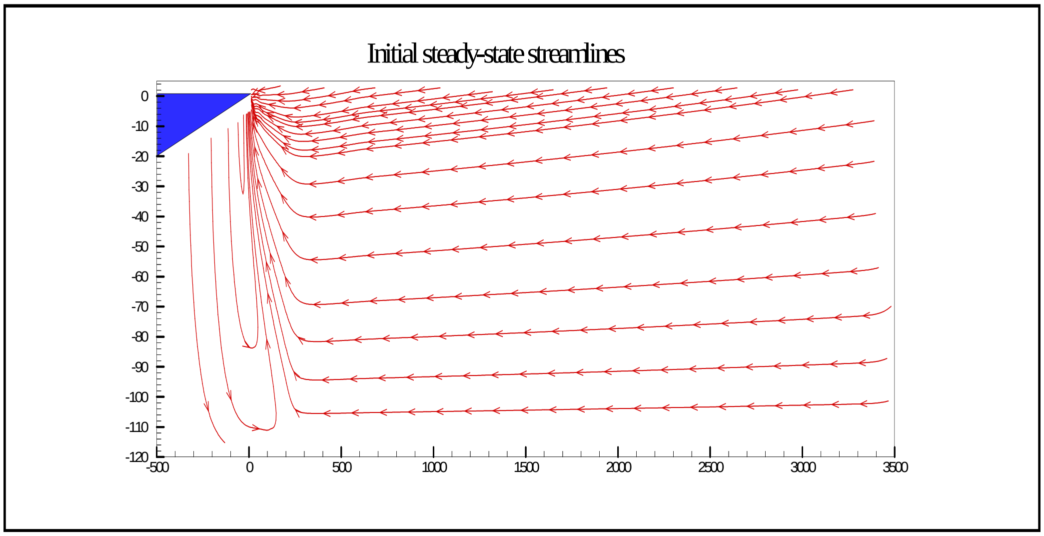


Figure 6.1: Initial steady state streamlines

The evolution over time of salinity in the pumped water, see Figure 6.8 for the distance of 250 m and Figure 6.9 for the distance of 0 m.

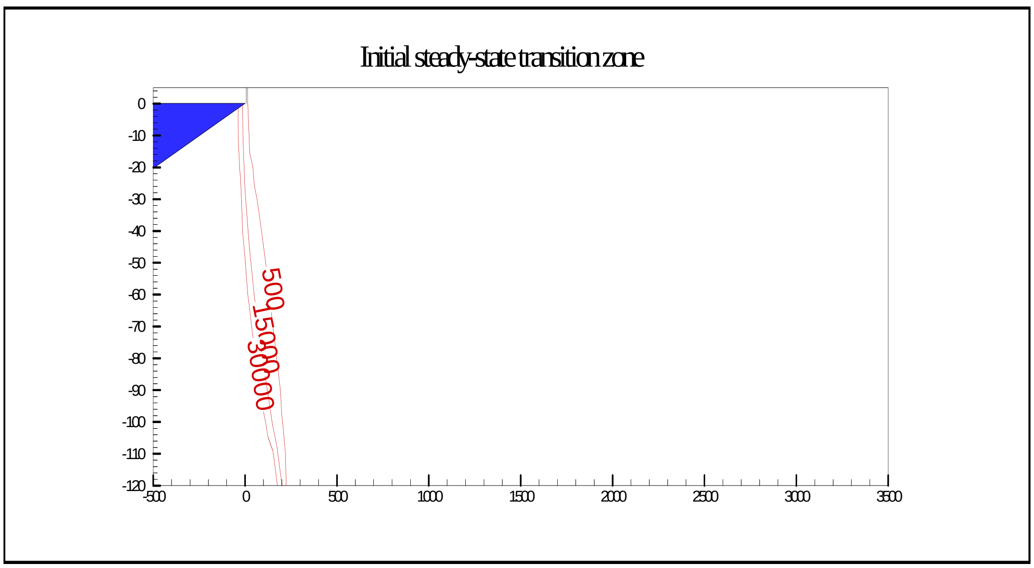


Figure 6.2: Initial TDS concentration lines

X-Axis – Distance from coastline (m); Y-Axis – Depth from mean sea level

The impact of pumping at various distances and pumping rates on the total discharge to the sea, see Figure 6.10. The impact of pumping at various distances and pumping rates on the total freshwater inflow across the inland boundary, see Figures 6.11, 6.12

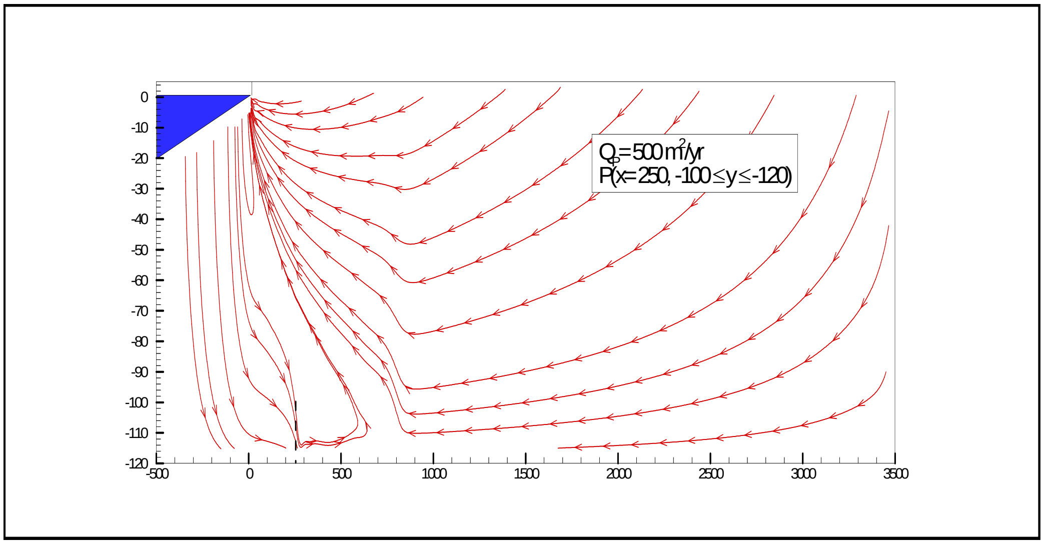


Figure 6.3: Streamlines after 100 years. - Distance: 250 m
Pumping 500 m^2 per year

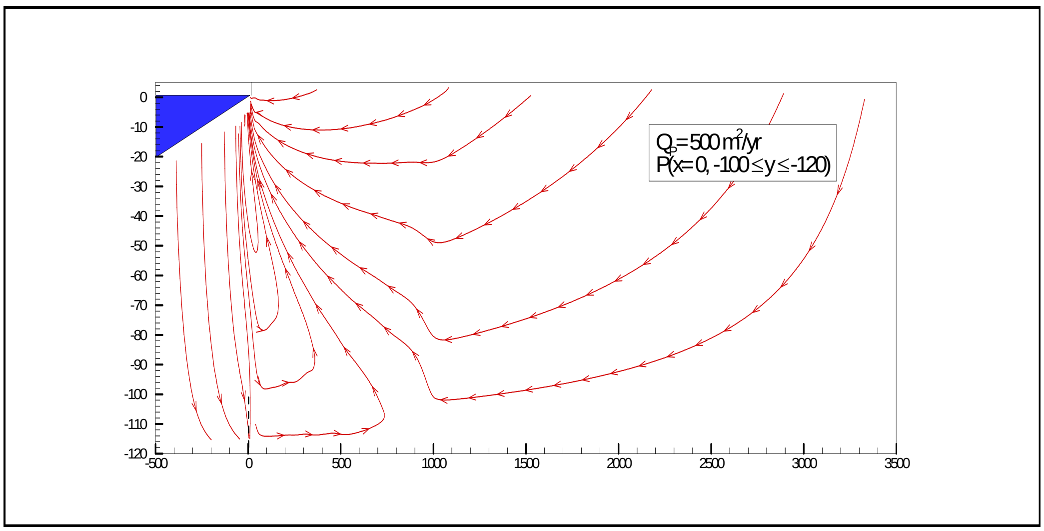


Figure 6.4: Streamlines after 100 years. - Distance: 0 m
Pumping 500 m^2 per year

X-Axis – Distance from coastline (m); Y-Axis – Depth from mean sea level

6.1.6. Analysis of results

Three hydrogeological parameters are required for cost estimates by the model developed for the assessment of Intake direct and indirect costs (Chapter 7.2 and Appendix C):

a) Single well capacity *i.e.* capacity per unit length of drain/gallery. These parameters are discussed in Chapter 9

b) Salinity of pumped water

c) Induced flow from the Hinterland as a fraction of pumped water. The cost of desalination is to be penalized accordingly

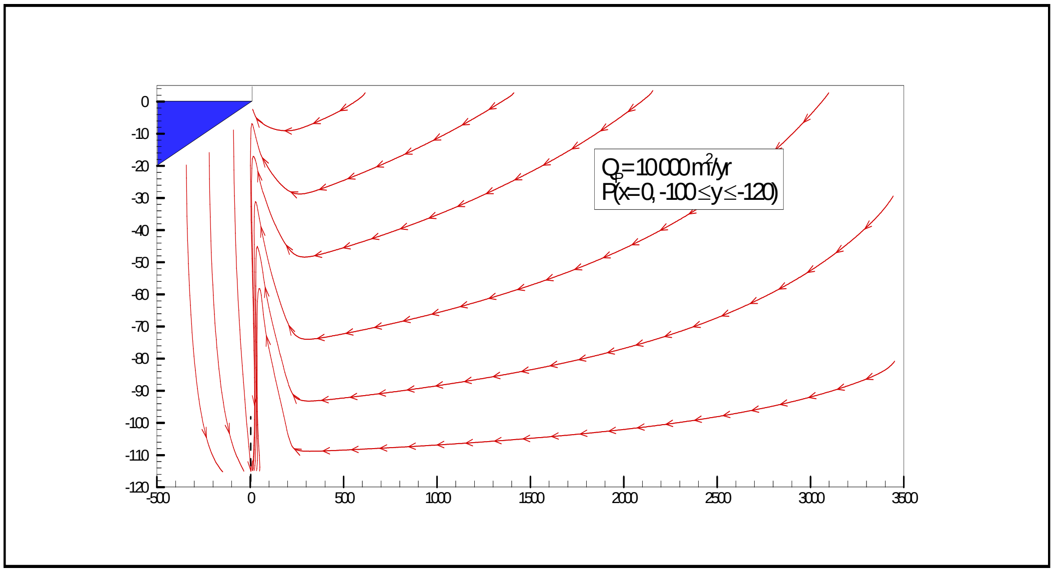


Figure 6.5: Streamlines after 100 years - Distance: 0 m
Pumping 10,000 m^2 per year

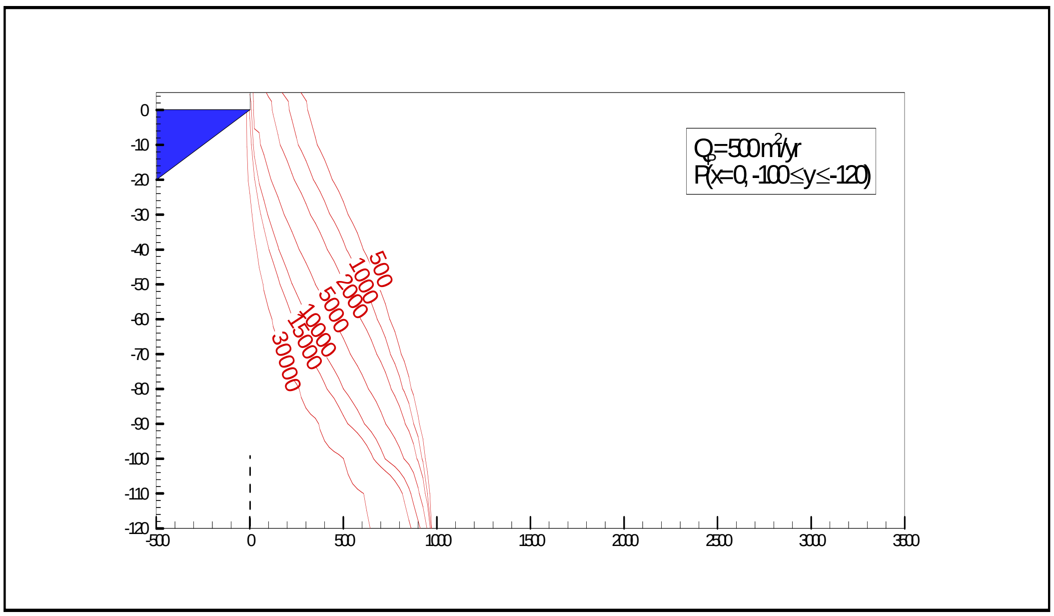


Figure 6.6: TDS Concentration lines after 100 years - Distance: 0 m
Pumping 500 m^2 per year

X-Axis – Distance from coastline (m); Y-Axis – Depth from mean sea level

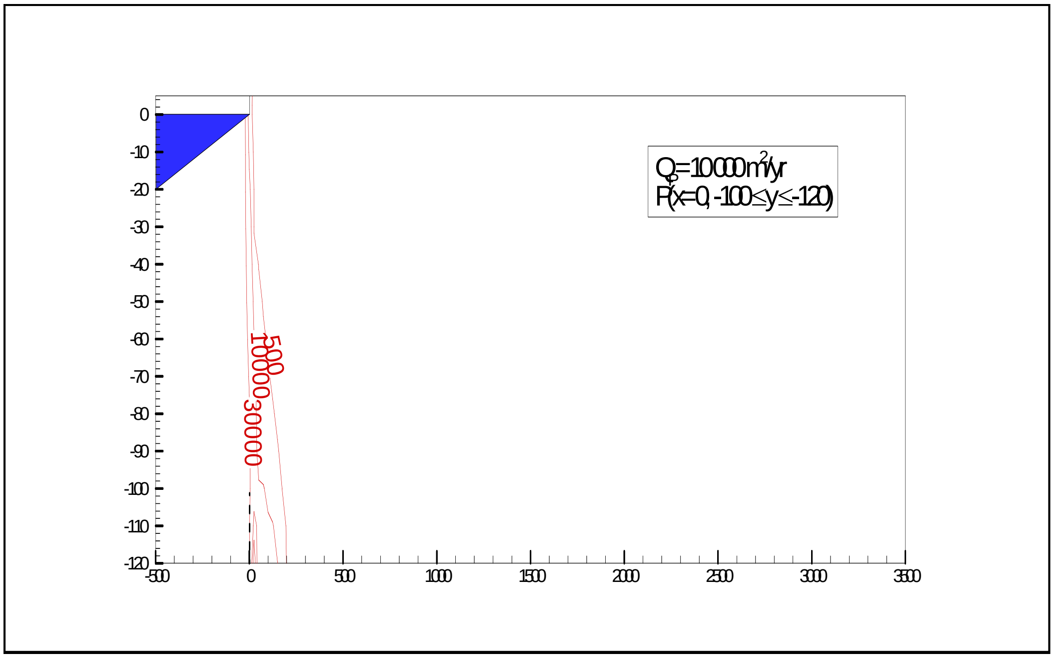


Figure 6.7: TDS Concentration lines after 100 years - Distance: 0 m
Pumping 10,000 m^2 per year

X-Axis – Distance from coastline (m); Y-Axis – Depth from mean sea level

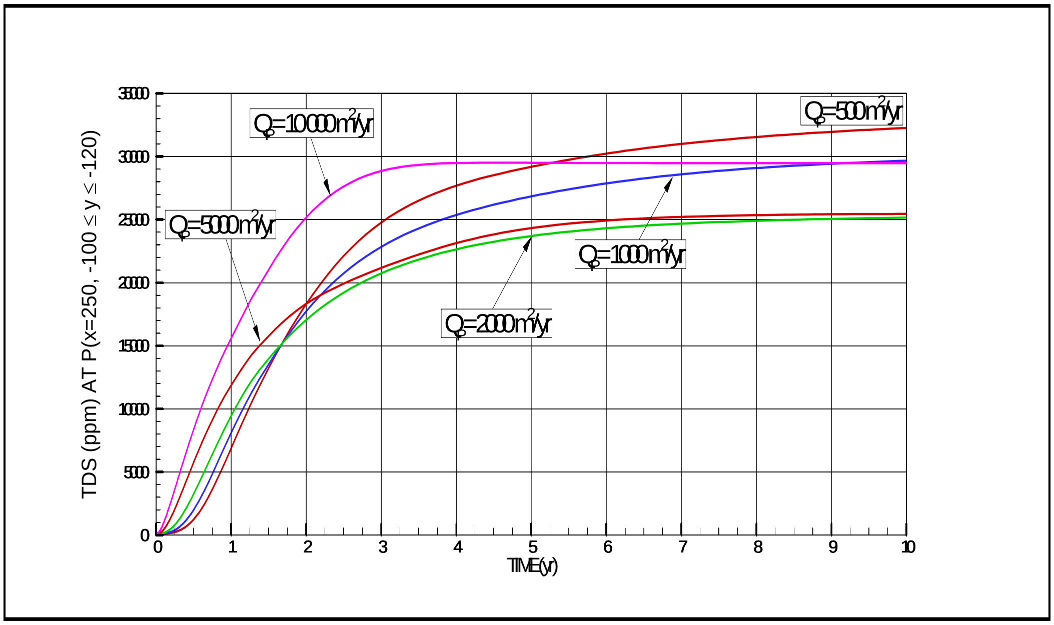


Figure 6.8: Evolution of salinity in the pumped water - Pumping at a distance of 250 m

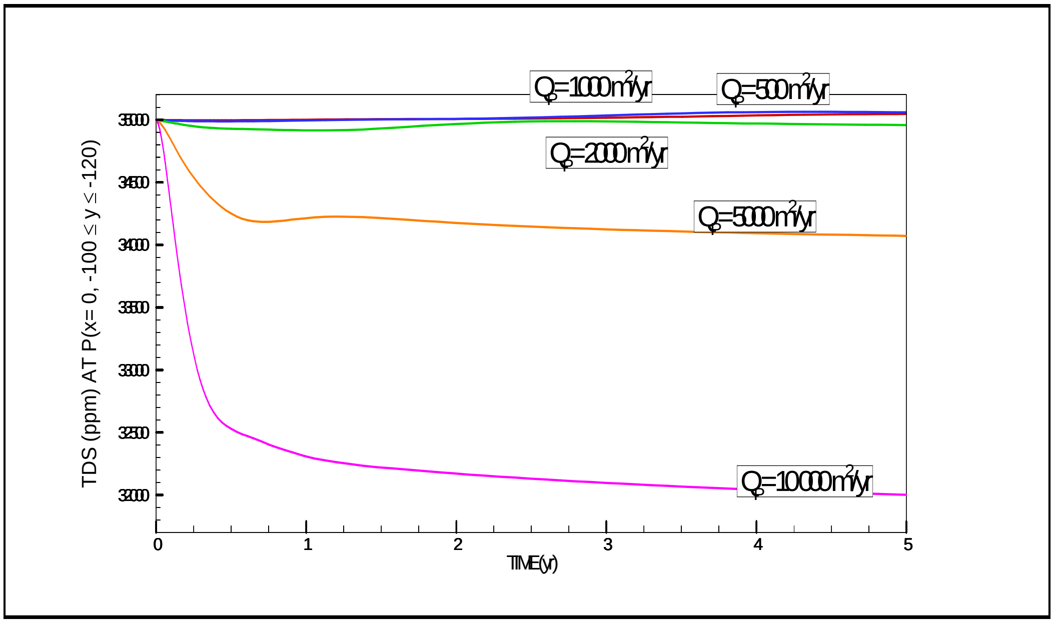


Figure 6.9: Evolution of salinity in the pumped water - Pumping at a distance of 0 m

The last two parameters can be evaluated as shown in Tables 6.3 and 6.4, which are based on Table 6.2. Figures 6.13 and 6.14, show the values at various distances from the sea and pumping rates for the depth of pumping -100 m -120 m in the Nitzanim area.

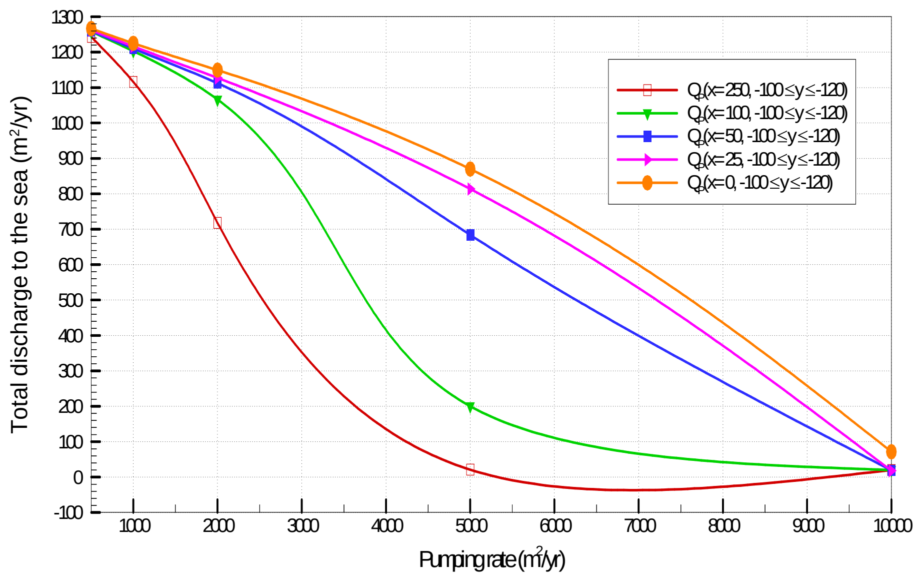
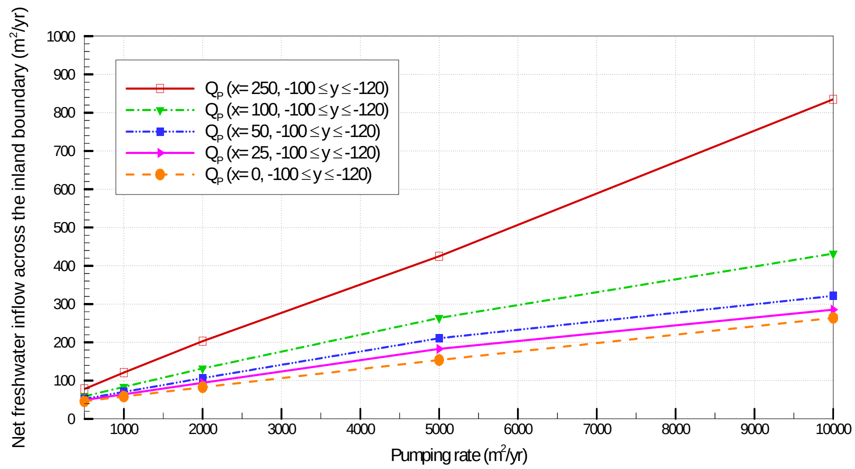


Figure 6.10: Total discharge to the sea



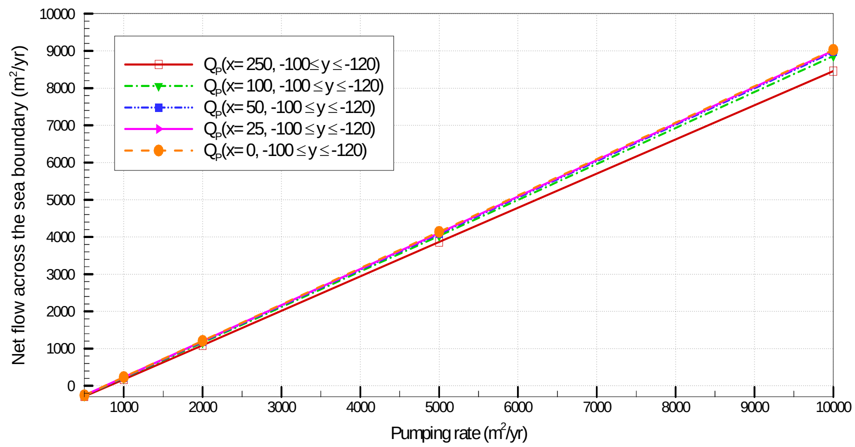


Figure 6.11: Net flow across the sea boundary
Figure 6.12: Net flow across the inland boundary

Table 6.3: Salinity of pumped water

Distance from Coast (m)	500 m^2 per yr	1,000 m^2 per yr	2,000 m^2 per yr	5,000 m^2 per yr	10,000 m^2 per yr
0	35,000	35,000	34,800	34,000	31,800
25	35,000	35,000	34,700	33,300	31,600
50	35,000	35,000	34,600	32,050	31,200
100	34,800	34,200	33,500	30,900	28,050
250	33,000	30,800	25,800	25,800	29,300

Table 6.4: Fraction of pumped water withdrawn from the Hinterland

Distance from Coast (m)	500 m^2 per yr	1000 m^2 per yr	2000 m^2 per yr	5000 m^2 per yr	10000 m^2 per yr
0	2.4%	2.4%	2.4%	2.4%	2.3%
25	3.0%	3.0%	3.0%	3.0%	2.5%
50	3.7%	3.6%	3.6%	3.5%	2.9%
100	5.0%	5.0%	4.9%	4.6%	4.0%
250	8.9%	8.8%	8.5%	7.8%	8.0%

From these results, it is clear that Beach well Intakes should be placed at a short distance from the sea at a maximum depth and at the highest possible pumping rate.

6.2. A model for cost estimates of desalination and Intake

A model for cost estimates was developed for the present study to enable economic comparisons among the various Intake alternatives and for the same alternative to compare different design layouts and dimensions. The model includes as input, all the variables in engineering, desalination technology, geohydrology, and finance related to: capacity, plant components, Intake components, hydrogeological characteristics, general financial parameters and unit costs. The outputs include comparison criteria such as: investments, annual capital costs, recurrent costs, (O&M, Energy and Chemicals) indirect costs and benefits.

Spreadsheets for cost estimates were prepared. These cost estimates reflect life cycle costs and include both capital and operation costs. The workbook was developed on MS-EXCEL and is described in Appendix C. Typical results are shown in Appendix D and Chapter 7.

6.3. Estimating energy requirement of desalination

The energy requirements and other design parameters of the RO desalination process depend on the properties of the membranes. Membrane producers and suppliers furnish spreadsheets for these computations. An example, based on Hydranautics spreadsheet, see Appendix E. Two different spreadsheets are shown, one for seawater salinity and the other for Beach well salinity. A decrease of 10% from seawater is assumed for Beach wells based on field observations and model results reflecting the Israeli conditions.

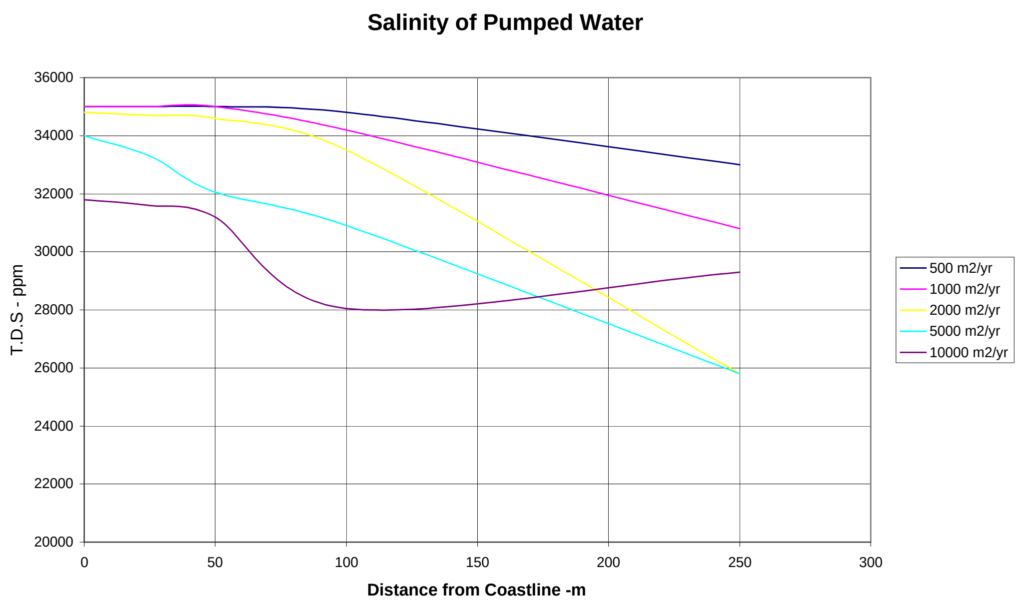


Figure 6.13: Salinity of water pumped at depths of -100m -120 m in the Nitzanim area

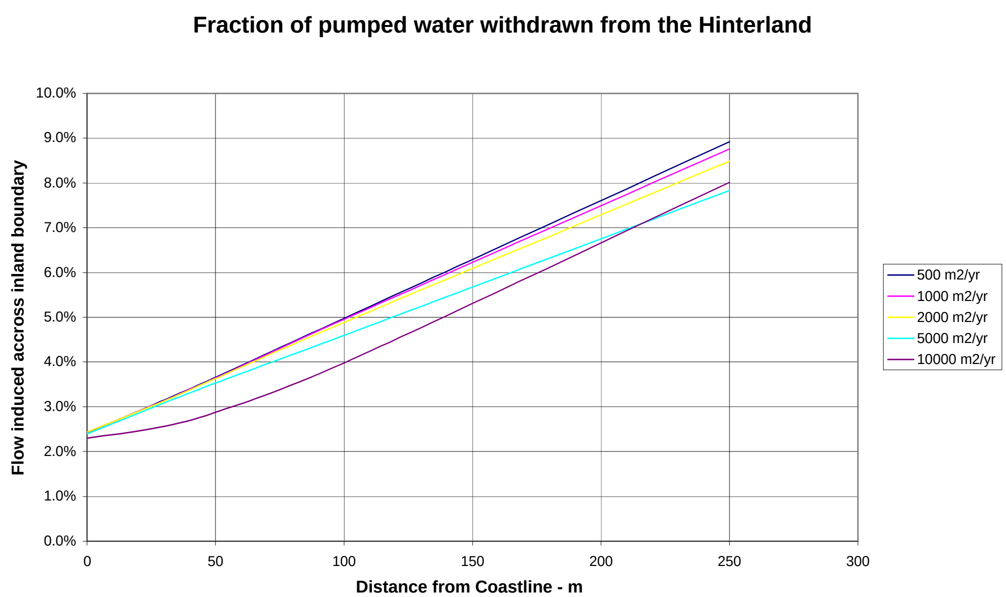


Figure 6.14: Groundwater flows from the Hinterland (distance of 3.5 km) as a fraction of water pumped in Beach wells from a depth of -100 m to -120 m in the Nitzanim area

7. COST ASSESSMENTS

Cost assessments in the present study are aimed at a preliminary comparison of Intake types and concerned with the following:

a) Unit cost estimates of the various components
b) Comparisons of cost of product water from the various Intake options for SWRO
c) Sensitivity analysis of these comparisons to hydrogeological, technological and financial components

The focus is on the application of the cost model presented in section 6.2 and Appendix C.

7.1. Cost factors in comparing Intake types

The following factors are reflected in the unit costs used in the cost spreadsheets:

7.1.1. Direct cost factors

The direct cost of feed-water is determined by site-specific physical cost factors. The main cost elements are:

a) Savings in feed water pretreatment, such as removal of suspended solids, organics and disinfection
b) Discharge capacity of a single well or of other Intake types and the number of wells
c) Spacing required between wells to avoid interference and excessive lowering of the water table and the length of the required collector
d) Depth and size of drilling/digging of the subsurface Intake structures
e) Lithological properties that determine unit cost of drilling/digging
f) Probability of failure
g) Length, size, materials of screens, casing pipes or other linings
h) Size and type of pumps and motors required for lifting the water
i) Length, size, and material of pipelines or other conduits needed for raw water conveyance from the groundwater source to the desalination plant
j) Energy required for water lifting and conveying
k) Expected life duration of the installations equipment with particular concern to corrosivity, which is high in seawater and may be higher in saline groundwater Intakes

The size and costs of the above elements depend both on site properties of the subsoil and seawater. The cost of the alternative surface seawater Intakes enjoys a high economy of scale. Their unit cost (per cubic metre) decreases with the increase of size. The preference of non-surface Intakes would therefore apply only for small size plants.

7.1.2. Indirect costs and environmental impacts

The major adverse environmental impacts to be considered with respect to SWRO desalination plants with non-surface Intakes are impacts on the inland fresh groundwater systems. Other environmental considerations, mostly related to brine disposal and plant nuisance are common to all desalination plants with all types of Intakes.

Two main impacts on the inland groundwater system are expected if pumping is maintained, inducing flows from the fresh groundwater body:

1. lowering groundwater tables within the usable inland fresh aquifer sections
2. loss of fresh groundwater by induced flow into seawater aquifers

The (1) and (2) above are governed by: the distance of the abstraction site from the coast line, the radius of influence (depending on the aquifer's transmissivity and storativity), the discharge rate, Intake depth, and the size and capacities of the inland aquifer. The quantitative evaluation of these impacts, see section 6.1.

If the inland water is completely utilized as in the water scarce MENA countries, then the indirect cost of producing additional water has to be considered. Otherwise, only direct costs have to be considered.

7.2. Cost estimates

Various Unit Cost Estimates have been prepared to be included in the cost estimates model, see Section 6.2 and Appendix C. Detailed unit costs were not available for most of the elements, therefore, costs for main plant components were taken instead.

The main effort was concentrated in identifying and evaluating the costs that vary among the various Intake methods and Intake designs.

No scientific references and no reliable published data are available for prices. Estimates are based on classified information from commercial sources. Prices should be adapted according to the local conditions for further applications of this model.

7.2.1. Land

The land required for a 40,000 m^3 per day product water plant is estimated as 22,500 m^2. TAHAL designed a plant of 30,000 m^2 per day with a land use of 13,000 m^2 (TAHAL, 1997). About 20% can be reduced when pumping from a subsurface system where part of the filtration system can be eliminated from the process due to lower turbidity of the feed water. The value of land in the densely inhabited coastal areas of Israel is estimated at 100$ per m^2.

7.2.2. Surface Intake

A 30,000 m^3 per day product water plant, designed by TAHAL (TAHAL, 1997), includes an Intake pipeline of 675 m length and a brine disposal pipeline of 450 m length both laid in the same trench. The cost of this pipeline was estimated at 6.5 million US$. An additional estimated 200 m pipeline length is considered to the plant in-land site, under regular soil and land use conditions at a unit cost of 260 US$. m^{-1}.

7.2.3. Pretreatment

The total investment in conventional pretreatment including raw water tank (about 1 hour of operation), low level pumps, coarse sand filters, treated water tank (1 hour of operation), cartridge filters and low level feed water pumps is estimated at 200 $\$.m^{-3}.d^{-1}$. A saving of 10% can be assumed when transforming from surface to subsurface Intakes mainly by eliminating the coarse sand filters.

7.2.4. Use and cost of chemicals

Table 7.1 presents an estimate of chemicals unit costs and doses for SWRO plants based on common experience and average values.

Table 7.1: Unit cost of chemicals and doses for the two SWRO Intake types

Treatment	**Chemical**	**Unit Cost NIS per kg**	**Dose $g.m^{-3}$**	
			Surface	**Subsurface**
Coagulant	$FeCl_3$	1.5	3	--
Antiscalant	H_2SO_4	0.75	45	10
Disinfection	Cl	1.0	4	2
Disinfection	NaOCl	0.8	2	--
Dechlorination	$NaHSO^4$	1.1		
pH Correction	$CaCO^3$	0.5	30	--
pH correction	NaOH	0.7	15	10

The dose is related to feed water volumes for the pretreatment components in the list. The dose in the last two post treatment components is for product water volumes. The above figures are an example for demonstration in the present study. Different quantities of chemicals may be used for other types of seawater and groundwater. Prices of chemicals are based on Israeli prices for May 2000.

7.2.5. Membranes

The investment in membranes is estimated as 60 $ per m^3 per day product water for seawater Intakes.

If the salinity of seawater is reduced by 10% because of mixing fresh groundwater, the pressure required can be reduced by about the same fraction and energy cost for desalination may be reduced accordingly, see Section 7.2.9.

7.2.6. Recovery ratio

The recovery ratio in SWRO plants is estimated as 45% - 50%. This ratio may be increased by approximately 0.5% by using less water for less frequent back washing of sand filters when operating with lower SDI values. Recovery ratios of 50% and 50.5% were assumed respectively for surface and non-surface Intakes.

7.2.7. Operation costs

The estimated saving in manpower by using a Beach well Intake is between 1 - 3 operators. However, this is reflected in the decrease of the O&M estimate, which in our model includes labour and is based on fixed percentages of the investment.

7.2.8. Plant availability

In case of failure of the single submarine pipe, the whole plant could be shut down, whereas, when one well is out of order, other wells can cover the shortfall in feed water.

In an open surface Intake system, there is a possibility that due to unexpected daily or seasonal changes in seawater streams regime, the plants brine or other polluted water, *e.g.* fuels or sewage might reach the suction inlet of the feed water pipe and require temporary shutdown of the plant.

No quantitative estimates are available for the increased availability expected in non-surface Intakes. For the present study, an estimated increase of 1% in plant availability (Load Factor) is proposed, and the annual production is increased accordingly by 1% with the same daily plant capacity.

7.2.9. Energy requirements

The total plant energy requirement in SWRO is estimated currently at 4.4 - 5.0 KWh per m^3 product water.

The following energy requirements were estimated for a 30,000 m^3 per day SWRO plant, and for Mediterranean seawater, in Israel. Unit requirements refer to product water quantities, whereas in the model, they refer to feed water quantities for the Intake and pretreatment sections.

Inlet and booster pump	0.70 $kwh.m^{-3}$
High pressure RO pump (after energy recovery)	3.00 $kwh.m^{-3}$
Product water pump	0.48 $kwh.m^{-3}$
Pre treatment	0.22 $kwh.m^{-3}$
Post treatment	0.10 $kwh.m^{-3}$
Total energy requirement	**4.50 $kwh.m^{-3}$**

Salinity of feed water may decrease by 10% as observed in typical Beach wells (Section 3.1.1) and in the SUTRA Model (Section 6.1). Such a reduction of 10% in feed water salinity results in about 10% reduction in the high pressure energy, (Appendix E) *i.e.* 0.3 $kwh.m^{-3}$ reduction. This estimate is based on the comparison of energy requirements shown in Appendix E. The 10% reduction in the pressure required on the membranes is obtained by comparing the two different process spreadsheets in Appendix E.

7.3. Comparison of costs for the various SWRO Intake options

The main comparison was carried out for the two main options: Beach wells and surface seawater Intake according to the data presented in the previous section for conditions in the Israeli Coastal Plain. The two workbooks for the two runs: Test 1 (surface Intake) and Test 2 (Beach wells) are shown in Appendix D. Similar comparisons can be prepared for other Intake types using the same worksheets with the respective adequate data.

The results are summarized and reproduced in Tables 7.2 and 7.3. The direct cost of desalinated water with Beach well Intake 0.656 $ per m^3, is smaller than with sea Intake at 0.784 $ per m^3. However, when considering the indirect costs rather than the total cost of Beach well water, it is 0.715 $ per m^3 compared to the same 0.784 $ per m^3 in surface sea Intake. In other words, a cost reduction of 16.3% is expected with Beach wells. When considering indirect costs, a cost reduction of only 8.8% is expected.

The resulting cost difference between Beach wells and surface seawater Intake is smaller than the differences found by previous authors (Wright and Missier, 1997) as shown in Table 5.1 (40% capital cost reduction and 17.3% O&M costs reduction). The results are of the same order of magnitude if compared to another study (Mercado and Kally, 1996) as shown in Table 5.2 (12.7% cost reduction).

7.4. Sensitivity analyses

The sensitivity of the resulting cost of product water and of the resulting cost difference between surface Intakes and Beach wells was analyzed for the following parameters: Financial Interest Rate, Plant Recovery Ratio, Land Cost, and Hydraulic Capacity of the Hydrogeological System.

Such sensitivity tests are easily conducted by changing values in the input spreadsheet and taking records of the final result in the same spreadsheet.

The sensitivity analyses results are summarized in Table 7.4. The first row shows the resulting costs for the basic scenario for both Intake types and the cost difference between the Intake types.

The next rows show first the parameters revised for each sensitivity test. The next two columns show the value of the parameter in the basic scenario and in the sensitivity test. In the right columns, the various resulting costs are shown for the basic scenario and for each one of the revised parameters.

The advantage of Beach wells, see difference in Table 7.4, change only slightly for the variations in financial parameters (interest rate and cost of land). Also, the advantage of Beach wells increases more significantly under conditions of lower plant recovery or higher hydraulic capacity of the Beach well system.

Table 7.2: Cost estimates for Intake and desalination plant – Surface sea Intake

A. Plant name: Test 1			Type of Intake: Sea Intake			Type of plant: SWRO		
Calculated costs								
Item	Investments costs				Annual costs			
	Investments		Interest during construction		Capital*		O&M	
	LC	FC	LC	FC	LC	FC	LC	FC
	IS	US$	IS	US$	IS	US$	IS	US$
Land and site	6,240,000	0	106,122	0	380,767	0	13,000	0
Intake	1	5,286,900	210,287	175,294	15,277	355,324	0	22,029
Pretreatment plant	0	6,600,000	179,591	149,918	17,541	610,763	0	168,000
Desalination plant*	0	14,817,000	777,384	646,929	55,779	1,220,558	0	386,100
Post treatment	0	8,778,000	372,785	310,690	25,129	538,062	0	155,400
Brine disposal	0	2,574,000	149,362	124,280	10,851	175,527	0	21,450
Total	6,240,001	38,055,900	1,795,531	1,407,109	505,345	2,900,233	13,000	752,979
Total single currency x 1000	158,464	39,616	7,424	1,856	12,106	3,027	3,025	756
Percentage	38.6%						9.6%	
*Including membranes								

Table 7.2(continued)

Item	Chemicals		Energy		Total	
	LC	FC	LC	FC	LC	FC
	IS	US$	IS	US$	IS	US$
	0	0	0	0	393,767	0
Land and site	0	0	1,470,000	0	1,485,277	377,352
Intake	0	877,000	462,000	0	479,541	1,655,763
Pretreatment plant	0	877,000	462,000	0	479,541	
Desalination plant*	0	0	6,300,000	0	6,355,779	1,606,658
Post treatment	0	510,000	2,436,000	0	2,461,129	1,203,462
Brine disposal	0	0	0	0	10,851	195,977
Total	0	1,387,000	10,668,000	0	11,186,345	5,040,212
Total single currency x 1000	5,548	1,387	10,668	2,667	31,347	7,837
Percentage	17.7%		34%		100%	
*Including membranes						

Table 7.2(continued)

	Unit costs (for one m^3) by currency		Percentage (%)	Total unit costs (for one m^3)	
Land and site	IS	US$		IS	US$
Intake	0.039	0.000	1.3	0.039	0.010
Pretreatment plant	0.149	0.038	9.6	0.299	0.075
Desalination plant*	0.048	0.166	22.7	0.710	0.178
Post treatment	0.636	0.161	40.8	1.278	0.320
Brine disposal	0.246	0.120	23.2	0.727	0.182
Total	0.001	0.020	2.5	0.080	0.020
Total single currency x 1000	1.119	0.504	100.0	3.135	0.784
Percentage					

Indirect cost	0.000	0.000	0.0	0.000	0.000
Cost including indirect	1.119	0.504	100.0	3.135	0.784

Table 7.3: Cost estimates for Intake and desalination plant – Beach wells

A. Plant name: TEST 2						Type of plant: SWRO					Type of Intake: Beach wells			
Calculated costs														
Item	Investment costs				Annual costs									
	Investments		Interest during construction		Capital		O&M		Chemicals		Energy			
	LC	FC	LC	FC	LC	FC	LC	FC	LC	FC	LC	FC		
	IS	US$	IS	US$	IS	US$	IS	US$	IS	US$	IS	US$		
Land and site	4,992,000	0	84,897	0	304,614	0	10,400	0	0	0	0	0		
Intake	2,821,782	4,106,324	145,025	120,938	205,374	295,894	3,713	15,625	0	0	1,897,000	0		
Pretreatment plant	0	6,011,881	163,587	136,559	15,992	556,855	0	153,267	0	190,000	462,000	0		
Desalination plant*	0	14,748,386	773,403	643,616	55,525	1,216,595	0	384,229	0	0	5,726,700	0		
Post treatment	0	8,691,089	369,094	307,614	24,880	532,734	0	153,851	0	140,000	2,436,000	0		
Brine disposal	0	2,574,000	149,362	124,280	10,851	175,527	0	21,450	0	0	0	0		
Total	7,813,782	36,131,681	1,685,369	1,333,006	617,236	2,777,605	14,113	728,432	0	330,000	10,521,700	0		
Total single currency x 1000	152,341	36,085	7,017	1,754	11,728	2,932	2,926	732	1,320	330	10,522	2,630		
Percentage					44.3%		11.0%		5.0%		39.7%			
*Including membranes														

Table 7.3 (continued)

A. Plant name: TEST 2			Type of plant: SWRO			Type of Intake: Beach wells	
Calculated costs					Percentage (%)		
Annual costs			Unit costs (for one m^3) by currency				
	Total					Total unit costs (for one m^3)	
	LC	FC	LC	FC			
	IS	US$	IS	US$		IS	US$
Land and site	315,014	0	0.031	0.000	1.2	0.031	0.008
Intake	2,106,087	311,518	0.209	0.031	12.7	0.332	0.083
Pretreatment plant	477,992	900,123	0.047	0.089	15.4	0.404	0.101
Desalination plant*	5,782,225	1,600,823	0.572	0.158	46.0	1.206	0.302
Post treatment	2,460,880	826,596	0.244	0.082	21.8	0.571	0.143
Brine disposal	10,851	196,977	0.001	0.020	3.0	0.079	0.020
Total	11,153,049	3,836,037	1.104	0.380	100.0	2.623	0.656
Total single currency x 1000	26,497	6,624					
Percentage	100.0%						
	Indirect cost		0.099	0.034	9.0	0.236	0.059
	Cost including indirect		1.204	0.414	109.0	2.860	0.715

Table 7.4: Sensitivity analyses

Parameter	Values		Cost of Product Water $ per m^3				
	Basic Scenario	Revised	Surface Intake	Beach wells		Difference*	
				Direct	Total	Direct	Total
Basic scenario			0.784	0.656	0.715	0.128	0.069
Interest rate	5%, 6%	7.5%, 9%	0.877	0.745	0.806	0.132	**0.071**
Recovery rate	50%	40%	0.898	0.751	0.818	0.147	**0.080**
Land cost ($ per m^2)	100	0	0.774	0.648	0.706	0.126	**0.068**
Single well capacity (m^3 per day) GW flow rate (m^2 per yr per m)	2,400 1,100	4,800 2,200	0.784	0.638	0.696	0.146	0.088

* Difference of product water cost between surface Intake and Beach wells.

8. GUIDELINES FOR INTAKE TYPE SELECTION AND DESIGN

8.1. Introduction

The present study deals with the selection of SWRO Intake Method (surface and non surface) and with type, location and depth of the Intake that will render optimal operation. First, some general concepts and principles are summarized to serve as a framework of preliminary selection and further quantitative criteria are identified.

To avoid adverse affects on the inland freshwater, the best location would be in the seabed as far as possible from the coastline. However, due to the high cost involved in offshore drilling and abstraction, only continental sites are considered. The best of these would be a coastline site.

The Intake depth is determined by the quantity to be abstracted and by the depth required for effective filtration. It will also be determined by the desire to minimize adverse effects on the freshwater zone. When the aquifer is not stratified, the Intake will be located as deep as possible; underneath the seawater interface.

A special dual fluid simulation model is proposed to study the geohydrological considerations and to set the optimal location and depth of the Intake (Chapter 6.1).

This chapter is concerned with Beach wells since they are the most important and common of the non-surface Intakes.

Beach wells constitute boreholes constructed close to the coastline which are equipped with pumps, piping and casing, made of materials resistant to corrosion in order to obtain seawater or a mixture of seawater and freshwater from the aquifer.

8.2. Advantages of the Beach well approach

One of the most significant benefits of this method is the natural pre-filtration that occurs by the soil layers adjacent to the borehole (if the regional lithology comprises well-graded material). This may cause an important reduction in the construction and operational costs of the installation since less filtration is required due to the higher initial quality of the water. Furthermore, energy consumption is lower and the Reverse Osmosis membrane lifetime increases, which decreases the maintenance costs.

Additionally, serious environmental impacts that may be caused by the conventional method of acquiring seawater (surface Intake) can be avoided by using Beach wells; since the amount of waste (brine and chemicals) is significantly reduced. Moreover, well-graded soil layers can provide very effective screening of aquatic microorganisms, which decreases the construction expenses and minimizes the potential risk of bio-fouling in the system (reduced backwashing activities).

Other important advantages of the Beach wells such as the avoidance of flocculation, sedimentation and the good balance of seawater temperature and salinity should be taken into consideration prior to designing the desalination plant.

8.3. Disadvantages of the Beach well approach

The use of Beach wells in Seawater Reverse Osmosis (SWRO) installations is not always the best option since there are other factors that should be examined.

Great attention should be given to avoid possible over-exploitation and depletion of groundwater. Consequently, careful hydrogeological investigation will be required to study adequately the interactions of the seawater with the freshwater in aquifers close to the coastline zone. A detailed discussion of this issue is presented in Chapter 6.1 and Chapter 9.

Further, another disadvantage of the Beach well, is their limited capacity compared to surface Intakes. Constructing a large number of Beach wells, with long collectors can solve this problem. However, the economic viability of this solution may become questionable.

The supervision and control system of Beach wells is more complicated than the control system for surface Intakes. It includes supervision of a large number of well pumps and monitoring of the state of groundwater adjacent to the wells and in other areas that may be affected.

8.4. Criteria for the implementation of the Beach well approach

In addition to considering the aforementioned concepts and principles and the considerations summarized in Chapter 5, there are additional scientific criteria that indicate when the Beach well approach is more beneficial than a conventional surface Intake.

Criteria of high importance comprise the Beach well capacity (Chapter 9) and the fraction of freshwater pumped (Chapter 6.1).

Lithology of the specific area determines to a great degree the efficiency of the Beach wells. The potential for natural filtration depends mainly on whether the strata close to the wells consists of well-graded material (fine to coarse sand, fine to coarse gravel and combinations of these). If this is the case, natural filtration will occur effectively and the acquired seawater will have a significantly improved quality.

If hard rock formations are encountered in the desalination plant area then additional considerations should be made concerning the engineering properties of these formations. Particularly, the rock quality (expressed by the RQD index), the total number of fissures and their specific characteristics (open, closed, filled with other material, dip) can indicate the difficulty in drilling as well as the extra equipment that may be necessary for the appropriate operation of the Beach wells. Moreover, careful investigation of the tectonic regime of the area should be carried out in order to reveal possible faults, landslides, or mass movements that may cause malfunctions in the seawater Intake. Such structures incorporate the danger of removing the underground water from the aquifer if the boreholes intersect them.

The regional geological conditions near the desalination plant constitute the most significant factor for the selection of the seawater Intake method. The level of the piezometric surface plays a very important role since if the aquifer is deeper than the construction, the operation costs of the boreholes can become very high and which might not be an acceptable option. The existing piezometric surface can be identified by studying the hydrogeological regime of the area by using test wells, or by using existing boreholes; hydrogeological maps and scientific surveys conducted in the area, see Chapter 9.

A criterion of high importance is the fraction of freshwater pumped (Chapter 6.1) and the Beach well capacity (Chapter 9).

In order to avoid over-exploitation of groundwater, careful quantification of the acquired freshwater must be carried out. This should be done by comparing the seawater salinity with the salinity of water acquired from test wells. In this way, water and salinity balances can be carried out to estimate freshwater abstraction and to maintain it within an acceptable range.

The aquifer type is another significant factor that should be examined profoundly. Confined aquifers may have their seawater boundary at a large distance from the coastline, which could result in a decrease of the seawater flow rate into the wells. However, a far-reaching influence of the pumping on the freshwater inland can be encountered; thus, phreatic aquifers are preferred in the Beach well approach.

Beach well capacity constitutes a crucial aspect for the economic viability and the appropriate operation of the plant on a long-term basis. Therefore, great attention should be paid in order to design the desalination installation in a sustainable way, capable of covering the possible increased needs of the future, without depleting the freshwater storages. Additionally, the exploitable Beach well water yield can be estimated by using test wells and by monitoring their capacities to remain under the limit of the aforementioned sustainable yield.

It should be emphasized that the monitoring process should continue during the operation of the plant in order to trace whether the fraction of abstracted freshwater is higher than allowable. In this case, the excessive fraction should be penalized following the legislative framework that protects the underground storage. It has to be stated, “that in many countries, *e.g.* Greece, there is not such a legislative framework and this often causes serious damages to groundwater”.

Finally, chemical analysis of the existing groundwater should be conducted to detect possible contamination of the feed water for which special treatment has to be designed.

Finally, it can be argued that the Beach well approach can be a beneficial method of pre-filtration that may reduce significantly the construction, operation, and maintenance costs of desalination plants, but careful study of the aforementioned site factors should be undertaken prior to the implementation of this technique to avoid potential problems during the operation of the plant.

8.5. Design criteria

Design criteria for 'Beach well Systems' were studied by Mercado and Kally, 1996, who performed a techno-economic study including hydrogeologic modelling. Their main conclusions for the conditions in the coastal plain of Israel are:

- The pumping capacity of Beach wells depends exponentially (quadratically) on the thickness of the aquifer. The minimal saturated thickness of the aquifer for Beach well application should not be smaller than 50 m
- Phreatic aquifers should be preferred over confined aquifers, which may have their seawater boundary at a large distance from the coastline, therefore decreasing the rate of seawater flowing to the wells. In a multilayered aquifer the upper aquifer should be selected
- Under the hydrologic conditions prevailing in the coastal plain of Israel, the maximum pumping capacity of a Beach well battery running parallel to the coastline cannot exceed 15,000 $m^3.day^{-1}.km^{-1}$
- The cost of water transport limits the maximum length of such a battery supplying a single plant to 5 - 10 km
- When assuming pumping of 10,000 $m^3.day^{-1}.km^{-1}$ and a recovery ratio of 50%, the maximum capacity of the SWRO plant that justifies non-surface Intake is 25,000 - 50,000 m^3 per day. For larger plants, conventional surface Intakes are preferable
- To minimize the freshwater fraction in the pumped water, the following design criteria should be followed:
- Wells should be located as near as possible to the coastline. (in cases of significant tidal variations, which are not common in MENA coasts, the location will be determined by the maximum sea level)
- Wells should penetrate all the depth of the aquifer and reach its base
- The well screen should be as short as possible with maximum percentage of perforated area

All the desalination processes require pretreatment of some type to condition the raw water supply before the actual desalting step. The extent and effectiveness of this pretreatment can be affected by the care taken in the development of the raw water source. With proper planning and design, such as casing and gravel packs (Appendix H), the amount of pretreatment can be reduced and the quality of the water delivered can be improved providing long-term benefits by reduced maintenance costs and increased operating time of the desalting facility.

Well screens are perforated with numerous small slots and are surrounded by gravel and sand. Proper development by pumping and back flushing grades the surrounding gravel/sand layer and opens channels for the water flow. The seawater then infiltrates through the sea bottom and flows toward the wells. A number of wells must be drilled since the Intake area of each well must be limited.

Beach wells or Ranney collectors can be used effectively in beds consisting of rock or fine-grained silts. Silts with D_{10} effective grain size of 0.05 mm or less and a coefficient of permeability below 10 - 3 $cm.s^{-1}$ present a problem because capillary forces hold water in tiny voids of the soil.

The wells should be designed by competent engineers and carefully constructed. This may include the use of test boreholes, geophysical logging, placements of well screens, gravel packing, and sealing of suitable zones and development of completed wells. The materials selected for the construction of the well, pump and water transmission line, should be compatible with the water quality involved. For instance, steel cased wells would be inadvisable in most brackish water or seawater locations because they can create problems due to corrosion of the casing and the pumping of loose corrosion products to the desalination facilities. Stainless steel (at least for the screens) and PVC are therefore the preferred materials.

9. SURVEY AND EXPLORATION APPROACHES

9.1. Methods and survey techniques

The choice of appropriate sites and Intakes and their design require the assessment of site properties. The following hydrogeologic methods and investigation techniques are considered for this purpose:

a) Geological mapping
b) Geological cross sections
c) Well logs including oil wells in general and off shore wells in particular
d) Well pumping tests including interference tests
e) Sampling and physico-chemical analyses of water quality in wells
f) Regional groundwater balances, including Mathematical Flow Models
g) Conventional geophysical surveys
h) TDEM (Time Domain Electromagnetic) methods to estimate depth of seawater intrusion (interface configuration)
i) Remote sensing methods and GIS

The purpose of these methods and techniques is to establish and determine quantitatively the specific values of the relevant site properties outlined above and to respond the following questions:

a) Is there a water bearing formation available?
b) How much water can be extracted?
c) Will the water quality remain stable during pumping?
d) How many production wells must be constructed to safely obtain the desired yield?
e) What are the optimum well locations and pumping rates?
f) If water quality changes, what will be the changes in concentration of key chemical parameters?
g) Should the well be 'open hole' type or screened?

9.2. Site specific selection methods

As for site selection of Beach wells for SWRO purposes, it is clear that terrigenous, granular aquifers are the most favourable. Hence, there seems to be no justification for any groundwater Intake development in karstic, volcanic, igneous and/or metamorphic environment but in granular aquifers such as sandstone, alluvial deposits and similar. That is, a good, reliable geological map/study or at least a reconnaissance report is a prerequisite for any site selection and advanced activities.

The required coastal geology, *i.e.* non-rocky, granular, and preferably loosely packed sediments may be found in some of the Mediterranean and the Arabian Peninsula coastal areas. Within the Mediterranean region, for example, the north (Turkey, Greece, Adriatic coasts, Italian and French Riveras') is characterized mainly by steep, rocky coastlines (and tectonically emerging) however, in the south, the coastal regions are sandy by nature due to some tectonic submerging events.

This is the general outline, however, it should be noted, that adequate geological configuration may be encountered even within the most precipitous coastal environment; in some deltaic deposits, river outlets, closed harbours, and short sandy shores. Numerous examples exist in the French Riviera, south east coast of Cyprus, the Balearic Islands and Spain. The second issue to be pursued is the hydrological characteristics of the particular coastal stretch selected. The easiest way is by means of collecting and reviewing available data, if it exists. In case of no data or insufficient information concerning the local structure, stratigraphy, and hydrogeological configuration, the required data may have to be generated from the beginning, as follows:

- Thickness, stratigraphy, hydrogeological outline and lateral extension of aquiferous units can be obtained by means of exploratory slim holes or exploratory/production wells, and/or a network of observation wells
- Prior to drilling a preliminary geoelectrical/electromagnetic survey is suggested in order to acquire a very rough indication, at least, as for the target sequence
- Later, after drilling within a strip parallel to the seashore, the boreholes will serve, *inter alias*, as calibration points between the lithological and hydro-stratigraphic profile and the geophysical findings. Suggested geophysical methods, are the VES (vertical electrical sounding) and TDEM (time domain electro-magnetic) surveys
- The main object of the TDEM survey once the depth and geometry of the seawater/fresh water interface has been established, is to adjust and correlate the findings with the outline of inland aquifer extension and thickness of saturated beds above and below the interface
- The hydrological properties of the explored sequence will be studied by means of hydrological tests in the exploratory/production wells. The main object of the tests is the transmissivity value and the number of required Beach wells will be set accordingly. In case of poor aquifer performance the feasibility of galleries (pending SWL depth, lithology, and thickeners of fresh water saturated beds) will be considered
- The above-obtained information will serve for setting up a simulation model

9.3. Water quality

In order to properly design and procure a desalination unit, the characteristics of the raw water supply must be known and specified. These characteristics usually include the chemical constituents, non-dissolved solids level, microbial content, and temperature. It is important to realize that these characteristics may change with time and quantity withdrawn. These changes could occur daily, annually, seasonally *etc.* depending on the circumstances. An adequate testing program is essential to avoid mistakes in design and later problems in operation. For ground water, it is prudent to complete the production wells and to test them at the expected production rates before drawing up final specifications. Hydro-geologists also perform suitable tests as outlined in section 9.1, to provide estimates of the future water quality available from the well field. With seawater sources, samples should be taken at various times and under different conditions in order to properly characterize the feed water.

Feed water from Beach wells is usually of the highest quality. SDI's are generally about 1. Biological activity is basically not present. If either SDI or biological activity are unsatisfactory, the well and its surroundings should be investigated to assess the reason for the problem.

9.4. Guidelines for the estimate of Beach wells efficiency and discharge rate

9.4.1. General

The issue of pumping properly filtered seawater from Beach wells is somewhat controversial because the expected filtration properties do not coincide with favourable hydraulic properties in terms of high producing wells. As is well known from water well construction in terrigenous; granular aquifers, a properly applied gravel pack (or formation stabilizer) as is currently implemented in Israel in order to get sand free and even turbidity free water, can result in high well losses, poor specific capacities and, consequently, low discharge rates.

As for Beach wells, bottomed and screened within the seawater-saturated beds, a high discharging well may withdraw seawater that may not be properly filtered. Pure, clean seawater may be pumped only from poorly discharging production wells, *i.e.* the issue is purely economic.

9.4.2. Aquifer types

The potential aquifers into which a Beach well is to be driven are, naturally, those which are proper for groundwater withdrawal in general. This property is discussed below for each aquifer that is found in MENA beaches.

Granular Aquifers

This lithological group, including sandstones, colluvial and alluvial deposits, is the most common one in coastal geology, even within stretches that are rocky and mountainous in nature. These aquifers are most easily approached and handled and usually bear no unforeseeable hydraulic scenarios. Notably, groundwater hydrology, such as Darcy's law, and all related analytical methods have been developed for granular aquifers. However, the established formulas and analytical methods are also used, with dependable results, for aquifers in which groundwater flow is turbulent in nature, such as Karstic ones. A Beach well constructed in a very coarse grained material, such as alluvial outwash comprising coarse gravel and boulders may discharge enormous quantities of water with negligible head losses, but with poor quality. However, fine-grained deposits may guarantee perfect filtration, but by means of low discharging wells. The issue to be pursued is, therefore, to satisfy both requirements, *i.e.* quality and discharge rate.

Karstic Aquifers

This group of limestone and dolomite aquifers that constitute the best in terms of water well drilling may not be that favourable concerning Beach wells. In highly karstic coastal aquifers, such as the cavernous reefal limestone aquifers of Yucatan, Caribbean and the Bahamas, the tides and ebbs can be detected in fresh water holes (collapsed dolinas) 5 - 15 kilometres inland, with almost no delay. Accessing the system of solution cavities by drilling is likely to produce enormous quantities of water but eventually adequate filtration is rather doubtful, if at all. Notably, limestone and dolomite aquifers, which are not karstic, are usually rather compact and may not constitute a promising media for Beach wells.

Igneous Rocks Aquifers

This group of aquifers is usually very poor. The water of this group hardly moves since it is stored in cracks, fissures, and brecciated belts. Therefore, it is not expected to support any Beach well project.

Volcanic Rocks Aquifers

This group of aquifers is usually very poor and may hardly sustain any groundwater withdrawal project (if so - most probably by numerous low discharging boreholes) and can not be considered as a favourable medium for Beach wells. However, there are some derivates of volcanic rocks and phenomena, such as scoriaceous basalts, tuff, and lapili that constitute superb aquiferous properties but, nevertheless, the combination of these types of volcanic lithology and coastal regions cannot be considered a common scenario.

9.4.3. Hydrological configuration and well discharge (single well)

The Beach well hydraulics is usually taken as steady state flow in unconfined aquifers. It is assumed that a concentric boundary of constant head surrounds the well (or line of wells). In this particular case, a constant head has also to be attributed to the sea boundary. The formulas for steady state flow that do not consider this boundary; can be applied as an acceptable low approximation for a single Beach well.

The original analytical method for this type of flow is based on the old works of Dupuit (1863), and later the Thiem- Dupuit's method. The formula established was eventually identical to the Thiem formula (1906) for a well in confined conditions, if the dynamic draw down in a well is small in relation to the saturated thickness of the aquifer (otherwise the assumption that the thickness of the operative aquifer is constant is no longer satisfied and the formula is not relevant.

The Thiem formula is as follows:

$$T = \frac{(lnR_2 - lnR_1)}{(2P(h_2 - h_1))}$$

where,
P is the well discharge, in $m^3.day^{-1}$
T is the transmissivity of the aquifer, in $m^2.day^{-1}$
R_1 and R_2 are the respective distances of two piezometers from the pumped well
h_1, h_2 are the respective elevations of the water levels in the two piezometers

Later developments resulted in simplified equations, such as Jacob's (after Cooper and Jacob, 1946) - although established for unsteady state in a confined aquifer is widely used for all types of aquifers, including non-granular ones. The relation of *P* and *T* is approximated from dynamic observation in the pumping well itself as:

$$T = \frac{2.3\,P}{4\pi\ \Delta S}$$

The discharge rate of a single well can therefore be estimated as:

$$P = \frac{\Delta S . T}{4.4}$$

P is the well discharge in $m^3.h^{-1}$, T is the transmissivity in $m^2.day^{-1}$, and ΔS *is* the acceptable draw down of water table in the well in metres. There is a linear relation between the components of the equation in terms of higher discharge when the transmissivity is higher, for the same ΔS.

The expression of Transmissivity, is the product of *k* (hydraulic conductivity) and the aquifer thickness. Following an estimated order of magnitude of *k* values in some granular lithologies (after Schoeller, 1962).

Material	***k* in m.day^{-1}**
Clay	10^{-5} to 10^{-7}
Silt	10^{-1}
Fine sand	10^{-1} to 10
Coarse Sand	10^{0} to 2 x 10^{2}
Gravel	10^{0} to 10^{3}, or more

The average *k* characterizing the Israeli coastal aquifer (calcareous sandstones) is 15 - 20 m.day^{-1} and, accordingly, the *T* value for the total thickness of the aquifer, at the shoreline, is 3,000 - 4,000 $m^2.day^{-1}$. However, the range of extracted, real values is rather wide and for a 50 - 100 m depth of a Beach well a *T* of 1,000-2,000 $m^2.day^{-1}$ can be considered as a reasonable approximation. A 50 m thickness of aquifer may discharge, by means of a well, 100 $m^3.h^{-1}$ with a dynamic draw down of 0.5 m only (aquifer loss only, not including well assembly losses), in optimal conditions. However, for practical purpose, based on many discharge and specific capacity figures, a properly constructed and gravel packed well will produce this discharge with a dynamic draw down of 3 - 6 m, *i.e.* specific capacities of 15 - 30 $m^3.h^{-1}.m$, owing to mainly well envelop and assembly losses. The above displayed configuration and figures apply to the terrigenous sequence of the Plio-Pleistocenic coastal aquifer of Israel, where information and data are abundant. In any other location, some basic exploratory measures have to be carried out in order to study the particular hydrological configuration and establish the production approach, as follows:

- Exploratory drilling required to investigate the depth and stratigraphy of the target aquifer. A preliminary study should always constitute drilling, however, the lateral extension of the stratigraphic information can utilize as an auxiliary tool some geophysical methods

- Pump testing of the boreholes in such a way that a general/average idea and/or the specific data as for a particular hydrologic behaviour and hydraulic properties (K, T) of the target aquifer will be attained. The pumping tests that are usually used are, the draw down and recovery tests for T well and interference test for T aquifer

By these measures, a preliminary idea as for depth, potential discharge, and number of required Beach wells in a given region could be obtained in order to comply with the demands of a given plant.

9.4.4. Galleries or coastal drains

Galleries or coastal drains are a practical tool to be utilized in the case of a thin aquifer or poor hydraulic properties requiring numerous boreholes. Originally, this approach was developed in order to skim fresh groundwater above the seawater interface but may be a practical tool (usually closer to the coast line), and economically viable to withdraw seawater for desalination purposes. The flow rate of galleries in the upper part of a thick aquifer can be estimated as follows:

$$Qu = \frac{\pi . k . a}{\log e(R / r)}$$

The value of R that can be approximated by the distance of the gallery from coast line, is defined as:

$$R = 1/\mathrm{i} \sqrt{Qu}.a / \pi.k$$

where,
k = Permeability, assume:15 m per day
a = Depth of gallery below the phreatic water level, assume: 3m
I = Hydraulic gradient
R = Radius of drain pipe, assume: 0.2 m
Qu = Discharge, in m^3 per day, for one m gallery
R = Radius of influence (assume: 25 m)

Because of the assumed values, we obtain:

$$Qu = \frac{\pi .15.3}{\log e(25/0.2)} = 29\ m^3.day^{-1}$$

The meaning is that the discharge, per day, of a gallery in the coastal plain of Israel, 25 m from the water line may be some 29,000 $m^3.day^{-1}$ for one km (25,000 $m^3.day^{-1}$ in case the gallery is 50 m from the water line). That is, 1000 - 1200 $m^3.h^{-1}$, or 8-10 MCM per year for a 1 km gallery.

10. CONCLUSIONS AND RECOMMENDATIONS FOR THE MIDDLE EAST AND NORTH AFRICA REGION

10.1. Potential for practical applications

The present study provides the desalination industry in its small SWRO sector, an improved design solution for seawater Intakes. This type of Intake will enable a supply of feed-water with improved quality and reliability. Under favourable site conditions *i.e.* granular formations with sufficient hydraulic conductivity but with adequate filtration capacity, a reduction of the Intake cost and pretreatment cost is expected. Such favourable conditions exist in many of the regional sand and sandstone coastal aquifers in the MENA countries. The potential impact on the total cost of desalination can be significant, particularly for small SWRO plants.

The study products include guidelines for selecting sites and technologies, and for assessing site properties. A data processing framework has been developed including spreadsheets for cost estimates based on site properties. These will enable scientifically based and competent decisions on the applicability of non-surface Intakes.

Following the completion of the present study and prior to its implementation in a certain coastal region additional efforts will be required to assess site properties by the proposed assessment methods.

10.2. Application in the MENA region

Sites for the application of the proposed Intakes can be identified in the Middle East and North Africa Region.

Beach wells or similar non-surface Intakes can be viable and cost effective where the following conditions apply:

- Capacity of plant not exceeding 50,000 m^3 per day
- SWRO process selected for desalination
- The beach geological formations are Granular
- Depth of formation and hydraulic conductivity result in a Transmissivity exceeding 1,000 m^2 per day
- Impact on groundwater stocks inland not exceeding 10% of plants capacity

Many of the coasts along the Mediterranean, Red Sea, Indian Ocean, and the Arab/Persian Gulf meet these conditions. A notable example is the coastal plain of Israel, which is widely discussed in the present study.

10.3. Follow - up

Follow-up activities are proposed in conclusion of the study. The follow-up activities include: in depth advanced studies in specific favourable sites, design of pilot plants in such sites, development or upgrading of site exploration methods that have been found relevant for the assessment of the site properties. Follow-up recommendations also include, monitoring existing non-surface seawater Intakes such as the performance of Beach wells and their interference with inland aquifers.

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Additional references on the Dynamics of Seawater intrusion are presented at the end of Appendix B.

APPENDIX – A

THE QUESTIONNAIRE

Contents

A questionnaire was prepared for circulation to:

1. SWRO Beach well planners and owners
2. other users of Beach wells
3. producers designers and owners of SWRO installations

Two forms were prepared and two letters of transmittal, one for Beach well systems and one for regular SWRO installations, see attached:

To the 85 Questionnaires distributed only 17 responses were received and presented in the progress reports. Asterisks in the mailing list indicate the responders. No activity in the relevant subjects was responded to by 9. Information on RO Beach wells was responded to by 5. Other Intakes were reported by 2 and general information received from 2 respondents. Important data on the existing experience are still missing.

The following reasons were given for going to Beach wells instead of surface Intakes:

1. Much better pre-filtration effect and therefore prolonged plant and membrane lifetime
2. No expensive offshore installation, excellent cleanings of the seawater, reduction of marine growth and microorganisms problems, no flocculation, sedimentation and media-filtration, good balance of seawater temperature and salinity
3. Site is not located adjacent to surface seawater. Better water quality with reduced filtration and pretreatment required

Problems encountered during operation were only reported on surface Intakes:

- Occasionally, shoals of small fishes. Excess of algae during bad weather or storms

No problems in the operation of Beach wells were reported by the responders.

Table A.1: Responses obtained on Beach wells, after some processing

Place	No. of wells	Reported by	Capacity $m^3.day^{-1}$	Distance from coast line (m)	Formations	Total depth m	Screen diameter (cm)	Salinity	
								Type	Value in well $mg.l^{-1}$
Oman	9	Roberto Cutinbo Al-Bourg Enterprises	2000-5000	50-300		20-50		TDS	38,000-56,000
U.A.E. Fujairoh	6-20	Peter Wolf VATECH	1125	10	Sand	> 50	> 20.32 (8”)	TDS	37,000
Bahamas Nassau	2-5	Klaus Peter Thiel Preasay	1000	25-100	Sand Volcanic	> 10	> 40.64 (16”)	TDS	36,000
Saudi Arabia Tanajeb		Klaus Peter Thiel Preusag	2250	25-100	Sand		> 16”	TDS	41,000-45,000
Cayman Is.	2-5	Gregori S. Mctagart	2500	100-500	Sandstone	20-50	> 40.64 (16”)		Seawater

TAHAL CONSULTING ENGINEERS LTD.
54 Ibn Gvirol St., P.O.B 11170, Tel-Aviv, 61111, Israel
Tel: 972-3-6924515 Fax: 972-3-6924666 E-mail: schwarzj@tahal.co.il

06 December 1999

Re: **Seawater Reverse Osmosis Plants - Beach wells and other Subsurface Intakes**

Dear Sirs,

We have been awarded by the Middle East Desalination Research Center, Sultanate of Oman, with a study on Beach wells and other Subsurface Intakes for Small Seawater Reverse Osmosis Plants. The study also includes comparisons to conventional suction pipe Intakes. The objective and essential goal of the project is to provide a comprehensive state of the art document on utilizing Beach wells, and similar non-surface seawater Intakes for SWRO systems, and to: (1) identify the likely improvements in this technology, and (2) develop and verify the criteria for the choice and design of these types of seawater Intakes for small Sea-water RO desalination plants.

The document will cover the different types of non-surface Intakes, their characteristics, their possible use in SWRO desalination plants, design methods including simulation, the site requirements and survey methods for Beach wells or related seawater Intakes. The document will also review existing facilities and their possible improvements.

We kindly ask for your assistance in this important study by filling the attached questionnaires and submitting to us information available to you on the design and performance of seawater Intake installations for both desalination and other purposes. The attached two questionnaires may help you in providing preliminary information. Questionnaire #1 on Beach wells and other non-surface Intakes, and Questionnaire #2 on seawater suction pipe Intakes. Please use the questionnaire appropriate to your plant(s) and for each question please tick the relevant answer. In some questions more then one answer may be ticked. However any other format convenient to you will also be highly appreciated.

Thanking you in advance for your response until September 30, 1999 to be addressed to:

Joshua Schwarz
Tahal Consulting Engineers Ltd.
54 Ibn Gvirol St., P.O.B 11170, Tel-Aviv, 61111, Israel
Fax: 972-3-6924666
E-mail: schwarzj@tahal.co.il

Yours Sincerely

(Joshua Schwarz)
Project Manager

Encl. MEDREC letter
Two Questionnaires

cc: Dr. K Venkat Reddy
The Middle East Desalination Research Center

Beach wells and other subsurface Intakes

1. Responder Name: ……….
   Organization: ………………….
   Address: ……………………..
   Tel: ………………….
   Fax: …………………
   E-mail: ………………

2. Function of Responder:
- ◻ Owner
- ◻ Operator
- ◻ Planner
- ◻ Contractor
- ◻ Other (Specify): ……………..

3. Location of Beach wells:
   Country: ……………………..
   State: …………………………
   Province: ……………………..
   Township: ……………………
   Longitude: … … …
   Latitude: … … …

4. Present Status of Intake:
- ◻ Planned
- ◻ Under construction
- ◻ Operative
- ◻ Abandoned
- ◻ To be rehabilitated
- ◻ Other (Specify): ……………..

5. Type of Subsurface Intake:
- ◻ Vertical Wells
- ◻ Radial Ranney Collectors
- ◻ Inflow Galleries
- ◻ Seabed filtration
- ◻ Self-jetting well points
- ◻ Other (Specify): ……………..

6. No. of Wells:

9. Distance of Wells from coastline Inland:
- ◻ Less than 10 m
- ◻ 10-25 m
- ◻ 25- 100 m
- ◻ 100-500 m
- ◻ More than 500 m

   Offshore wells distance from coastline:
- ◻ Less than 10 m
- ◻ 10-50 m
- ◻ More than 50 m

10. Depth of Intake (below ground surface):
- ◻ Less than 1 m
- ◻ 1-3 m
- ◻ 3-6 m
- ◻ 6-10 m
- ◻ 10-20 m
- ◻ 20-50 m
- ◻ More than 50 m

11. Diameter of Intake Well or Gallery:
- ◻ 100-200 mm
- ◻ 200-400 mm
- ◻ More than 400 mm

12. Other Intake dimensions:
    …………………………….
    …………………………….

13. Operating capacity of Beach well:
- ◻ Less than 10 m^3/day
- ◻ 10-100 m^3/day
- ◻ 100-1000 m^3/day
- ◻ 1000-2500 m^3/day
- ◻ 2500-10000 m^3/day
- ◻ More than 10000 m^3/day

    Operating capacity of treatment plant: …………… m^3/day

    Number of days operating in a year:
- ◻ Less than 50 days
- ◻ 50-100 days
- ◻ 100-200 days

- ◻ Single well
- ◻ 2-5 wells
- ◻ 6-20 wells
- ◻ More than 20 wells

7. Use of Seawater:
- ◻ Desalination
- ◻ Swimming Pool
- ◻ Cooling System
- ◻ Other (Specify): ……………..

8. Reasons for going to Beach well instead of surface Intake:
………………………………
………………………………
…………

- ◻ 250-365 days

14. Aquifer characteristics (lithology):
- ◻ Sand
- ◻ Sandstones
- ◻ Carbonates
- ◻ Alluvium
- ◻ Volcanics
- ◻ Igneous
- ◻ Other (Specify): ……………..

Beach wells and other subsurface Intakes (cont.)

15. Salinity of feed water
- ◻ Seawater ……. TDS
- ◻ Mixed (above 50% seawater)
- ◻ Mixed (Less than 50% seawater)
- ◻ Varying over time

16. Turbidity and impurities in feed water
- ◻ Sand
- ◻ Algae
- ◻ Oil
- ◻ Sewage
- ◻ Micro organisms and Bacteria
- ◻ Other (Specify): ……………..

17. Interference with other nearby groundwater wells
- ◻ Non Existent
- ◻ Not Known
- ◻ Lowering of water table in nearby wells
- ◻ Increase of Salinity in nearby wells
- ◻ Other (Specify): ……………..

18. Year of commissioning
- ◻ Future
- ◻ 1998-2000
- ◻ 1995-1997
- ◻ 1990-1995
- ◻ 1980-1990
- ◻ Earlier than 1980

19. Intake Investment costs
- ◻ Less than 10,000 US$
- ◻ 10,000-50,000 US$
- ◻ 50,000-250,000 US$
- ◻ 250,000-1,000,000 US$
- ◻ 1,000,000-10,000,000 US$
- ◻ More than 10,000,000 US$

20. Intake Operation and maintenance costs
- ◻ Less than 10,000 US$/yr.
- ◻ 10,000-50,000 US$/yr.
- ◻ 50,000-250,000 US$/yr.
- ◻ 250,000-1,000,000 US$/yr.

21. Type of Pre-treatment
- ◻ Coarse filtration
- ◻ Flocculation
- ◻ Coagulation
- ◻ Sedimentation
- ◻ Settling
- ◻ Rapid sand filtration
- ◻ Slow sand filtration
- ◻ Diatomaceous earth filtration
- ◻ Chlorination
- ◻ Other (Specify): ……………..

22. Desalination: Method
- ◻ BWRO
- ◻ SWRO
- ◻ MSF
- ◻ MED
- ◻ VC
- ◻ Other (Specify): ……………..

23. Desalination+Pretreatment plant Investment cost.
- ◻ Less than 10,000 US$
- ◻ 10,000-50,000 US$
- ◻ 50,000-250,000 US$
- ◻ 250,000-1,000,000 US$
- ◻ 1,000,000-10,000,000 US$
- ◻ More than 10,000,000 US$

24. Desalination+Pretreatment plant operation and maintenance cost
- ◻ Less than 10,000 US$/yr.
- ◻ 10,000-50,000 US$/yr.
- ◻ 10,000-50,000 US$/yr.
- ◻ 50,000-250,000 US$/yr.
- ◻ 250,000-1,000,000 US$/yr.
- ◻ More than 1,000,000 US$/yr.

25. Please add more information on performance and costs if available.

…………………………………………

………………………………………

………………………………………

- ☐ More than 1,000,000 US$/yr. ……………………………………… ………………………………

26. Special Problems encountered during operation of Intake and of desalination plant:

Date: Signature:

Sea-water Suction Pipe Intakes

1. Responder Name: …………….
   Organization: ………………….
   Address: ……………..………..
   Tel: ………………….
   Fax: …………………
   E-mail: ………………

2. Function of Responder:
- ◻ Owner
- ◻ Operator
- ◻ Planner
- ◻ Contractor
- ◻ Other (Specify): ……………..

3. Location of Intake:
   Country: ……………………..
   State: …………………………
   Province: ……………………..
   Township: ……………………
   Longitude: … … …
   Latitude: … … …

4. Present Status of Intake:
- ◻ Planned
- ◻ Under construction
- ◻ Operative
- ◻ Abandoned
- ◻ To be rehabilitated
- ◻ Other (Specify): ……………..

5. Type of Sea-water Intake:
- ◻ Pipe anchored to sea ground
- ◻ Covered pipe
- ◻ Cooling water basin of a power plant
- ◻ Other (Specify): ……………..

6. No. of Intake pipes:
- ◻ Single pipe
- ◻ 2 pipes
- ◻ More than 2 pipes

8. Length of pipe(s) into sea:
- ◻ Less than 10 m
- ◻ 10-25 m
- ◻ 25- 100 m
- ◻ 100-500 m
- ◻ More than 500 m

9. Distance of desalination plant (Or another end) from Coastline:
- ◻ Less than 20 m
- ◻ 20- 50 m
- ◻ 50-100 m
- ◻ 100- 200m
- ◻ 200-500 m
- ◻ 500-1000 m
- ◻ More than 1000 m

10. Diameter of Intake Pipe:
- ◻ 100-200 mm
- ◻ 200- 300 mm
- ◻ 300 – 400 mm
- ◻ 400 – 500 mm
- ◻ 500 – 600 mm
- ◻ More than 600 mm

11. Other Intake dimensions:
    ………………………….
    ………………………….

12. Intake pipe material
- ◻ Cast Iron/Carbon steel
- ◻ Plastic/Fiberglass
- ◻ Prestressed concrete
- ◻ Coated steel pipe
- ◻ Asbest-Cement
- ◻ Other (Specify): ……………..

13. Operating capacity of treatment plant:
- ◻ Less than 10 m3/day
- ◻ 10-100 m3/day

7. Use of Seawater:

- ◻ Desalination
- ◻ Swimming Pool
- ◻ Cooling System
- ◻ Other (Specify): ……………..

- ◻ 100-1000 m3/day
- ◻ 1000-2500 m3/day
- ◻ 2500-10000 m3/day
- ◻ More than 10000 m3/day

14. Number of days operating in a year:

- ◻ Less than 50 days
- ◻ 50-100 days
- ◻ 100-200 days
- ◻ 250-365 days

Sea-water Suction Pipe Intakes (cont.)

15. Salinity of feed water
- ☐ Seawater ……. TDS
- ☐ Mixed (above 50% seawater)
- ☐ Mixed (Less than 50% seawater)
- ☐ Varying over time

16. Turbidity and impurities in feed water
- ☐ Sand
- ☐ Algae
- ☐ Oil
- ☐ Sewage
- ☐ Micro organisms and Bacteria
- ☐ Other (Specify): ……………..

17. Year of commissioning
- ☐ Future
- ☐ 1998-2000
- ☐ 1995-1997
- ☐ 1990-1995
- ☐ 1980-1990
- ☐ Earlier than 1980

18. Intake Investment costs
- ☐ Less than 5,000 US$
- ☐ 5,000-25,000 US$
- ☐ 25,000-100,000 US$
- ☐ 100,000-500,000 US$
- ☐ 500,000-5,000,000 US$
- ☐ More than 5,000,000 US$

19. Intake Operation and maintenance costs
- ☐ Less than 5,000 US$/yr.
- ☐ 5,000-25,000 US$/yr.
- ☐ 25,000-100,000 US$/yr.
- ☐ 100,000-500,000 US$/yr.
- ☐ More than 500,000 US$/yr.

20. Type of Pre-treatment
- ☐ Coarse filtration
- ☐ Flocculation
- ☐ Coagulation
- ☐ Sedimentation
- ☐ Settling
- ☐ Rapid sand filtration
- ☐ Slow sand filtration
- ☐ Diatomaceous earth filtration
- ☐ Chlorination
- ☐ Other (Specify): ……………..

21. Desalination: Method
- ☐ BWRO
- ☐ SWRO
- ☐ MSF
- ☐ MED
- ☐ VC
- ☐ Other (Specify): ……………..

22. Desalination + Pretreatment plant Investment Cost
- ☐ Less than 10,000 US$
- ☐ 10,000-50,000 US$
- ☐ 50,000-250,000 US$
- ☐ 250,000-1,000,000 US$
- ☐ 1,000,000-10,000,000 US$
- ☐ More than 10,000,000 US$

23. Desalination + Pretreatment plant operation and maintenance cost
- ☐ Less than 10,000 US$/yr.
- ☐ 10,000-50,000 US$/yr.
- ☐ 10,000-50,000 US$/yr.
- ☐ 50,000-250,000 US$/yr.
- ☐ 250,000-1,000,000 US$/yr.
- ☐ More than 1,000,000 US$/yr.

24. Please add more information on performance and costs if available.

……………………………………………
……………………………………………
……………………………………………
……………………………………………
……………………………………………
……………………………

25. Special Problems encountered during operation of Intake and of desalination:

Date: Signature:

Mail List - 1

ABB Sae Sadelmi Spa Piazzale Lodi 3 20137 Milano Italy Attention: Maria Cristina Tel. +39 2 5759 1 Fax. +39 2 5797 7222	Ace Water Treatment Co. Ltd. (AWT) No. 1 Takiguchi-Bldg., Ginza 1-20-7 Chuo-ku Tokyo 104 Japan Tel. +81 3 3564 5771 Fax. +81 3 3564 5698	ACWa Services Ltd. Air, Water & Effluent Treatment Specialists ACWa House, Keighley Road Skipton, North Yorkshire BD23 2 UE United Kingdom Attention: R. Ingham Tel. +44 1756 794 794 Fax. +44 1756 790 898	Aiton and Co. Ltd. Stores Road Derby DE2 4BG United Kingdom Tel. +44 1332 47111 Telex: 37444
Al-Kawther Industries Ltd. P. O. Box 7771 Industrial Estate Jeddah 21472 Saudi Arabia Tel. +966 2 636 0644 Fax. +966 2 637 4337 Telex: 602907 KAWTJER SJ	Austrian Energy and Environment SGO Waagner Biro GmbH Siemensstrasse 89 Vienna 1211 Austria Tel: +43 1 250 454 411 Fax: +43 1 250 452 00	American Engineering Services, Inc. (AES) Water and Wastewater Treatment 5912 Breckenridge Parkway, Ste F Tampa, FL. 33610 U.S.A. Attention: Robert Kadaj Tel: +1 813 621 3932 Fax: +1 813 621 4085	*Alfa Laval Desalt A/S Maskinvej 5 DK-2860 S_borg Denmark Attention: Joachim Schult Tel: +45 39 536 000 Fax: +45 39 536 566

Ambient Technologies Inc. 2999 Northeast 191 Street, Suite 407 North Miami Beach, FL. 33180 U. S. A. Attention: Philip Elovic Tel: +1 305 937 0610 Fax: +1 305 937 2137	Ambient Technologies Inc. 2999 Northeast 191 Street, Suite 407 North Miami Beach, FL. 33180 U. S. A. IDECAN Leon y Castillo 42 Las Palmas 35002 Spain	Aqua-Chem Inc. Water Technologies Division POB 421 Milwaukee, WI 53201 U. S. A. Attention: Jeffrey Miller Tel: +1 414 359 0600, 577 2993 Fax: +1 414 577 2723	Bharat Heavy Electricals Ltd. Corporate Research and Development Div., Central Technical Services Vikasnagar Hyderabad 500 093 India Attention: G. V. Subba Rao Tel: +91 40 279 494 Fax: +91 40 278 320
Biwater International Limited Biwater House, Station Approach, Dorking Surrey RH4 1TZ United Kingdom Tel: +44 1 306 740 740 Fax: +44 1 306 885 233	Cadagua SA Gran V_a, 45 (Edif. Sota), 7.a y 8.a plta 4801 Bilbao Spain Attention: Dr. J. Etxaniz Tel: +34 94 481 73 00 Fax: +34 94 481 73 01	Chemitreat Private Limited 28 Tuas Avenue 8 Singapore 2263 Singapore Tel: +65 861 3603 Fax: +64 861 3853	*Culligan Italiana SpA Cadriano di Granarolo Emilia BO 40057 Italy Attention: L. Coccagna Tel: +39 051 601 7111 Fax: +39 051 765 602

Degremont 183 Avenue du 18 Juin 1940 92508 Rueil Malmaison Cedex France Attention: J. Sennepin Tel: +33 1 46 256 000 Fax: +33 1 42 041 699	Desal Co. Ltd. 48 Par-La-Ville Road, Ste. 381 Hamilton HM11, Bermuda, U. S. A. Attention: Dr. William T. Andrews Tel: +1 441 292 2060 Ext. 11 Fax: +1 441 292 2024	Ebara Corporation Shinagawa Office 1-6-27 Kohnan, Minato-ku Tokyo 108, Japan. Attention: H. Jogan Tel: +81 3 5461 5263 Fax: +81 3 5461 6011	*Fisia-Italimpianti SpA Via di Marini, 16 16149 I-Genova Italy Attention: R. Borsani Tel: +39 010 609 6210 Fad: +39 010 609 6488
GAWA Gesselschaft fuer Automatische Wasseraufbereitung mbH/ Gebr. Heyl GmbH Postfach 41 49 50155 Kerpen, Germany. Tel: +49 223 769 060 Fax: +49 223 769 0669	General Enterprises & Trading Co. Ltd. (GETCO) POB 294, Riyadh 11411 Saudi Arabia. Attention: T. R. Fahim Tel: +966 1 402 3722 Fax: +966 1 402 3856	Hager & Els_sser GmbH POB 800 540 Ruppmannstrasse 22 D-70565 Stuttgart 80 (Vaihingen) Germany. Attention: Kurt Marquardt Tel: +49 711 7866-0 Fax: +49 711 7866-202	Hitachi Zosen Corporation Palaceside Building 7th Floor 1-1-1, Hitotsubashi 1-Chrome Chiyoda-ku, Tokyo 100 Japan. Attention: Mr. K. Oka Tel: +81 3 3217 8520 Fax: +81 3 3212 0914

Mail List - 1 (continued)

HOH Vandteknik A/S Germinvej 24 DK-2670 Greve Denmark. Attention: Peter Sorensen Tel: +45 43 600 500 Fax: +45 43 600 900	Hydropro, Inc. 1346 South Killian Drive Lake Park, FL. 33403-1919 U. S. A. Attention: William K. Hendershaw Tel: +1 561 848 6788 Fax: +1 561 881 0318	*IDE Technologies Ltd. POB 591 13 Zarchin Road Raanana 43 104 Israel. Attention: D. Waxman Tel: +972 9 747 9777 Fax: +972 9 747 9715	Inima Servicios Europeos de Medio Ambiente, S.A. Zurbar_n, 28 28010 Madrid Spain Attention: Emilio Cabrera Tel: +34 91 330 0261 Fax: +34 91 330 0232
Ionics, Incorporated Corporate Headquarters Engineers Society of Western PA POB 9131 65 Grove Street Watertown, MA 02472-9131 U. S. A. Attention: Arthur L. Goldstein Tel: +1 617 926 2500 Fax: +1 617 926 4304	Ishikawajima-Harima Heavy Industries Co. Ltd. Shin Ohtemachin Bldg. 2-16, 3-Chome, Toyosu, Koto-ku Tokyo 135 Japan. Attention: Toshiro Takei Tel: +81 3 244 5541	Kawasaki Heavy Industries Ltd. Industrial Plant Eng. Division World Trade Center Bldg. 4-1 Hamamatsu-cho, 2-Chome Minato-ku, Tokyo Japan. Attention: K. Kaneko Tel: +81 3 3435 2406 Fax: +81 3 3436 2986	Kurita Water Industries Ltd. 4-7 Nishi-Shinjuku 3-Chome, Shinjuku-ku Tokyo 160 Japan. Attention: M. Kusano Tel: +81 3 3347 3194 Fax: +81 3 3347 3099

Matrix Desalination, Inc. 3255 S. W. 11th Avenue Fort Lauderdale, FL. 33315 U. S. A. Attention: Whitney E. Tel: +1 954 524 5120 Fax: +1 954 524 5216	METITO Arabia Industries Ltd. POB 6133 Prince Musaed Bin Abdul Aziz Street Riyadh 11442 Saudi Arabia. Attention: Mr. Kassem Mazloum Tel: +966 1 478 7001 Fax: +966 1 479 4250	Mitco Water Laboratories POB 1699 Winter Haven FL 33882-1699 U. S. A. Attention: Mike Wethern Tel: +1 813 967 4456 Fax: +1 813 967 7475	Mitsubishi Heavy Industries Ltd. Machinery Headquarters, Plant Department 5-1, Marunouchi, 2-Chome Chiyoda-ku, Tokyo 100 Japan. Attention: Kenichiro Fuji Tel: +81 3 3212 3111, -9652 Fax: +81 3 3212 9669
*Mitsui Engineering and Ship Building Co. Ltd. Energy Plant Division 6-4, Tsukiji 5-chome, Chuo-ku Tokyo 104 Japan. Attention: Yoshihisa Shibata Tel: +81 3 3544 3306 Fax: +81 3 3544 3051	Nippon Koei Co. Ltd. Research and Development Center 2304 Takasaki, Kukizaki-cho, Inashikigun, Ibaraki 300-12 Japan. Attention: M. Murakami Tel: +81 298 2045 Fax: +81 298 71 2022	Japan Organo Co. Ltd. Central Research Laboratories 4-9 Kawagishi, 1-Chome, Toda City, Saitama Pref. 335 Japan. Attention: Akira Mizuuchi Tel: +81 484 46 1881, -46 1397 Fax: +81 484 46 1966, -43 9060	Osmo Sistemi SrL Water Treatment Technologies Via Tonlolo 8/B 61032 FANO (PS) Italy Attention: Nava Marco Tel: +39 0721 855 023 Fax: +39 07 21 85 005

Osmonics Inc. 5951 Clearwater Drive Minnetonka, MN 55343-8995 U. S. A. Attention: D. Dean Spatz Tel: +1 612 933 2277 Fax: +1 612 933 0141	Petrolite Equipment & Instruments Group POB 2546 Houston, TX 77252 U. S. A. Attention: James Chen Tel: +1 713 926 7431 Fax: +1 713 926 1162	Sasakura Engineering Co. Ltd. 7-32 Takeshima 4-Chome Nishiyodogawa-ku, Osaka 555 Japan. Attention: Toshihiko Sasakura Tel: +81 6 473 2131 Fax: +81 6 475 2899, 473 4558	SeaTec Systems, Inc. 1516 Cypress Drive Jupiter, FL 33469 U. S. A. Attention: Paul L. Culler Tel: +1 561 744 0533 Fax: +1 561 744 5569
Setec, Separation Technologies BV POB 90 2250 AB Voorschoten The Netherlands Attention: Ir. G.A.T. Schroder Tel: +31 71 561 4940 Fax: +31 71 561 8180`	Snamprogetti SpA Dept. Aqua Viale A. de Gasperi, 16 20097 San Donato Milanese Milan, Italy. Attention: A. Bonetti Tel: +39 02 520 9808 Fax: +39 02 520 38524	Servicios y Procesos Ambientales, S.A. (SPA) Agustin de Fox 8, 29, ___ plta 28036 Madrid Spain. Attention: Enrique Carrasco Tel: +34 91 9323 1407 Fax: +34 91 9323 0880	Steinmueller-Rompf Wassertechnik GmbH & Co. Fabrikstrasse 1 51641 Gummersbach Germany. Attention: Andras Friedrich Tel: +49 2261 85 0 Fax: +49 2261 85 3239

Mail List - 1 (continued)

Tedagua Tecnicas de Desalinizacion de Agua S.A. Polig. Industrial Arinaga c/Brenzo, 15 y 17 35119 Arinaga, Gran Canaria Spain. Attention : J. L. Loidi Arregui Tel : +34 928 754 518 Fax : +34 928 754 565	Termomeccanica SpA Via del Molo 1 19126 La Spezia Italy. Attention : A. Sammartano Tel : +39 0187 552 111 Fax: +39 0187 552 267	Thyssen Engineering GmbH Desalination and Water Technology POB 103854, Am Thyssenhaus 1 4300 Essen 1 Germany. Attention: Helmut Lang Tel: +49 201 106 3532 Fax: +49 201 2233 64	Toray Engineering Co. Ltd. Nihonbashi Muromachi Bldg., 6-3 Chome, Nihonbashi-Hongoku-cho Chaujo-ku, Tokyo 103 Japan. Attention: Shingo Miyata Tel: +81 3 241 1591
UNIHA Wasser & Abwasseraufbereitung GmbH Stockhofstrasse 19, A-4020 Linz Austria Attention: R. Ochsner Tel: +43 732 663 837 Fax: +43 732 661 870	US Filter Corp. POB 560 4669 Shepherd Trail Rockford, IL 61105-0560 U. S. A. Attention : H. L. Pelegrin Tel: +1 815 877 3041 Fax: +1 815 633 5906, 877 0172	Weir Westgarth Ltd. Head Office Weir Group Alexandra Court, 27, 29 Denmark St. Wokingham, Berkshire, RG11 2AY United Kingdom. Attention: A. G. Turner Tel : +44 1734 792 201 Fax : +44 1734 775 276	Zenon Environmental Inc. 845 Harrington Court Burlington, ONT L7N 3P3 Canada. Attention: Dr. A. Benedek Tel: +1 905 639 6320 Fax: +1 905 639 1812

Mail List – 2

Ansaldo Energia SpA Piazza Cariganano 2 16152 Genova Italy. Attention : A. Eusepi Tel : +39 10 655 7373 Fax: +39 10 655 7297	Aqua Engineering Mpserstrasse 29 5020 Salzburg Austria Attention : H. Laimer Tel: +43 662 4088 0 Fax: +43 662 4088 188	Asahi Glass Co. Ltd. Chemical Engineering Division 2-25-14, Kameido Koutou-ku, tokyo 136 Japan. Attention: Takuo Kawahara Tel: +81 3 3636 9252 Fax: +81 3 3636 9254	Battelle Memorial Institute 505 King Avenue Columbus, OH 43201-2693 U. S. A. Attention: D. E. Olesen Tel: +1 614 424 6424 Fax: +1 614 424 5263
Bechtel National Inc. Washington Executive Office 9801 Washingtonian Blvd. Gaithersburg, MD 20878-5356 U. S. A. Attention: J. Christopher Judd Tel: +1 301 417 4545 Fax: +1 301 990 1642	*Cayman Water Co. Ltd. POB 1114 GT Grand Cayman, Cayman Islands British West Indies. Attention: J. M. Parker Tel: +1 809 947 4277, 744 09 (Plant) Fax : +1 809 947 4191	*Christ Ltd. POB 130, Haupstrasse 192 CH-4147 Aesch Switzerland. Attention: H. Uebersax Tel: +41 161 708 111 Fax: +41 161 784 485	*Curacao Water & Power Corporation Kompania di Awa Electrisidat di Korsou NV POB 2097, Rector Zwijsenstraat 1 Curacao, Netherlands Antilles Attention: G.J. M. Governeur Tel: +599 9 623 1881 Fax: +599 9 626 685

Deutsche Ver Fahrenstechnik (DVT) Wallbergstrasse 7 8028 Taufkirchen b. Mchn. Germany Attention : Dipl. Ing. Hussein K. Morsy Tel: +49 89 612 8094-99 Fax: 49 89 612 5669	DHV Water BV POB 484 3800 AL Amersfoort The Netherlands. Attention: Prof. J. C. van Dijk Tel: +31 33 468 2360 Fax: +31 33 468 2301	*DuPont Co. Permasep Products Wilmington, EE 19898 U. S. A. Attention: F. A. Salem Tel: +1 302 451 3228, 9938, 3207 Fax: +1 302 451 9686	Emco Engineering Inc. Water & Wastewater Treatment Systems 25 North Street Canton, MA 02021 U. S. A. Attention: Bassam E. Shalhoub Tel: +1 617 828 7340 Fax: +1 617 828 1714
GKSS Forschungszentrum Geesthacht GmbH Institute of Chewmistry POB 1160 21494 Geesthacht Germany. Attention: Dr. Albrecht Tel: +49 4152 87 2461 Fax: +49 4152 87 2444	Hubert BV Kooyweg 20 8715 EP Stavoren The Netherlands. Tel: +31 5149 1625 Fax: +31 5149 2198	*Hydranautics Water Systems 401 Jones Road Oceanside, CA 92054 U. S. A. Attention: D. Date Tel: +1 619 901 2500, 2521 Fax: +1 619 901 2578	Ion Exchange (India) Ltd. Tiecicon House POB 6273 Dr. E. Moses Road Mahalaxmi, Bombay 400 011 India. Attention: Satish N. Chilekar Tel: +91 22 493 9520/23/25 Fax: +91 22 493 8737 E-Mail:- ionxchng@giasbm01.vsnl.net.in

Kobe Steel Ltd. Kobelco Tekko Building, 8-2 Marunouchi, 1-Chome Chiyoda-ku, Tokyo 100 Japan. Attention: Tetsuo Matsumura Tel: +81 3 218 7792 Fax: +81 3 218 6592	Polymetrics Seawater Systems, Inc. 550 So. Winchester Blvd., #410 San Jose, CA 95128 U. S. A. Attention: M. F. Lamendola Tel: +1 408 983 2684 Fax: +1 408 261 3041	Preussag Noell GmbH D-97064 Wurzburg Germany. Tel: +49 931 903 0 Fax: +49 931 903 1000	PWT Projects Limited 632-652 London Road Isleworth, Middlesex, TW7 4EZ United Kingdom. Attention: Charles Macrorie Tel: +44 181 560 5199 Fax: +44 181 568 5479
Reliable Water Company, Inc. 209 Harvard St., Suite 207 Brookline, MA 02146 U. S. A. Tel: +1 617 670 2300 Fax: +1 617 663 5060	ROI Technology Reverse Osmosis International POB 20035, Thessaloniki 55110 Greece. Attention: Dr.. Skaldis Tel: +30 31 996 168 Fax: +30 31 996 168	SIDEM - Societe Internationale de Dessalement 75009 Paris France. Tel: +33 1 4285 3648 Fax: +33 1 4995 7695 Telex: 280415 F	Sowit - Division of De Cardenas Via Pio La Torre, 14 20090 Vimodrone (MI) Italy. Attention: dott. Ing. Marco Rognoni Tel: +39 2 265 0585, 265 10021 Fax: +39 2 250 5121, 265 0941

Mail List - 2 (continued)

Tedagua Tecnicas de Desalinizacion de Aguas, S.A. Polig, Industrial Arinaga, C/Brezo, 15 y 17 35119 Arinaga, Gran Canaria, Spain. Attention: J. L. Loidi Arregui Tel: +34 28 754 518 Fax: +34 28 754 565	Thames Water International Nugent House (PBH3) Vastern Road Reading, Berkshire RG1 8DB United Kingdom. Attention: Dr. Emilio Gabbrielli TEl: +44 1734 593 371 Fax: +44 1734 593 300, 557 526	Water Equipment Technologies Inc. (WET) 832 Pike Road West palm Beach, FL 33411 U. S. A. Attention: Jorg Menningmenn Tel: +1 407 684 6300 Fax: +1 407 697 3342	Water Services of America Inc. POB 23848 8165 West Tower Avenue Milwaukee, WI 53233 U. S. A. Attention: Don A. Dieke Tel: +1 414 354 6470

Additional contacts were made with the following companies and persons:

Mail List - 3

Seawater Reverse Osmosis (SWRO) Plants with Seawater/Beach wells

RO Plant	Location	Capacity (m^3 per day)	Contact person/ results
Taba Riviera	Taba, Egypt	2000-9000	Mr. Ibrahim Elwan constructed by Nov 2000 971-4-883-969
Mullet Bay, St. Maarten	Netherlands Antilles	1514	Mr. Thomas Kelley/ not contacted
Pelican Resort, St. Maarten	Netherlands Antilles	208	Mr. Carlos Alvarado/ not contacted 011-5995-42503
Cappoons Bay, Govt, Tortola	British Virgin Islands	568	Mr. Mario Mundo 284-494-6634
Royal Antiguan Resort	Antigua, West Indies	182	Mr. Allister Forrest 268-462-3733
Curtain Bluffs Resort	Antigua, West Indies	91	Mr. Robert Sherman 268-462-8400
Jolly Harbor Resort	Antigua, West Indies	454	Mr. Franz Bigler 268-462-3084
Malliouhana Hotel	Anguilla	201	Mr. Leon Roydon 264-497-6111
Coccoloba Plantation	Anguilla	95	Mr. Ethelbert Edwards 264-497-5849
Cap Juluca Hotel	Anguilla	935	Mr. Kerry Knotts 264-497-6666
Cove Castles	Anguilla	20	Ms. Sylvine Petty 264-497-6801
Great House	Anguilla	38	Mr. Walton Flemming 264-497-6061
Club Med Paradise Island	Bahamas	303	Mr. Abbaoui Jilil 242-363-2675
Sheraton	Bahamas	227	Mr. Stewart Culmer 242-363-7000

The following people were contacted by postal or electronic mail:

- Sulaiman Al-Matawa, Kuwait
- Mohamad K. Nemer, Abu Dhabi
- Mahmoud Al Hadidi, Jordan
- Maroun Aoun, Ionics

- A.N. Al-Bastaki, Bahrain
- Abdul- Raouf Sharshar, Qatar
- Taher Al Ghasham, Saudi Arabia
- Abd Al Qader Ben Younis, Tunissia
- B.A. Abd Al- Latif

Only a few additional responses have been received.

APPENDIX - B

MODEL OF THE DYNAMICS OF SEAWATER / INTRUSION INTO COASTAL AQUIFERS

Contents

1. SALT WATER INTRUSION IN COASTAL AQUIFERS AND ITS DEPENDENCE ON THE ABSTRACTION OF FRESH WATER AND OF SEAWATER*

The density of the seawater is greater than the density of fresh water due to the difference in the content of dissolved solids. As a result, the saline water at the bottom of the sea usually encroaches into the fresh water in the coastal aquifer with a steady or abrupt decrease in the content of the dissolved solids. The mixture of the fresh and salt water creates a zone with a salinity gradient, which is in a state of dynamic equilibrium under natural conditions.

The landward encroachment of seawater is limited by the flow of fresh water towards the sea that usually exists in coastal aquifers. The natural discharge of fresh water causes a steady return to the sea of part of the saltwater flow, producing a cyclic flow of the salt water in the aquifer. The shape and position of the interface or transition zone between the saline water and the fresh water is a function of the volume of fresh water discharging from the aquifer. Any changes of the discharging fresh water volume results in a new dynamic equilibrium position of the boundary. Groundwater extraction lowers the water table surface; causing a decrease in the amount of fresh water discharging to the sea, thereby permitting the intrusion of salt water into the fresh water parts of the aquifer. This type of encroachment sometimes takes a long time to move a significant distance before it eventually reaches the new dynamic equilibrium position. In cases of active withdrawal of groundwater near coastal areas, the saltwater intrusion problems are considerably more severe because the natural hydraulic gradient has been reversed and fresh water is actually moving away from the sea. In this case, the interface between fresh and salt water is moving inland very rapidly towards the low point of the hydraulic gradient, *i.e.* the point of pumping.

Similarly, when the amount of fresh water discharging into the sea increases due to substantial man made recharge, the interface moves seawards. Thus, the aim of any water management strategy for a coastal aquifer is to arrange the active groundwater withdrawal and recharge, in terms of locations and flow rates such that the required amount of fresh water can be produced with a minimum of saltwater intrusion.

Seawater extraction from the saltwater zone due to Beach wells acts contrary to the extraction of fresh water. It reduces water pressures in the seawater zone and reduces seawater intrusion.

Mercado and Kally (1996), show that along the coast of Israel, Beach wells pumping seawater may contribute to the management of the fresh water aquifer by enabling the reduction of the flow of fresh water lost to the sea, which is usually required to keep fresh water levels to bar seawater intrusion. Seawater intrusion may be avoided by

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* This section and the following ones are based on the KTH, 1999 Report. The last sections are based on Mercado and Kally (1996).

seawater Beach wells that would reduce the pressure and hydraulic head in the seawater underground.

2. ANALYTICAL BACKGROUND

Seawater intrusion is a phenomenon that has been the subject of analytical solutions and numerical simulations. This section reviews some simple analytical solutions developed to assess the position of the fresh water-seawater interface. One of the major assumptions common to all studies presented below is that the two fluids *i.e.* seawater and fresh water are considered immiscible and therefore separated by a sharp interface. This assumption is valid when the width of the transition zone, *i.e.* the zone where the content of dissolved solids in the groundwater changes between seawater to fresh water levels is relatively small compared with the thickness of the aquifer. Furthermore, the analytical expressions that follow are only valid for assessing the interface position under steady-state conditions.

Among the earliest studies on salt- and fresh water interaction conducted by Ghyben (1889) and Herzberg (1901), the analytical expression that resulted from their studies is commonly referred to as the Ghyben-Herzberg formula. It is derived through simple hydrostatics: the weight of the column of fresh water of length hf + hs equals the weight of the column of salt water of length hs (Figure 1). The hydrostatic balance can be expressed as

$$h_s = \frac{\rho_f}{\rho_s - \rho_f} h_f = \delta h_f \qquad (2.1)$$

where:

h_f	-	fresh water table above sea level
h_s	-	depth of interface below sea level
ρ_s ρ_f	-	densities of seawater and fresh water
δ	-	density ratio 40: for ocean water, 35 for Mediterranean water

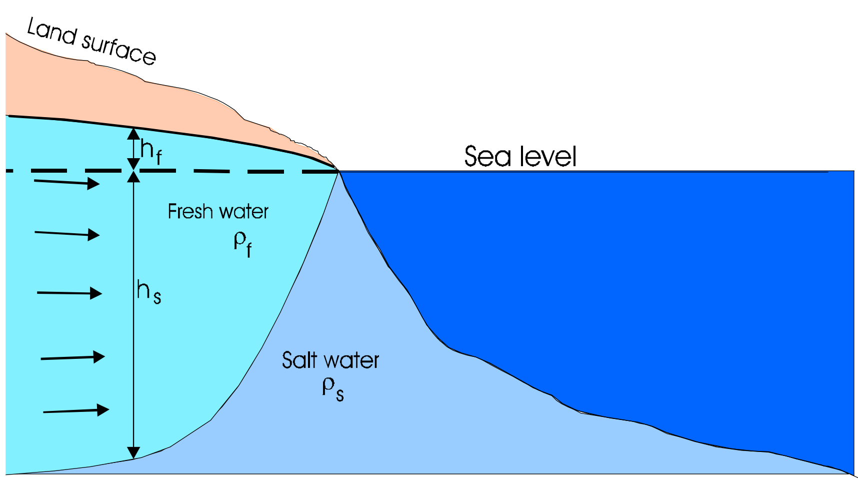

Figure B.1: Illustration of the Ghyben-Herzberg principle

The ratio of $h_s = 40\ h_f$ was confirmed by Ghyben and Herzberg through observations.

Figure B.2, shows the interface position using Ghyben-Herzberg formula for the two different seawater densities, assuming a linear shape of the interface. (This assumption is correct when no natural replenishment occurs into the aquifer above the interface.) The distance from the shoreline to the toe of the interface can be approximated assuming that Darcy's law is valid in the region above the interface (cf., Domenico and Schwartz, 1997). This implies that the discharge per unit length of shoreline is given by $Q_o = KBh_L/L$, where K is hydraulic conductivity, B is aquifer thickness, and h_L is the hydraulic head at the seawater interface at a distance L from the shoreline. Using the Ghyben-Herzberg formula (2.1) for h_L, the intrusion length can be estimated as:

$$L = \frac{KB^2}{Q_0 \delta} \tag{2.2}$$

The above equation shows that the length of seawater intrusion is directly proportional to the hydraulic conductivity and the squared thickness of the aquifer. Since the length of intrusion is inversely proportional to the flow of fresh water to the sea, the reduction of that flow, due to the pumping near shoreline, will increase the length of salt-water intrusion. As shown below, the solution of the equation for steady flow above the interface with natural recharge yields a parabolic shape of the interface. However, L is proportional to K and B^2, and inversely proportional to the fresh water discharge.

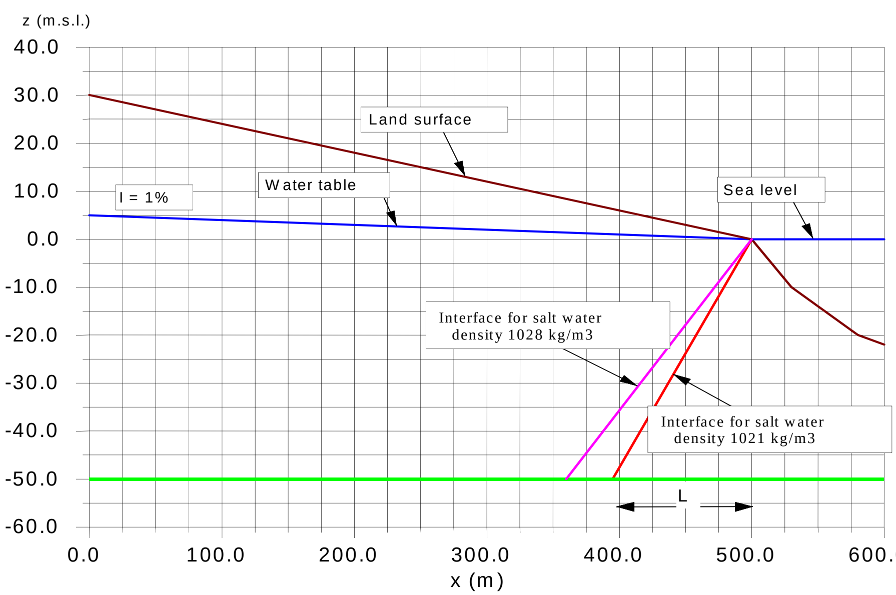

Figure B.2: Interface position using Ghyben-Herzberg principle for two seawater densities without natural recharge[1]

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[1]Figure B.2 trough 6 have been obtained assuming the following data: B = 50 m, i = 1%, a = 250 m, $K = 10^{-4}\ m.s^{-1}$ and N = 0.

A particular formulation of the flow equation, which accounts for a constant natural recharge using the Dupuit assumption and the Ghyben-Herzberg principle was given by Fetter (1972):

$$\frac{\partial^2 h_f^2}{\partial x^2} + \frac{\partial^2 h_f^2}{\partial y^2} = \frac{-2N}{K(1+\delta)} \tag{2.3}$$

where N is the recharge to the aquifer [length/time]. The above equation can be solved for an infinite-strip coastline, yielding

$$h_f^2 = \frac{N[a^2 - (a-x)^2]}{K(1+\delta)} \tag{2.4}$$

where a is half width over which recharge applies and h_f is the head of the water table at distance x from the shoreline. Figure B.3 shows the interface for two levels of salt water density, $\rho_s = 1.021\ kg.l^{-1}$ and $\rho_s = 1.028\ kg.l^{-1}$, using (4.4). The denser fluid has the stationary interface more upward inland and hence a larger intrusion length.

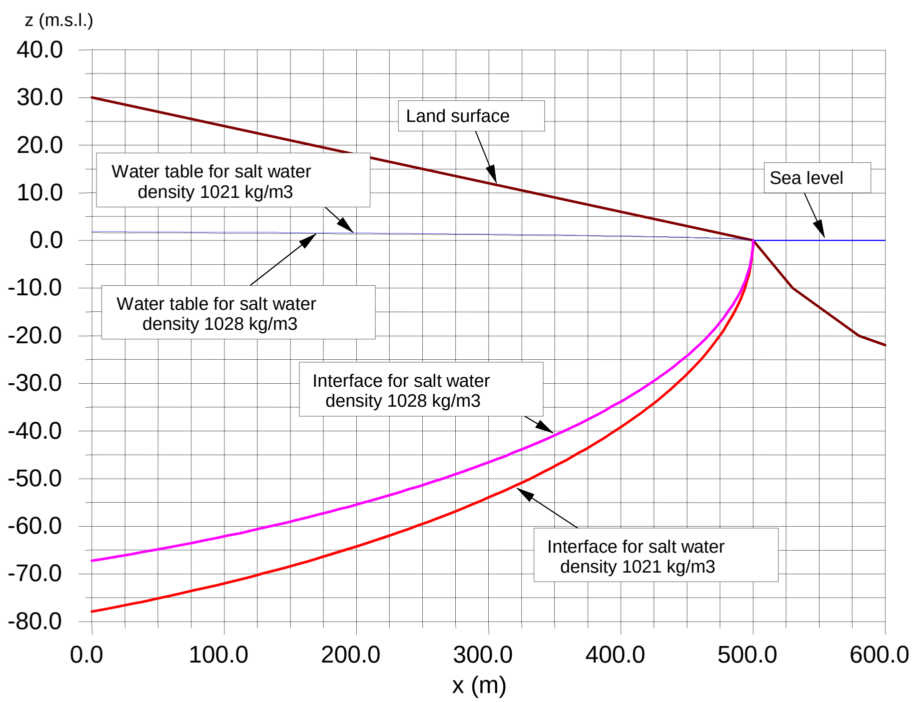

Figure B.3: Solution based on Fetter (1972) for two seawater densities

A key disadvantage of the above models is that the interface intercepts the water table at the coastline. This is a consequence of the Dupuit assumption of horizontal flow, implying that fresh water discharge into the sea takes place at a point rather than along a seepage face as known from field observations. To account for the vertical flow components in the region where fresh water discharges into the sea, Glover

(1964) developed a simple model that allows a seepage face at the coastline. The expression for the interface coordinates starting from the coastline is:

$$\zeta = \frac{Q_0\delta}{K} + \sqrt{\frac{2Q_0\delta\, x}{K}} \tag{2.5}$$

where ζ is a vertical coordinate which is zero at sea level and increases in the negative z direction, and Q_0 is the discharge from the aquifer at the coastline per unit width. It is clear from the above that when x = 0, ζ will be non-zero, allowing a finite seepage face. The width and thickness of the seepage face are given by:

$$\zeta_0 = \frac{Q_0\delta}{K} \qquad\qquad x_0 = \frac{-Q_0\delta}{2K} \tag{2.6}$$

The height of the water table above sea level at any distance x from the coast is given by:

$$h_f = \sqrt{\frac{2Q_0 x}{K\delta}} \tag{2.7}$$

Figure B.4 presents the solution based on (4.5) for two levels of seawater density.

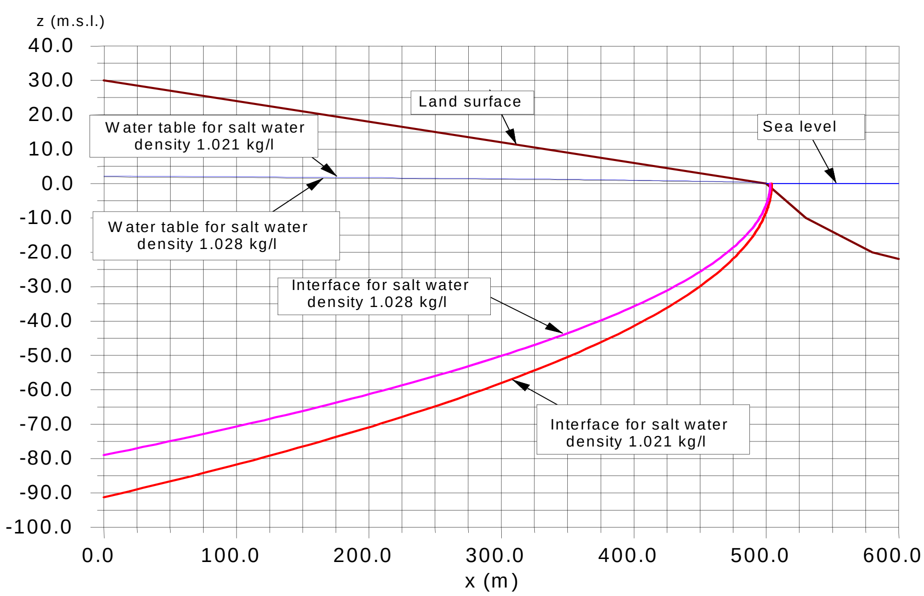

Figure B.4: Solution based on Glover (1964) for two seawater densities

The analytical solutions by Fetter (1972) and Glover (1964) differ in the fact that the former assumes a constant uniform recharge over the aquifer, whereas, the latter assumes a uniform discharge in the aquifer towards the sea. Both solutions are displayed in Figure B.5 for ρ_s =1.028 $kg.l^{-1}$, a salinity level usually considered for the

Mediterranean Sea. Figure B.6, shows the same solutions in an enlarged seepage face detail near the coastline.

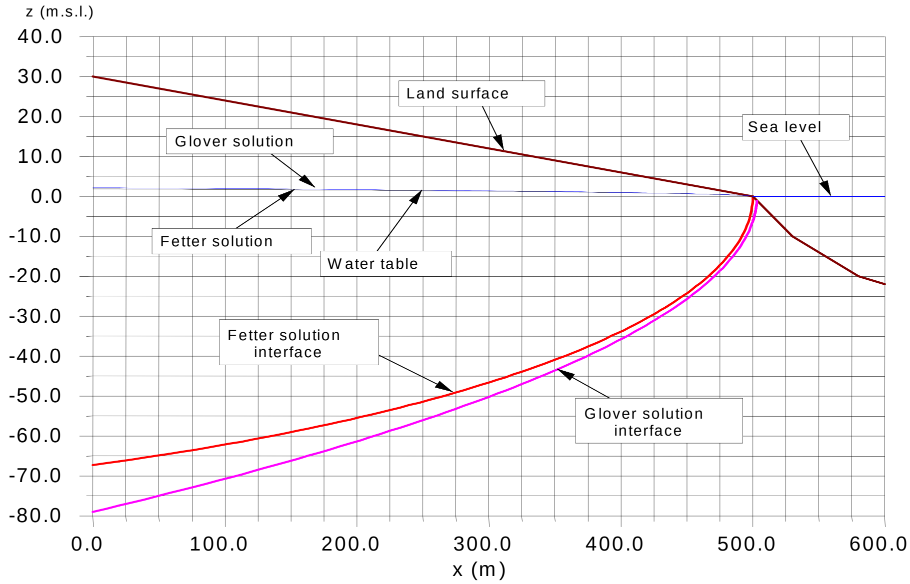

Figure B.5: Comparison between Fetter's and Glover's solution for typical Mediterranean Sea density

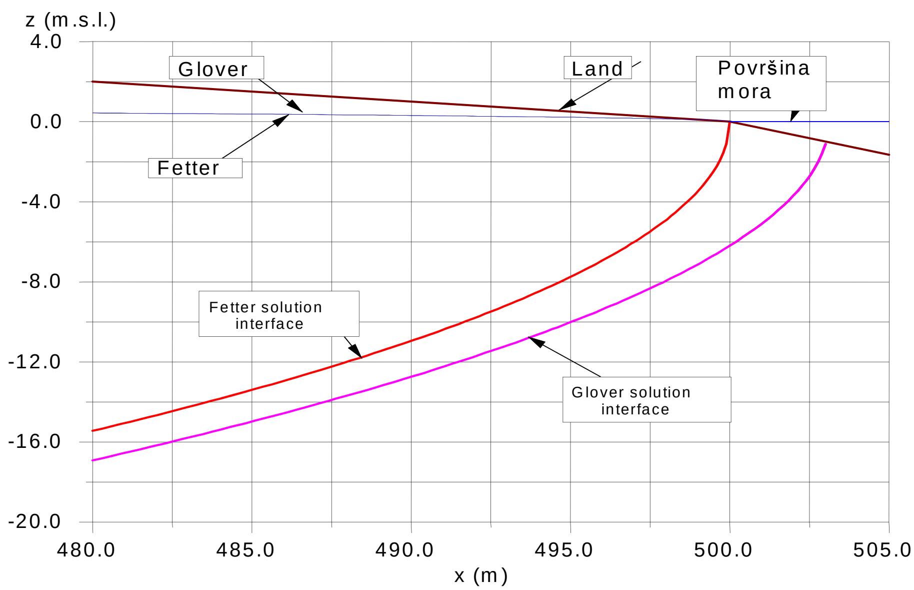

Figure B.6: Seepage face resulting from the solution by Glover (1964)

3. AVAILABLE ANALYTICAL SOLUTIONS FOR THE EFFECTS OF NATURAL REPLENISHMENT AND EXTRACTION WELLS

3.1. Assumptions in analytical solutions

Simple analytical solutions of the seawater intrusion problem are usually based on the following assumptions (*e.g.* Bear, 1979; Domenico and Schwartz, 1997; Fetter, 1994):

1. The seawater and the freshwater are separated by a sharp interface, and the solutions are valid within the region where the interface is present
2. The seaward flow of freshwater is steady, whereas seawater is immobile
3. The Dupuit assumption and the Ghyben-Herzberg relation are applicable, implying horizontal flow, no seepage face, and a hydrostatic relation between interface position and hydraulic head (Equation 3.1)
4. Flow is one-dimensional in a cross-section perpendicular to an infinite coastline and driven by a natural gradient and/or natural recharge; point sources or sinks are not considered
5. The hydraulic properties of the aquifer are homogeneous, *i.e.* the hydraulic conductivity and porosity associated with freshwater flow are constant in space

As discussed in Section 2, the assumption of horizontal flow was relaxed by Glover (1964), who developed a solution that allows a seepage face along the coastline. Furthermore, the vertical flow that is induced when pumping takes place above the interface, and the associated instability of the interface has been the subject of a number of studies (Bear, 1979; Various authors, 1995).

In the remainder of this section, discussed is how assumptions 4 and 5 above can be relaxed when solving the flow problem analytically. Specifically, consider an analytical solution that incorporates a pumping well inland from the region with the interface (Strack, 1976), and an analytical stochastic model that accounts for heterogeneity in the hydraulic conductivity (Dagan and Zeitoun, 1998). The solution described in Section 2; will be redefined to enable their further development.

3.2. Steady interface in a homogeneous aquifer

Consider the unconfined aquifer illustrated in Figure B.1 and let $x = 0$ at the coastline, *i.e.* at the discharge point. Under the assumptions listed in the previous section, the solution for the interface position or the water table in the aquifer is obtained from a simple water balance in the region above the interface. With $h_s(x)$ and $h_f(x)$ being the vertical distances from the sea level to the interface and the water table, respectively (h_f is the hydraulic head relative to sea level and $h_s + h_f$ is the total thickness of the flow domain), the balance equation can be written as (Bear, 1979)

$$q_k = K\left(h_s + h_f\right)\frac{dh_f}{dx} \qquad x \le L \tag{3.1}$$

Where q_k is the discharge per unit length perpendicular to the section (positive in the seaward direction), and L is the toe of the interface; that is, the position in the x direction where the interface reaches the bottom of the aquifer. Application of the Ghyben-Herzberg relationship, $h_s = h_f \delta$ where $\delta = \rho_f /(\rho_s - \rho_f)$, and the boundary condition $h_f = 0$ at $x = 0$, yields the solution.

$$q_k = \frac{Kh_f^2}{2x}\left(1+\delta\right) = \frac{Kh_s^2}{2x}\frac{1+\delta}{\delta^2} \qquad x \le L \tag{3.2}$$

This shows that the interface and the water table in the aquifer have a parabolic shape. The toe of the interface is given by:

$$L = \frac{KB^2}{2q_0}\frac{1+\delta}{\delta^2} \tag{3.3}$$

Where *B* is the depth of the aquifer below sea level and q_o is the flow above the toe. As discussed above, *L* increases when *K* and *B* increase, and when q_o decreases.

The water balance (3.1) is readily extended to include a constant, uniformly distributed natural recharge. Since the total discharge towards the sea varies with *x* in this case, q_o is defined as the discharge above the toe of the interface. With *N* denoting the natural recharge (flow rate per unit horizontal area), the water balance takes the following form (cf., Bear, 1979):

$$q_0 + N(L - x) = K(h_s + h_f)\frac{dh_f}{dx} \qquad x \le L \tag{3.4}$$

Integration with the same boundary condition as above, and use of the Ghyben-Herzberg relationship with $h_f = B/\delta$ at $x = L$, leads to the following expression for *L*:

$$L = \frac{Q_0}{N}\left[\left(1 + \frac{KB^2 N}{Q_0^2}\frac{1+\delta}{\delta^2}\right)^{1/2} - 1\right] \tag{3.5}$$

To illustrate the sensitivity of *L* to the various hydrogeological parameters in (3.3) and (3.5), we express the discharge as $Q_0 = KB(1+1/\delta)i$, where *i* and $B(1+1/\delta) = B(1+\delta)/\delta$ are the hydraulic gradient and the thickness of the aquifer at $x = L$, respectively. Some results obtained with different values of *i*, *K* and *N*, and the constant values *B* = 50 *m* and δ = 48 (which corresponds to $\rho_f = 1.00\, kg/l$ and $\rho_s = 1.021\, kg/l$) are shown in Table B.3.1. Note: that if there is no recharge, the present formulation of Q_0 implies that *L* is inversely proportional to *i* and independent of *K* (cf., Equation 3.3).

Table B 3.1: Sensitivity of the maximum intrusion length, L (metres) to hydrogeological parameters

Hydraulic Gradient above Toe	N = 0 Any K	N = 0.25 m.yr^{-1} K = 10^{-4} m.s^{-1}	N = 0.25 m.yr^{-1} K = 10^{-5} m.s^{-1}
I = 0.01	52.1	51.9	50.1
i = 0.001	521	398	203

Reference Table B.3.1, the natural recharge causes a decrease in *L*. However, if *i* is large the effect of the recharge is small. This is because the total seaward flow of freshwater is dominated by the gradient-driven groundwater flow, rather than by the natural recharge. Furthermore, for fixed *i* and $N \neq 0$, *L* increases with *K*. A comparison with Figure B.8 shows that the results for *i* = 0.01 are approximately in agreement with those obtained in the numerical simulations.

3.3. Effects of extraction wells on the interface position

The discharge Q_0 above the toe of the interface in the above analysis is the net discharge resulting from all recharge and discharge in the aquifer inland from the toe. Thus, Q_0 and the solutions presented above also account for the presence of pumping and recharge wells, provided they are located inland from the interface region. Furthermore, the flow induced by the wells must fulfil the assumption of horizontal, one-dimensional flow. Strictly, this implies an infinite number of fully penetrating wells arranged at close spacing along a line perpendicular to the considered cross-section. We denote the *x* coordinate (that is, the distance from the coastline) of the well line as x_p, whereas Q'_p is the pumping rate per unit length and Q_0 is defined as the uniform discharge in the aquifer if no wells are present (natural recharge is neglected).

First, it may be observed that if $Q_0 < Q'_p$, the flow in the whole region $x < x_p$ is directed towards the wells. Therefore, the interface advances to the wells under these conditions. If $Q_0 > Q'_p$, it depends on the net discharge, $Q_0 - Q'_p$, whether the interface reaches the wells. That is, the position of the interface toe, evaluated from (3.3) with the net discharge, indicates whether a certain pumping location is within the interface region. By setting $L = x_p$ in (3.3), the critical situation for comparative purposes may be expressed as a simple relation between dimensionless parameter combinations:

$$\lambda = 1 - \nu \tag{3.6}$$

where

$$\lambda \equiv \frac{KB^2}{Q_0 x_p} \frac{1+\delta}{\delta^2}; \qquad \nu \equiv \frac{Q'_p}{Q_0} \qquad Q'_p < Q_0$$

If the parameters governing natural flow are known, equation (3.6) can be used to evaluate the minimum distance from the coastline required for extraction of a given pumping rate. Alternatively, (3.6) may be used to evaluate the maximum pumping rate that can be extracted at a given location. In both cases, the design criterion then, is that the interface toe does not reach the line of extraction wells. However, pumping in shallow wells above the interface may be a viable alternative. In which case, the critical pumping rate and well depth are determined through an analysis of the stability of the interface (Bear, 1979).

In many cases of practical significance, the above assumptions concerning the well configuration are not applicable. In particular, when water is extracted from a single well or from a system of wells arranged in some irregular configuration, the two-dimensional nature of the flow field in the horizontal *x,y* plane cannot be ignored. Strack (1976) developed analytical solutions for flow in coastal aquifers with a single extraction well. The approach is based on the formulation of a single potential, which is valid both in the region above the interface and in the part of the aquifer where no seawater is present.

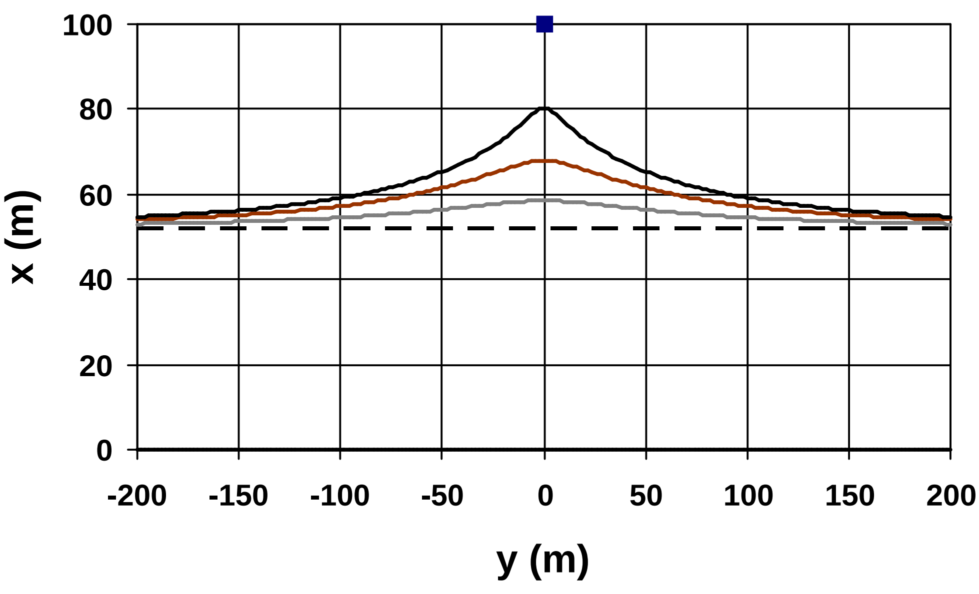

Figure B.7: Position of the toe of the interface (Eq: 3.7) for different pumping rates, represented by the γ values 0 (no pumping, constant x co-ordinate of the toe), 30, 60 and 80 m (top curve); the well is located at $x_p = 100$ m along the *y* axis

The total potential associated with the combination of a uniform discharge towards the sea and an extraction well at $(x = x_p, y = 0)$ is obtained through superposition. Specifically, the co-ordinates of the toe of the interface in an unconfined aquifer may be obtained from the following equation (Strack, 1976):

$$\frac{KB^2}{2Q_0}\frac{1+\delta}{\delta^2} = x + \frac{Q_p}{Q_0}\frac{1}{4\pi}\ln\left[\frac{(x-x_p)^2+y^2}{(x+x_p)^2+y^2}\right] \qquad (3.7)$$

Where Q_p is the volumetric pumping rate and the LHS is equal to L as given by Equation (3.3). The solution (3.7) is illustrated in Figure B.3.1, where $\gamma = Q_p / Q_0$ (units of length), $x_p = 100\,m$ and, as in Section 3.2, $Q_0 = KB(1+1/\delta)i$ with $B = 50\ m$ and δ = 48. In the present case, i is the hydraulic gradient above the toe before pumping started, and far away from the well during pumping. It can be seen in the figure that the leading point along the curved toe of the interface (y = 0) approaches the well as γ increases.

As discussed in some detail by Strack (1976) and Bear (1979), the definition of the critical situation when the interface toe reaches the well is somewhat different in the case of a single well, compared with the simple one-dimensional situation described above. Essentially, pumping creates a cone of depression in the x, y plane, with a stagnation point, x_s, where the x- and y-components of the discharge are zero, along the y axis seaward of the well. The stagnation point is located along the water divide that defines the region supplying water to the well; hence, $x < x_s$ implies that flow is directed towards the sea, whereas the flow direction is towards the well in the region $x_s < x < x_p$.

It follows that the critical situation occurs when the toe of the interface goes through x_s, which can be expressed by the equation (Strack, 1976):

$$\lambda = 2\left(1-\frac{\mu}{\pi}\right)^{1/2} + \frac{\mu}{\pi}\ln\left[\frac{1-(1-\mu/\pi)^{1/2}}{1+(1-\mu/\pi)^{1/2}}\right] \qquad (3.8)$$

$$\mu \equiv \frac{Q_p}{Q_0 x_p}$$

where λ is defined in connection with Equation (6.6), and is a dimensionless pumping rate similar to ν in (3.6). Equation (3.8) is illustrated in Figure B.8. Similar to the previous case, the relevant design issues that can be addressed by means of (3.8) include to determine the location for a given pumping rate and *vice versa*. Some useful limits can be obtained from the fact that μ is restricted to the interval $0 \le \mu \le \pi$; that is, $x_p \ge Q_p /(Q_0\pi)$ or $Q_p \le x_p Q_0 \pi$.

Bear (1979) also discusses the use of artificial recharge to prevent the interface from advancing to the extraction well. The hydraulic head above the toe of the interface,

$h_f = B/\delta$, is an important design variable in this context, since it constitutes the minimum head required to keep the interface from moving landwards in response to pumping. Thus, if the analysis shows that the critical condition expressed by (3.8) is not fulfilled, artificial recharge may be used to create a head $h_f > B/\delta$ between the extraction well and the initial position of the toe.

The methodology presented by Strack (1976) is readily extended to include additional extraction wells and recharge wells. Furthermore, natural recharge may also be taken into account. Hence, it constitutes a flexible and potentially useful tool for hydrogeologic analysis, within the limitations posed by the assumptions of a sharp interface, steady, horizontal flow and a homogeneous aquifer.

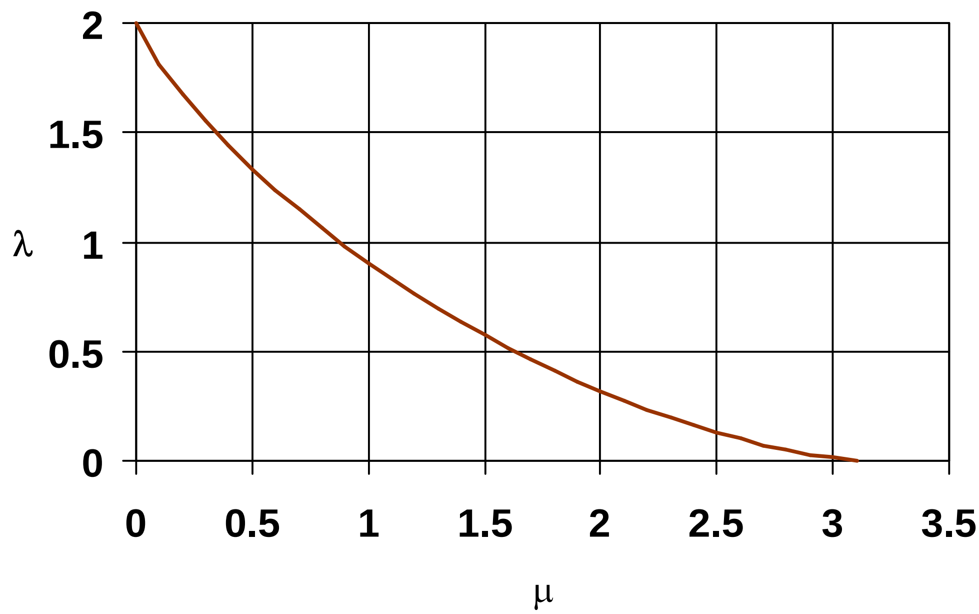

Figure B.8: Relation between dimensionless parameters representing the critical situation (after Strack, 1976); λ and μ are defined in connection with Equations (3.6) and (3.8), respectively

3.4. Steady interface in a heterogeneous aquifer

In the vast majority of modelling studies dealing with flow in coastal aquifers, the aquifer is either regarded as homogeneous or as consisting of a few distinct units of well-defined hydraulic properties. However, field evidence shows that the hydraulic conductivity, *K*, within an aquifer as a rule varies by orders of magnitude in an irregular manner. The common approach to account for heterogeneity is therefore to regard *K* as random and characterize it statistically; hence, variables that depend on *K* are also random (Dagan, 1989). Although the importance of variability in *K* for solute transport in well documented, few attempts have been made to model the effects of random heterogeneity on seawater intrusion. However, Dagan and Zeitoun (1998), which provides the basis for the following discussion, reported one such study in a recent paper.

Dagan and Zeitoun (1998) considered seawater intrusion in a confined aquifer where K varies in the vertical direction only, *i.e.* the structure of heterogeneity in assumed to follow that of a perfectly stratified formation. We consider a special case of the more general analysis presented in their paper, such that assumptions 1 through 4 in Section 6.1 are applicable. The hydraulic conductivity is regarded as a stationary random space function of the vertical coordinate, written as $K(z) = K_A[1+\kappa(z)]$, where K_A is the arithmetic mean of K, and κ is its normalized fluctuation. The fluctuation is characterized by a zero mean and its covariance function; the variance and integral scale of κ are denoted as σ^2 and I, respectively. The integral scale is a measure of the vertical distance over which the local values of K are correlated.

Since K varies in the vertical direction, so does the groundwater velocity. The local specific discharge 'Darcy velocity' can be written as $u(z) = -K(z)d\phi / dx$, where ϕ is the piezometric head, which is constant in the vertical direction. Thus, a water balance equation, equivalent to (3.1) for the homogeneous aquifer is obtained by integration as:

$$Q_0 = \int_0^{\eta} K(\eta')d\eta' \frac{d\phi}{dx} = K_A\left[\eta + \int_0^{\eta}\kappa(\eta')d\eta'\right]\frac{d\phi}{dx} \tag{3.9}$$

where Q_0 is the constant discharge, and $\eta = \eta(x)$ is a function that describes the position of the interface, with η defined as zero at the top of the aquifer and increasing downwards.

To transform (3.9) into an equation that can be solved for the interface position, use the Ghyben-Herzberg relation to obtain $d\phi / dx = (1/\delta)d\eta / dx$. Furthermore, since heterogeneity is in the vertical direction, it is more convenient to use the dependent variable $\chi = \chi(\zeta)$, instead of $\eta = \eta(x)$, where χ is the position of the interface in the x direction and ζ is an independent variable defined similar to η. The relation between the relevant derivatives is given by $(d\eta/dx)(d\chi/d\zeta) = 1$.

Changes of variables in (3.9) as described above and solution of the resulting equation with the boundary condition $\chi(0) = 0$ yields the following solution for χ:

$$\chi(\zeta) = \frac{K_A}{Q_0\delta}\left[\frac{\zeta^2}{2} + \int_0^{\zeta}\int_0^{\zeta'}\kappa(\zeta'')d\zeta''d\zeta'\right] \tag{3.10}$$

Since κ is a random function, (3.10) cannot be evaluated in a deterministic manner. Therefore, χ is characterized statistically, *i.e.* by its statistical moments. Specifically, we focus on the statistical description of the toe of the interface, using the notation $\chi(B) = L$, with $\langle L\rangle$ and σ_L^2 being the mean and the variance of L, respectively.

Assuming an exponential autocorrelation for κ, the first two statistical moments are obtained as:

$$\langle L \rangle = \frac{1}{2}\frac{K_A B^2}{Q_0 \delta}; \qquad \sigma_L^2 = \sigma^2 \left(\frac{I}{B}\right)^4 \left(\frac{K_A B^2}{Q_0 \delta}\right)^2 f(b) \qquad (3.11)$$

where $b = B/I$, and the function f is defined as:

$$f(b) = \frac{2b^3}{3} - b^2 - 2\exp(-b)(b+1) + 2$$

In equation (3.11) the expression for $\langle L \rangle$ is similar to the one for the interface toe in a homogeneous confined aquifer, see Section 6.2, provided K in the homogeneous aquifer is taken equal to the arithmetic mean of K in the heterogeneous one. Furthermore, the variance of L depends on both the magnitude (σ^2) and spatial scale (I) of heterogeneity.

To illustrate the sensitivity of $\langle L \rangle$ and σ_L^2 to the heterogeneity parameters, the discharge $Q_0 = K_A B i$, where i is the hydraulic gradient above the interface toe, and use the parameter values i = 0.01, B = 50 m and δ = 48. The conductivity is assumed to follow a lognormal distribution, which implies that $\sigma^2 = \exp(\sigma_Y^2) - 1$, where σ_Y^2 is the variance of $Y = \ln K$. The standard deviation of L is shown in Table B. 3.2 for a few combinations of (realistic) heterogeneity parameters. For comparative purposes, also the corresponding mean value has been included. Note, that both $\langle L \rangle$ and σ_L^2 are inversely proportional to i; therefore, we only considered one value of i.

Table B. 3.2. Statistical moments of *L* for different values of the heterogeneity parameters

	Standard deviation, σ_L (m)			Mean, $\langle L \rangle$ (m)
I (m)	$\sigma_Y^2 = 0.1$	$\sigma_Y^2 = 0.5$	$\sigma_Y^2 = 1.0$	
0.1	1.2	3.1	5.0	52.1
1.0	3.8	9.5	15.5	52.1

Table B.3.2 shows that the uncertainty about the position of the toe, expressed, for example, as a confidence interval $\pm 2\sigma_L$, may be large compared to the mean. The uncertainty increases with the variance of Y and with I. For example, an increase in σ_Y^2 from 0.1 to 1 leads to an increase in the standard deviation by a factor of four, whereas increasing I from 0.1 m to 1 m triples the uncertainty. The reason why the uncertainty increases with I is that a decreasing fraction of the total distribution of K-values (a decreasing number of layers), which underlies the ensemble statistics, is contained within the finite-size domain.

As shown by Dagan and Zeitoun (1998), the solution (3.10) may also be rearranged to enable an analysis of the statistics of the water discharge for a given (known) position of the interface (shift χ and Q_0 in (3.10)). Hence, in analogy with Section 3.3, the critical situation for seawater intrusion may be evaluated statistically in the simple case of one-dimensional flow. Further extension of the stochastic-analytical analysis to cases such as that considered by Strack (1976) appears to be possible, but these possibilities have not yet been investigated at this stage.

3.5. Conclusions regarding analytical solutions

Our current knowledge is, that there is no analytical solution that allows the mixing of seawater and freshwater. Thus, a numerical model is required if the transition zone cannot be approximated as a sharp interface. The problem of a moving interface has been studied within an analytical framework (Bear, 1979). However, it may be concluded that most time-dependent problems require a numerical solution of the flow equations.

Since the available analytical solutions are based on the assumption that seawater and freshwater are immiscible, they are not useful for prediction of salt concentrations in extraction wells, which to some extent will be affected by mixing. However, available analytical solutions permit hydraulic analysis of the aquifer, where extraction and recharge wells may be taken into account, and uncertainty analysis of simple cases. These solutions may be useful in an optimization of the aquifer system, provided that it can be shown (by numerical modelling) that the underlying assumptions are reasonable.

4. NUMERICAL MODELS

Numerical models are based on the basic equations described above with extensions to vertical flow components and flows in the saline water zone. However, all the models use some simplifications and aggregations.

In numerical models, the flow domain is divided into a large number of discrete cells, these cells may be constant. In some models they may vary in time. The presentation of singular points like pumping wells is by some geometric polygonal shape.

Most of the models are two dimensional in the vertical cross section. Only a small number are three dimensioned.

Most of the models assume a sharp interface; however, some address a dispersed interface.

Some of the models address only the flow in the fresh water domain and calculate the variations of the interface location without considering the flows in the saltwater domain.

All the models that include extraction terms either explicitly or implicitly relate to the extraction of fresh water above the interface. However, the process of down-coning of the interface to seawater Beach wells are similar to the process of its up-coning to fresh water collectors and wells. Such types of models are sought for the present study and the application of two of them is summarized in the following by examples of their outputs.

4.1. Application of the SUTRA model

A detailed description of the SUTRA model and its application in the present study is given in the main report (Chapter 6.1).

4.2. Application of a SHARP model

Mercado and Kally (1996) have applied the SHARP model to simulate the effects of seawater Beach wells on the location and shape of the interface. This is a quasi three dimensioned finite difference scheme developed by Essaid, USGS which simulates the flow of fresh and saline water in a multi-layered aquifer with a sharp interface.

Typical results of this study are shown in Figure 9. The maximum pumping in $m^3.day^{-1}.km^{-1}$ is shown for different sizes of aquifer depth (b) and the distance of the Beach wells from the sea. These relationships are shown for prevailing hydrogeologic characteristics of Hydraulic Conductivity (K), Direct Natural Replenishment (R), Interface Location (L) and density of seawater.

Discharge capacity $m^3.day^{-1}.km^{-1}$ and distance of Beach well from coastline (m).

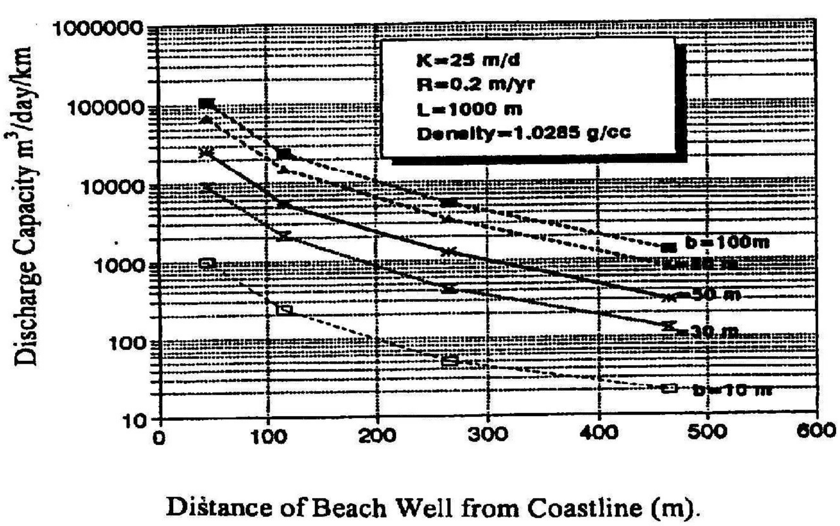

Figure B.9: Influence of well location of aquifer's depth on Beach well discharge capacity (Mercado and Kally, 1996)

The description of the SHARP Model in the Website of the USGS is as follows:

Name

Sharp - A quasi-three-dimensional, numerical finite-difference model to simulate freshwater and saltwater flow separated by a sharp interface in layered coastal aquifer systems.

Abstract

When the width of the freshwater-saltwater transition zone is small relative to the thickness of the aquifer, it can be assumed that freshwater and saltwater are separated by a sharp interface. The sharp interface modelling approach; in conjunction with vertical integration of the aquifer flow equations, facilitates regional scale studies of coastal areas. This approach does not give information concerning the nature of the transition zone but does reproduce the regional flow dynamics of the system and the response of the interface to applied stresses. SHARP is a quasi-three-dimensional, numerical model that solves finite-difference approximations of the equations for coupled freshwater and saltwater flow separated by a sharp interface in layered coastal aquifer systems. The model is quasi-three dimensional because each aquifer is represented by a layer in which flow is assumed to be horizontal.

Method

An implicit finite-difference discretization scheme that is central in space and backward in time is used to solve the freshwater and saltwater flow equations for each model layer. Spatial discretization is achieved using a block-centred finite-difference grid that allows for variable grid spacing. In the central difference approximations for the space derivatives, the thickness at the grid block boundaries is linearly interpolated and the conductivity terms are estimated using the harmonic mean of nodal values. At blocks containing pumped wells, the amount of freshwater and saltwater extracted depends on the position of the interface relative to the elevation of the screened interval of the well. The rate of freshwater and (or) saltwater extraction from a block, relative to the total fluid extraction rate, is determined linearly on the basis of the proportion of screen penetrating the freshwater and saltwater zones relative to the total open interval of the well. The interface elevation in each finite-difference block is calculated using the numerically determined freshwater and saltwater head distributions. The shape of the interface can be obtained by connecting the discretized interface elevations. The position of the interface tip (the intersection of the interface with the top of the aquifer) and the interface toe (the intersection of the interface with the bottom of the aquifer) are located by linearly projecting a line defined by the interface elevations in adjacent blocks until it intersects the top and bottom of the aquifer.

Data Requirements

SHARP requires all of the input parameters typically required by a finite-difference ground-water flow model (initial conditions, boundary conditions, aquifer properties). However, because it solves both freshwater and saltwater flow equations, it has additional input requirements. The fresh and saltwater specific gravities and dynamic viscosities must be specified. Freshwater hydraulic conductivities are specified and saltwater hydraulic conductivities are calculated in the model. Fresh and saltwater specific storages, effective porosity, and confining layer leakage values must be specified. For interface tip and toe tracking, SHARP requires elevations of the base of each layer and the thickness of the layer. Offshore bathymetric elevations are required to represent offshore boundary conditions. See documentation for details.

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APPENDIX - C

A MODEL FOR COST ESTIMATIONS OF DESALINATION AND INTAKE

Contents

1. Introduction

A model for cost estimates is required for the present study to enable economic comparison among the various Intake alternatives and for the same alternative to compare different design layouts and dimensions. The model has to include as input; all the variables in engineering, desalination technology, geohydrology and finance. The output has to include comparison criteria such as: investments, recurring costs, indirect costs and benefits. Spreadsheets for cost estimates were prepared. These cost estimates are intended to reflect life cycle costs and include both capital and operation costs. The workbook developed on MS-EXCEL includes the following sheets:

Input sheet: Data on: capacity, plant components, Intake components, hydrogeological characteristics, general financial parameters and unit costs.

Output sheet: This sheet is now attached as a lower part to the input sheet. It includes extracts from the model spreadsheet showing:

Total Investments; Total Annual Costs including: Capital, O&M, Energy, and Chemicals. Breakdown of annual and capital costs to the following components: Land acquisition & site development, Intake, Pretreatment, Desalination, Post treatment and Brine disposal. Costs can be quoted and reported in foreign currency (F.C) and/or local currency (L.C). The user can copy the input - output spreadsheet as ‘Values’ to another spreadsheet for further reference in the future.

Default sheet: Values entered that may be used as input but are overridden by the input data sheet.

Data sheet: The selection of values from the Input and Default spreadsheets. No data are entered to this sheet.

Model sheet: The main components of the model are calculations of the different cost components. Dependent input values are also computed. These include: length of gallery, or number of Beach wells; based on total discharge and specific infiltration capacity, Desalination recovery. This sheet includes also an executive summary of inputs and results, which is also copied to the lower part of the input spreadsheet. No data are entered on this sheet.

Figures sheet: Shows the main cost components as a pie chart.

Delay sheet: Contains series of annual values as percentage of water taken from inland users and computation of their present value (based on L.C. Interest Rate). Data are not entered on this sheet.

The following sections describe all the components of the model.

2. Inputs

All the inputs are included in a single spreadsheet. The data in this sheet are entered by the user except those that had already been entered to the ‘default’ sheet and have not to be repeated. Seven groups of input data are included (see Input - Output Spreadsheet in Appendix D):

a) Titles
b) Desalination plant
c) Hydrogeology
d) Finance
e) Unit cost of chemicals
f) Unit cost of energy
g) Desalination plant and Intake – components and inputs

The next sections detail these inputs:

2.1. Titles

a) Plant name
b) Type of plant
c) Type of Intake

2.2. Desalination plant

a) Plant capacity ($m^3.day^{-1}$) – maximum daily water production
b) Annual production ($m^3.yr^{-1}$) – annual water production (plant capacity multiplied by 365 and by availability)
c) Feed water salinity ($mg.l^{-1}$), total dissolved solids
d) Recovery (%) – ratio between product water quantity and feed water quantity
e) Total site area for all plant components (m^2)
f) Distance from Intake (m) - length of the feed pipeline, not including the marine Intake pipeline or the collector pipeline
g) Distance to brine disposal (m) – length of the disposal pipeline

2.3. Hydrogeology

a) GW Flow Rate ($m^3.year^{-1}.m$) - the quantity of groundwater that will flow under operational conditions to 1 m length of a beach gallery, or to 1 m length of a long battery of Beach wells, or to 1m length of a sea bottom infiltration gallery
b) Fresh water fraction pumped (%) - the fraction that will flow to the subsurface Intakes from the inland direction. This component may decrease the salinity of feed water and decrease desalination energy costs
c) Fraction Penalized (%) – part of the pumped water may be indirectly taken from inland potential users. The treatment plant has to be penalized for the water eliminated from other groundwater users. In the present study it is assumed that these costs will be accounted for by assuming that part of the desalinated water is reduced from the total volume produced to compensate these users. The reduction of the denominator will increase the unit cost of desalinated water. These costs are regarded as 'indirect costs'. This value is compiled in section G as a weighted average over time. Only if the number of years given; is zero, then the value is taken from here or from the parallel default sheet

d) Single well capacity ($m^3.day^{-1}$) - for estimating the number of wells required
e) Depth of well (m) - for estimating costs of drilling and well casing
f) Depth to dynamic water level (m) For estimating energy requirements
g) Length of well screens (m) used in the SUTRA model

2.4. Finance

a) Symbol of local currency (L.C) or another desired currency
b) Symbol of foreign currency (F.C) as desired
c) Exchange rate – the quantity of local currency equivalent to one unit of foreign currency
d) Date of cost estimate – to enable Updates in the future
e) Interest rate (%) for local and for foreign currencies
f) Lifetime and Lead (Construction) time for financial discounting are given for each component separately (Section 2.7 G)

2.5. Chemicals

Up to 7 types of chemicals can be selected. The cost of 1 kg has to be given in L.C. or in F.C or a combination.

2.6. Energy

Up to 14 Electricity / Fuel tariffs can be selected. For each one, the price of energy in three time spans (*e.g.* off peak, non peak, peak) may be given (in L.C, or F.C or combination) and the percentage in time of each time span.

2.7. Fraction penalized

When impact on inland pumping is delayed, then it can be given as a time series of up to 100 annual values. The number of years is to be given in the top. If this number is not larger then zero, then this value is taken from section C Hydrogeology or its default value.

2.8. Desalination plant components and inputs

These data are given in a matrix form (Section H of the Input-Output sheet). The rows are the main components of the desalination process. The columns are the inputs required for construction and operation. The main components of the plant are:
a) Land and site
b) Intake – including wells or galleries, collectors and low level pumping
c) Pretreatment system
d) Desalination plant (including high-level pressure pumping and energy recovery turbine)
e) Post treatment
f) Brine disposal

Each of these components is divided into sub components and contingencies. Costs of sub-components may be lumped for each component if no information is available on the cost breakdown. The production inputs are given in the data input table as unit values and unit costs. All unit costs can be given either in local currency values, or in foreign currency or a combination of both.

a) Investments may be composed of two parts: unit cost (per m, per $m^3.day^{-1}$ *etc.*) and a fixed cost per plant unit or a single Beach well
b) Time schedules for computing capital costs. Construction time (lead-time) for computing interest during construction (assuming evenly distributed investments during that time) and depreciation time (lifetime of component) for computing annual capital costs
c) Operation and maintenance – composed of two components: fraction of investment and a fixed annual cost for the component (for wells; this is for the whole well system and not for a single well)
d) Chemicals: various chemicals can be given for each main component group in accordance with the list of chemicals given in 2.5. Quantity (in $g.m^{-3}$ or ppm) is given for the water quantity in that main component (different for feed water and for product water)
e) Energy: for each main component group, various tariff types can be given in accordance with the list given in 2.5. For each component the quantity ($kWh.m^{-3}$) is given

Note: the cost of membranes is included in investments and capital costs of the desalination plant, however, with a short depreciation time. This approach seems to be more appropriate for the present study than including them in operational costs

3. Outputs

The outputs are extracted and summarized in the lower part of the input sheet. This summary table includes:

a) Investments and Interest during construction for each of the main components
b) Annual cost for all plant components and inputs
c) Unit cost per m^3 product water in L.C and F.C for each one of the components and for indirect costs
d) Breakdown of the annual costs into: Capital, Energy, O&M and Chemicals
e) Indirect cost and total cost including both direct and indirect

4. Default

This is a sheet similar in shape to the Input sheet. Default values can be given which have not been repeated later in the Input sheet when studying various alternatives.

5. Data

This is the input sheet used by the model. It selects values as relevant from the Input or from the default sheet. Input values override default values unless the input value remains blank.

6. The Model

This sheet includes all computations. A summary is extracted in the bottom part. This extract is also shown in the bottom part of the Input sheet. The structure of this sheet is similar to the previous sheets with minor modifications. The following computations are included:

6.1. Physical values

No. of wells = (Capacity of plant) / (Single well capacity) / Recovery

Length of collectors = (Capacity of plant) / (GW Flow Rate) / Recovery
(if not given as input)

Load Factor = (Annual Production) / (Plant capacity * 365)

6.2. Financial values

Monthly Interest Rate = (Interest Rate + 1) ^ (-1/12)-1

6.3. Unit costs of energy

Weighted Mean Price = Σ(Unit cost * Fraction of time)

6.4. Desalination and Intake costs

Investment total cost = Fixed cost + (Quantity) * (Unit cost)

Interest during construction (separately for local and foreign Interest rates)
= FV (Interest, construction time, (Total investment/construction time))

Annual capital costs (separately for local and foreign Interest rates)
= PMT (Interest, Depreciation time, (Investment + Interest during construction))

Annual O&M costs = (Fraction) * (Investment) + Annual costs

Chemicals = (Cost of chemical) * (Unit consumption) * (Annual water quantity)

Energy = (Cost of energy) * (Unit consumption) * (Annual water quantity)

Unit direct costs = (Total Direct Annual costs) / (Annual production)

Unit indirect costs = (Unit direct cost) * (Fraction penalized)

7. Extensions

Some additional components have been introduced:

7.1. Transient hydrological impacts

The computation procedure assumes that the hydrological impact is immediate and constant over the years. In reality, groundwater processes have a very long time response factor. Impacts are transient and delayed.

For dealing with this discrepancy, an additional sheet (DELAY) has been added and the following procedure is applied in the model.

a) The data on the fractions of fresh water pumped and penalized are given as a time series of annual values for a long time span (50 - 100 years)
b) The Present Value (PV) of this series is computed with the given Interest Rate
c) An economically equivalent fixed annual value (PMT) is computed and used in the model

This procedure is included in the DELAY sheet. The input is a time series of annual values as percentage of total plant capacity. The output is the equivalent Fraction penalized to be input in 2.3.C (c).

7.2. Indirect benefits

Pumping of seawater may limit its inland intrusion and enable increased pumping inland. This increase may be regarded as the opposite of indirect costs. The fraction penalized in such a situation is given Negative values. Total cost in this case will be smaller than direct costs. These data may be incorporated as negative values in the Delay Time Series (Section 7.1 above).

7.3. Impact of feed water quality on unit costs

This is a function that the user applies outside the model, and unit costs are given by the user accordingly.

8. Example

The sheets: Input-Output, Model, and Charts are an example of application of the model to Beach wells. These are represented in the following Appendix D, for two cases: Sea Intake (Test 1) and Beach wells (Test 2).

APPENDIX - D

EXAMPLES OF THE COST ESTIMATES MODEL APPLICATION

Contents

Cost estimate of non-surface Intakes and seawater desalination plants – input–output

A. Plant Name: TEST 1 Type of Plant: SWRO Type of Intake: Sea Intake

B: Desalination plant		
Plant capacity	$m^3.day^{-1}$	30,000
Annual production	$m^3.y^{-1}$	10,000,000
Feed water salinity	$mg.l^{-1}$	39,660
Recovery	%	50.0%
Site area	m^2	13,000
Distance from Intake/sea	m	200
Distance to brine disposal	m	650

C. Hydrogeology		
GW flow rate	$m^3.y^{-1}.m^{-1}$	100,000,000,000
Fresh water fraction pumped	%	0.0
Fraction penalised*	%	0.0
Single well capacity	$m^3.day^{-1}$	2,000,000,000
Depth of well	m	0
Depth to dynamic water Intake	m	0
Length of well screen	m	0
Length of gallery/sea Intake	m	675

*Fraction of water pumped on account of inland supply. Data given here are over-ridden by Table G

TEST 1 (continued)

D. Finance	
Legal currency	IS
Foreign currency	US$
Exchange rate	4.00
Date of cost estimates	1/1/00
Interest rate – local	6.00%
Interest rate – foreign	5.00%

E. Unit costs of chemicals		
Type	Cost of 1 kg	
	LC	FC
	IS	US$
$FeCl_3$		1.50
H_2SO_4		0.75
Cl		1.00
NaOCl		0.80
$NaHSO_3$		1.10
$CaCO_3$		0.50
NaOH		0.70

F. Unit costs of energy									
Taanf Type	Unit costs by time rates						Fraction of $h.y^{-1}$		
	LC IS per kWh			FC US$ per kWh			%		
	Off peak	Non peak	Peak	Off peak	Non peak	Peak	Off peak	Non peak	Peak
A	0.15	0.20	0.30				40.0	30.0	30.0
B	0.20	0.25	0.40				40.0	30.0	30.0
C	0.25	0.30	0.50				40.0	30.0	30.0

TEST 1 (continued)

G. Fraction penalised for using fresh water*

No. of years 100

Year	Fraction	Year	Fraction
1		51	
2		52	
3		53	
4		54	
5		55	
6		56	
7		57	
8		58	
9		59	
10		60	
11		61	
12		62	
13		63	
14		64	
15		65	
16		66	
17		67	
18		68	
19		69	
20		70	
21		71	
22		72	
23		73	
24		74	
25		75	
26		76	
27		77	
28		78	
29		79	
30		80	
31		81	
32		82	
33		83	
34		84	
35		85	
36		86	
37		87	
38		88	
39		89	
40		90	
41		91	
42		92	
43		93	
44		94	
45		95	
46		96	
47		97	
48		98	
49		99	
50		100	

* Data here over-ride data given in Table C

TEST 1 (continued)

H. Desalination plant and Intake – components and inputs

Ser: No.	Item	Unit	Quan -tity	Investments				Schedules		Operation & maintenance costs			Chemicals		Energy	
				LC		FC		Const:	Deprec:	Fracti -on of invest -ment	Additional annual cost					
				Unit cost	Fixed cost	Unit cost	Fixed cost				LC	FC	Type	Consu- mption	Type	Consu- mption
				IS	IS	US$	US$	Months	Years	(%)	IS	US$	(–)	($g.m^{-3}$)	(–)	($kWh.m^{-3}$)
10	Land and site															
11	Land acquisition / rental	m^2	1	300.00				12	1,000							
12	Site development / infrastructure	m^2	1	100.00				12	1,000	1.0						
13	Contingency	%	20.0					12	1,000							
20	Intake															
21	Well drilling	m	1	800.00				12	35							
22	Pump columns	m	1			700.00		12	20							
23	Single well pump and electricity	–	1			6,000		12	15	3.0						
24	Single well infrastructure	–	1	15,000.00				12	50	1.0						
25	Collection / gallery	m	1			6,450		18	30	0.5						

TEST 1 (continued)

26	Connecting pipeline	m	1			260.00		12	30	0.5						
27	Low level pumping	$m^3.d^{-1}$	1					6	20	3.0					A	0.35
28	Contingency	%	20.0					12	30							
30	Pre-treatment plant															
31																
32	Structures	$m^3.d^{-1}$	1			10.00		12	50	1.0			$FeCl_3$	3.00		
33	Electro-mechanical	$m^3.d^{-1}$	1			45.00		12	30	3.0			H_2SO_4	45.00	A	0.11
34	Media/Filters	$m^3.d^{-1}$	1			45.00		12	10	3.0			Cl	4.00		
35	Chemicals	$m^3.y^{-1}$											NaOCl	2.00		
36	Contingency	%	10.0					12	30							
60	Desalination plant															
61	High pressure pumps + turbine	$m^3.d^{-1}$	1			105.00		24	50	3.0					A	3.00
62	Structures	$m^3.d^{-1}$	1			30.00		24	50	1.0						
63	Electro-mechanical	$m^3.d^{-1}$	1			149.00		24	30	3.0						
64	Membranes	$m^3.d^{-1}$	1			60.00		6	5	3.0						
65	Other	$m^3.d^{-1}$														

TEST 1 (continued)

66	Contingency	%	10.0					24	40							
70	Post treatment															
								18	50							
72	Structures	$m^3.d^{-1}$				70.00		18	50	1.0			$NaHSO_3$			
73	Electro-mechanical	$m^3.d^{-1}$				63.00		18	30	3.0			$CaCO_3$	30.00	A	0.58
74	Media/chemicals	$m^3.d^{-1}$						6	10	3.0			NaOH	15.00		
75	Other	$m^3.d^{-1}$						18	50							
76	Contingency	%	10.0					18	50							
80	Brine disposal															
81	Pipelines	m				3,300		24	30	1.0						
82	Structures	$m^3.d^{-1}$						24	50	1.0						
83	Electro-mechanical	$m^3.d^{-1}$						24	15	3.0						
84	Other	$m^3.d^{-1}$						24	30							
85	Contingency	%	20.0					24	30							

TEST 1 (continued)

Calculated costs										
Item	Investment costs				Annual costs					
	Investments		Interest during construction		Capital*		O&M		Chemicals	
	LC	FC	LC	FC	LC	FC	LC	FC	LC	FC
	IS	US$	IS	US$	IS	US$	IS	US$	IS	US$
Land and site	6,240,000	0	106,222	0	380,767	0	13,000	0	0	0
Intake	1	5,286,900	210,287	175,294	15,277	355,324	0	22,029	0	0
Pretreatment plant	0	6,600,000	179,591	149,918	17,541	610,763	0	168,000	0	877,000
Desalination plant*	0	14,817,000	777,384	646,929	55,779	1,220,558	0	386,100	0	0
Post treatment	0	8,778,000	372,785	310,690	25,129	538,062	0	155,400	0	510,000
Brine disposal	0	2,574,000	149,362	124,280	10,851	175,527	0	21,450	0	0
Total	6,240,001	38,055,900	1,795,531	1,407,109	505,345	2,900,233	13,000	752,979	0	1,387,000
Total single currency × 1000	158,464	39,616	7,424	1,856	12,106	3,027	3,025	756	5,548	1,387
Percentage:					38.6%		9.6%		17.7%	

*Including membranes

TEST 1 (continued)

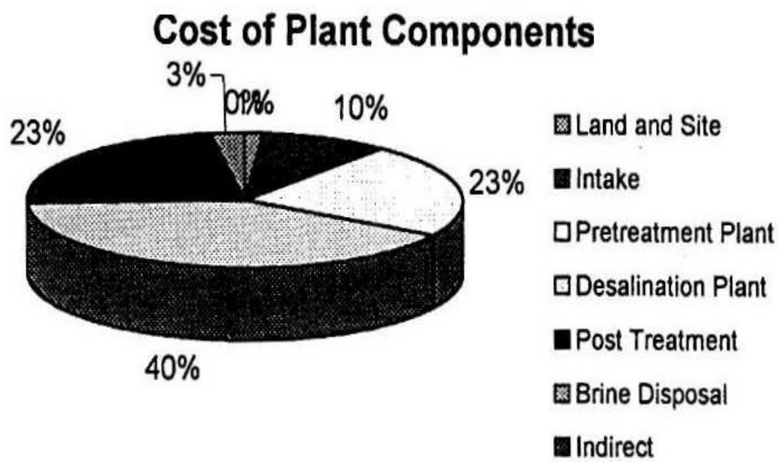
	Energy		Total		Unit costs (for one m^3) by currency		Percentage (%)	Total unit costs (for one m^3)	
	LC	FC	LC	FC	LC	FC			
	IS	US$	IS	US$	IS	US$			
Land and site	0	0	393,767	0	0.039	0.000	1.3	0.039	0.010
Intake	1,470,000	0	1,485,277	377,352	0.149	0.038	9.6	0.299	0.075
Pretreatment plant	462,000	0	479,541	1,655,763	0.048	0.166	22.7	0.710	0.178
Desalination plant*	6,300,000	0	6,355,779	1,606,658	0.636	0.161	40.8	1.276	0.320
Post treatment	2,436,000	0	2,461,129	1,203,462	0.246	0.120	23.2	0.727	0.182
Brine disposal	0	0	10,851	196,977	0.001	0.020	2.5	0.080	0.020
Total	10,668,000	0	11,186,345	5,040,212	1.119	0.504	100.0	3.135	0.784
Total single currency × 1000	10,668	2,667	31,347	7,837					
Percentage:	34.0%		100%						

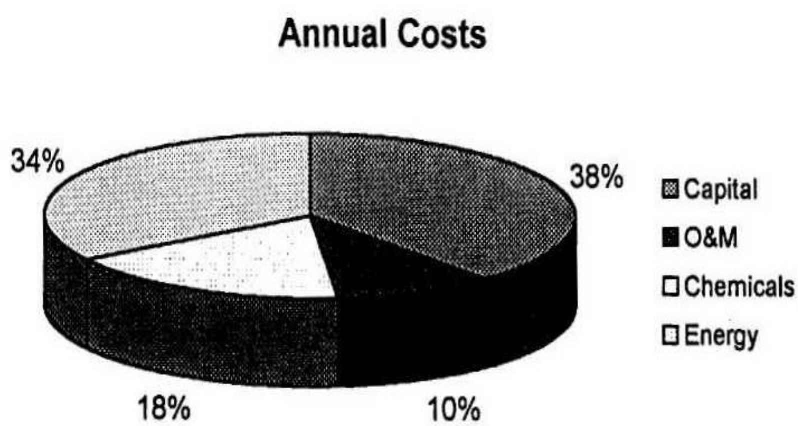
*Including membranes

Indirect cost	0.000	0.000	0.0	0.000	0.000
Cost including indirect:	1.119	0.504	100.0	3.135	0.784

No. of wells:	0
Length of gallery/collection (m)	675
Load factor:	0.91
Equivalent fraction penalised	0.00%

TEST1





Cost estimation of desalination and Intakes

Cost of desalination and Intakes – model

Type of Plant: SWRO **A.** Plant name: Test1		Type of Intake: Sea Intake
B: Desalination plant		
Plant capacity	$m^3.day^{-1}$	30,000
Annual production	$m^3.y^{-1}$	10,000,000
Feed water salinity	$mg.l^{-1}$	39,660
Recovery	%	1
Salinity rejection	%	0.0
Site area	m^2	13,000
Distance from Intake	m	200
Distance to brine disposal	m	650

C. Hydrogeology		
GW flow rate	$m^3.y^{-1}.m$	-----
Fresh water fraction pumped	%	0.0
Fraction penalised*	%	0.0
Single well capacity	$m^3.d^{-1}$	
Depth of well	m	0
Depth to dynamic water table	m	0
Length of well screen	m	0
Length of collector / Gallery	m	675
*Fraction of water pumped on account of Inland Supply.		
No of wells	0.00	
Collection length, m	675	
Load factor	0.913	

Test1 (continued)

D. Finance			
Legal currency	IS		
Foreign currency	US$		
Exchange rate	4.00		
Date of cost estimates	1/1/00		
Intert. rate – local	6%	Interest rate – foreign	5%
Monthly interest – local	0.49%	Monthly interest – foreign	0.41%

F. Unit costs of energy

Taanf Type	Weighted mean rate		Unit costs by time rates						Fraction of hours per year			
			LC IS per WH			FC US$ per KWh			%			
	LC	FC	Low	Regular	Peak	Low	Regular	Peak	Low %	Regular %	Peak %	10 %
A	0.21	0.0	0.15	0.20	0.30	0.00	0.00	0.00	40.0	30.0	30.0	10 %
B	0.22	0.00	0.20	0.25	0.40	0.00	0.00	0.00	40.0	6.0	30.0	
C	0.27	0.00	0.25	0.30	0.50	0.00	0.00	0.00	40.0	7.0	30.0	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		

E. Unit costs of chemicals

Type	Cost of 1 kg	
	LC	FC
	IS	US$
$FeCl_3$	0	2
H_2SO_4	0	1
Cl	0	1
NaOCl	0	1
$NaHSO_3$	0	1
$CaCO_3$	0	1
NaOH	0	1

Test1 (continued)

Ser No.	Item	Unit	Quantity	Investments				Schedule	
				LC		FC		Construction	Depreciation
				Unit cost IS	Total cost IS	Unit cost US$	Total cost US$	(Months)	(Years)
10	Land and site								
11	Land acquisition / rental	m^2	13,000	300.00	3,900,000	0.00	0	12	1,000
12	Site development	m^2	13,000	100.00	1,300,000	0.00	0	12	1,000
13	Contingency	%	20.0		1,040,000		0	12	1,000
	Sub total				6,240,000		0		
20	Intake	$m^3.y^{-1}$	20,000,000						
21	Well drilling	m	0	800.00	0	0.00	0	12	35
22	Pump columns	m	0	0.00	0	700.00	0	12	20
23	Wells pumps and electricity	–	0	0.00	0	6,000.00	0	12	15
24	Wells infrastructure	–	0	15,000.00	0	0.00	0	12	50
25	Collector/gallery	m	675	0.00	0	6,45.00	4,353,750	18	30
26	Connecting pipeline	m	200	0.00	0	260.00	52,000	12	30
27	Low level pumping	$m^3.d^{-1}$	60,000	0.00	0	0.00	0	6	20
28	Contingency	%	20.0		0		881,150	12	30
	Sub total				1		5,286,900		
30	Pretreatment plant	$m^3.y^{-1}$	20,000,000						
31		$m^3.d^{-1}$	60,000						
32	Structures	$m^3.d^{-1}$	60,000	0.00	0	10.00	600,000	12	50
33	Electro-mechanical	$m^3.d^{-1}$	60,000	0.00	0	45.00	2,700,000	12	30
34	Media	$m^3.d^{-1}$	60,000	0.00	0	45.00	2,700,000	12	10
35	Chemicals	$m^3.y^{-1}$	20,000,000	0.00	0	0.00	0	0	0
36	Contingency	%	10.0		0		600,000	12	30

Test1 (continued)

	Sub total				0		6,600,000		
60	Desalination plant	$m^3\ y^{-1}$	10,000,000						
61	High pressure pumps + turbine	$m^3\ d^{-1}$	60,000	0.00	0	105.00	6,300,000	24	50
62	Structures	$m^3\ d^{-1}$	30,000	0.00	0	30.00	900,000	24	50
63	Electro-mechanical	$m^3\ d^{-1}$	30,000	0.00	0	149.00	4,470,000	24	30
64	Membranes	$m^3\ d^{-1}$	30,000	0.00	0	60.00	1,800,000	6	5
65	Other	$m^3\ d^{-1}$	30,000	0.00	0	0.00			
66	Contingency	%	10.0		0		1,347,000	24	40
	Sub total				0		14,817,000	0	0
70	Post treatment	$m^3\ y^{-1}$	10,000,000						
71	Infrastructure	$m^3\ d^{-1}$	60,000	0.00	0	0.00	0	18	50
72	Structures	$m^3\ d^{-1}$	60,000	0.00	0	70.00	4,200,000	18	50
73	Electro-mechanical	$m^3\ d^{-1}$	60,000	0.00	0	63.00	3,780,000	18	30
74	Media/chemicals	m3 d^{-1}	60,000	0.00	0	0.00	0	6	10
75	Other	m3 d^{-1}	60,000	0.00	0	0.00	0	18	50
76	Contingency	%	10.0		0		798,000	18	50
	Sub total				0		8,778,000	0	0
80	Brine disposal	m3 y^{-1}	10,000,000						
81	Pipelines	m	650	0.00	0	3,300.00	2,145,000	24	30
82	Structures	m3 d^{-1}	0	0.00	0	0.00	0	24	50
83	Electro-mechanical	m3 d^{-1}	0	0.00	0	0.00	0	24	15
84	Other	m3 d^{-1}	0	0.00	0	0.00	0	24	30
85	Contingency	%	20.0		0		429,000	24	30
	Sub total				0		2,574,000		
	Grand total				6,240,001		38,055,900		

Test1 (continued)

Ser No.	Interest during construction		Annual capital costs		Operation & maintenance costs								
	LC	FC	LC	FC	Fraction of investment	Total annual O&M costs		Chemicals		Energy		Annual direct costs	
						LC	FC	LC	FC	LC	FC	LC	FC
	IS	US$	IS	US$	(%)	IS	US$	IS	US$	IS	US$	IS	US$
10													
11	106,122	0	240,367	0	0.0	0	0					240,367	0
12	0	0	78,000	0	1.0	13,000	0					91,000	0
13	0	0	62,400	0	0.0	0	0					62,400	0
	106,122	0	380,767	0		13,000	0					393,767	0
20													
21	0	0	0	0	0.0	0	0	0	0	0	0	0	0
22	0	0	0	0	0.0	0	0	0	0	0	0	0	0
23	0	0	0	0	3.0	0	0	0	0	0	0	0	0
24	0	0	0	0	1.0	0	0	0	0	0	0	0	0
25	184,896	154,097	13,432	293,242	0.5	0	21,769	0	0	0	0	13,432	315,011
26	1,415	1,181	103	3,460	0.5	0	260	0	0	0	0	103	3,728
27	0	0	0	0	3.0	0	0	0	0	1,470,000	0	1,470,000	0
28	23,977	20,015	1,742	58,622	0.0	0	0	0	0	0	0	1,742	58,622
	210,287	175,294	15,277	355,324		0	22,029	0	0	1,470,000	0	1,485,277	
30													
31													
32	16,326	13,629	1,036	33,613	1.0	0	6,000	0	90,000	0	0	1,036	
33	73,469	61,330	5,337	179,628	3.0	0	81,000	0	675,000	462,000	0	467,333	

Test1 (continued)

34	73,469	61,330	9,982	357,605	3.0	0	81,000	0	80,000	0	0	9,982
35					0.0	0	0	0	32,000	0	0	0
36	16,326	13,629	1,186	39,917	0.0	0	0	0	0	0	0	1,186
	179,591	149,919	17,541	610,763		0	168,000	0	877,000	462,000	0	479,541
60												
61	365,570	304,181	23,193	361,755	3.0	0	189,000	0	0	6,300,000	0	6,323,193
62	52,224	43,454	3,313	51,679	1.0	0	9,000	0	0	0	0	3,313
63	259,381	215,824	18,844	304,820	3.0	0	134,100	0	0	0	0	18,844
64	22,047	18,433	5,234	420,012	3.0	0	54,000	0	0	0	0	5,234
65					0.0	0	0	0	0	0	0	0
66	78,162	65,037	5,196	62,291	0.0	0	0	0	0	0	0	5,195
	777,384	646,929	55,779	1,220,558		0	386,100	0	0	6,300,000	0	6,335,779
70												
71	0	0	0	0	0.0	0	0	0	0	0	0	0
72	178,366	148,655	11,316	238,205	1.0	0	42,000	0	0	0	0	11,316
73	160,530	133,790	11,662	254,598	3.0	0	113,400	0	300,000	2,436,000	0	7,447,662
74	0	0	0	0	3.0	0	0	0	210,000	0	0	0
75	0	0	0	0	0.0	0	0	0	0	0	0	0
76	33,890	28,245	2,150	45,259	0.0	0	0	0	0	0	0	2,150
	372,785	310,490	26,129	538,062		0	155,400	0	510,000	2,436,000	0	2,461,129
80												
81	124,468	103,566	9,047	146,272	1.0	0	21,450	0	0	0	0	9,042
82	0	0	0	0	1.0	0	0	0	0	0	0	0
83	0	0	0	0	3.0	0	0	0	0	0	0	0
84	0	0	0	0	0.0	0	0	0	0	0	0	0
85	24,894	20,713	1,808	29,254	0.0	0	0	0	0	0	0	1,808
S- total	149,362	124,280	10,851	175,527		0	21,450	0	0	0	0	10,851
Grand total	1,795,531	1,407,109	505,345	2,900,733		13,000	752,979	0	1,387,000	10,668,000	0	11,186,345

Cost estimate of non surface Intakes and seawater: Desalination plants – input–output

A. Plant name: Test2			
B: Desalination plant			
Plant capacity	$m^3.day^{-1}$	30,000	
Annual production	$m^3.y^{-1}$	10,100,000	3.125
Feed water salinity	$mg.l^{-1}$	35,692	
Recovery	%	50.5	
Site area	m^2	10,400	
Distance from Intake/sea	m	200	
Distance to brine disposal	m	650	

C. Hydrogeology		
GW flow rate	$m^3.y^{-1}.m^{-1}$	1,100
Fresh water fraction pumped	%	10.0
Fraction penalised*	%	10.0
Single well capacity	$m^3.day^{-1}$	2,400
Depth of well	m	100
Depth to dynamic water table	m	50
Length of well screen	m	10
Length of gallery/sea Intake	m	
*Fraction of water pumped on account of Inland Supply. Data given here are over-ridden by Table G		

D. Finance	
Local currency	IS
Foreign currency	US$
Exchange rate	4.00
Date of cost estimates	1/1/00
Interest rate – local	6.00%
Interest rate – foreign	5.00%

E. Unit costs of chemicals		
Type	Cost of 1 kg	
	LC	FC
	IS	US$
$FeCl_3$		1.50
H_2SO_4		0.75
Cl		1.00
NaOCl		0.80
$NaHSO_3$		1.10
$CaCO_3$		0.50
NaOH		0.70

F. Unit costs of energy									
Taanf Type	Unit costs by time rates						Fraction of $h.y^{-1}$		
	LC IS per kWh			FC US$ per kWh			%		
	Off peak	Non peak	Peak	Off peak	Non peak	Peak	Off peak	Non peak	Peak
A	0.15	0.20	0.30				40.0	30.0	30.0
B	0.20	0.25	0.40				40.0	30.0	30.0
C	0.25	0.30	0.50				40.0	30.0	30.0

G. Fraction penalised for using fresh water*			
No. of years		100	
Year	Fraction (%)	Year	Fraction (%)
1	0	51	10
2	2	52	10
3	5	53	10
4	6	54	10
5	7	55	10
6	8	56	10
7	9	57	10
8	9	58	10
9	10	59	10
10	10	60	10
11	10	61	10
12	10	62	10
13	10	63	10
14	10	64	10
15	10	65	10
16	10	66	10
17	10	67	10
18	10	68	10
19	10	69	10
20	10	70	10
21	10	71	10
22	10	72	10
23	10	73	10
24	10	74	10
25	10	75	10
26	10	76	10
G. (continued)			
27	10	77	10
28	10	78	10
29	10	79	10
30	10	80	10
31	10	81	10
32	10	82	10
33	10	83	10
34	10	84	10
35	10	85	10
36	10	86	10
37	10	87	10
38	10	88	10
39	10	89	10
40	10	90	10
41	10	91	10
42	10	92	10
43	10	93	10
44	10	94	10
45	10	95	10
46	10	96	10
47	10	97	10
48	10	98	10
49	10	99	10
50	10	100	10

* Data here over-ride data given in Table C

H. Desalination plant and Intake – components and inputs									
Ser. No.	Item	Unit	Quantity	Investments				Schedules	
				LC		FC		Construction	Depreciation
				Unit cost	Fixed cost	Unit cost	Fixed cost		
				IS	IS	US$	US$	(Months)	(Years)
10	Land and site								
11	Land-acquisition/rental	m^2	1	300.00				12	1,000
12	Site development-infrastructure	m^2	1	100.00				12	1,000
13	Contingency	%	20.0					12	1,000
20	Intake								
21	Well drilling	m	1	800.00				12	35
22	Pump columns	m	1			700.00		12	20
23	Single well pump-and electricity	–	1			6,000.00		12	15
24	Single well-infrastructure	–	1	15,000.00				12	50
25	Collector/gallery	m	1			120.00		18	30
26	Connecting pipeline	m	1			260.00		12	30
27	Low level pumping	$m^3.d^{-1}$	1					6	20

(continued)

28	Contingency	%	20.0					12	30
30	Pretreatment plant								
31									
32	Structures	$m^3.d^{-1}$	1			9.00		12	50
33	Electro-mechanical	$m^3.d^{-1}$	1			41.50		12	30
34	Media/Filters	$m^3.d^{-1}$	1			41.50		12	10
35	Chemicals	$m^3.y^{-1}$							
36	Contingency	%	10.0					12	30
60	Desalination plant								
61	High pressure pumps + turbine	$m^3.d^{-1}$	1			105.00		24	50
62	Structures	$m^3.d^{-1}$	1			30.00		24	50
63	Electro-mechanical	$m^3.d^{-1}$	1			149.00		24	30
64	Membranes	$m^3.d^{-1}$	1			60.00		6	5
65	Other	$m^3.d^{-1}$							
66	Contingency	%	10.0					24	40
70	Post treatment								
		m^3.d-1						18	50
72	Structures	$m^3.d^{-1}$				70.00		18	50
73	Electro-mechanical	$m^3.d^{-1}$				63.00		18	30
74	Media/chemicals	$m^3.d^{-1}$						6	10
75	Other	$m^3.d^{-1}$						18	50
76	Contingency	%	10.0					18	50

(continued)

80	Brine disposal								
81	Pipelines	m				3,300.00		24	30
82	Structures	$m^3.d^{-1}$						24	50
83	Electro-mechanical	$m^3.d^{-1}$						24	15
84	Other	$m^3.d^{-1}$						24	30
85	Contingency	%	20.0					24	30

	Operation & maintenance costs			Chemicals		Energy	
	Fraction of investment	Additional annual cost					
		LC	FC	Type	Consumption	Type	Consumption
	(%)	IS	US$	(–)	($gm.m^{-3}$)	(–)	($kWh.m^{-3}$)
10							
11							
12	1.0						
13							
20							
21							
22							
23	3.0					C	0.35
24	1.0						
25	0.5						
26	0.5						
27	3.0						
28							

(continued)

30								
31								
32	1.0			$FeCl_3$	0.00			
33	3.0			H_2SO_4	10.00	A	0.11	
34	3.0			Cl	2.00			
35				NaOCl	0.00			
36								
60								
61	3.0					A	2.70	
62	1.0							
63	3.0							
64	3.0							
65								
66								
70								
72	1.0			$NaHSO_3$				
73	3.0			$CaCO_3$	0.00	A	0.58	
74	3.0			NaOH	10.00			
75								

(continued)

76								
80								
81	1.0							
82	1.0							
83	3.0							
84								
85								

Calculated Costs										
	Investments		Interest during construction		Capital*		O&M		Chemicals	
	LC	FC	LC	FC	LC	FC	LC	FC	LC	FC
	IS	US$	IS	US$	IS	US$	IS	US$	IS	US$
Land and site	4,992,000	0	84,897	0	304,614	0	10,400	0	0	0
Intake	2,821,782	4,106,324	145,025	120,938	205,374	295,894	3,713	15,625	0	0
Pretreatment plant	0	6,011,881	163,587	136,559	15,992	556,855	0	153,267	0	190,000
Desalination plant*	0	14,748,386	773,403	643,616	55,525	1,216,595	0	384,229	0	0
Post treatment	0	8,891,089	369,094	307,614	24,880	532,734	0	153,861	0	140,000
Brine disposal	0	2,574,000	149,362	124,280	10,851	175,527	0	21,450	0	0
Total	7,813,782	36,131,681	1,685,369	1,333,006	617,236	2,777,605	14,113	728,432	0	330,000
Total single currency × 1000	152,341	36,085	7,017	1,754	11,728	2,932	2,928	732	1,320	330
Percentage:					44.3%		11.0%		5.0%	

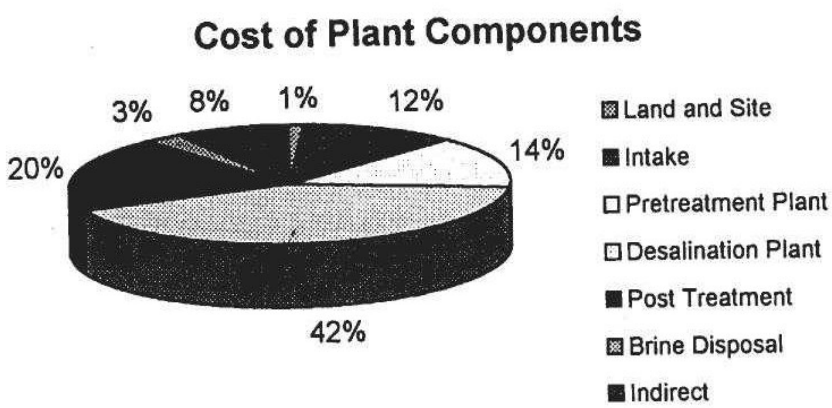
Energy		Total		Unit costs (for one m^3) by currency		Percentage (%)	Total unit costs (for one m^3)	
LC	FC	LC	FC	LC	FC			
IS	US$	IS	US$	IS	US$		IS	US$
0	0	315,014	0	0.031	0.000	1.2	0.031	0.008
1,897,000	0	2,106,087	311,516	0.209	0.031	12.7	0.332	0.083
462,000	0	477,992	900,123	0.047	0.089	15.4	0.404	0.101
5,726,700	0	5,782,225	1,600,823	0.572	0.158	46.0	1.206	0.302
2,436,000	0	2,460,880	826,596	0.244	0.082	21.8	0.571	0.143
0	0	10,851	196,977	0.001	0.020	3.0	0.079	0.020
10,521,700	0	11,153,049	3,836,037	1.104	0.380	100.0	2.623	0.656
10,522	2,630	26,497	6,624					
39.7%		100%						
*Including membranes		Indirect cost		0.099	0.034	9.0	0.236	0.059
		Cost including indirect:		1.204	0.414	100.0	2.860	0.715

No. of wells	25
Length of gallery/collector (m)	18182
Load factor	0.92
Equivalent fraction penalised	8.26%

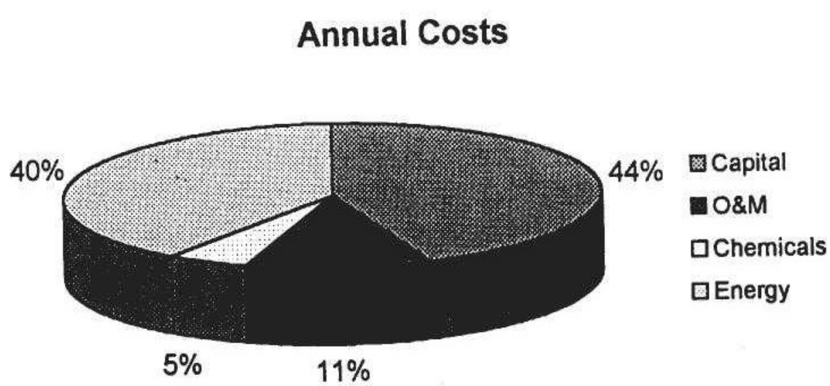
Cost Estimate of Desalination Intakes - Graphs

24/08/2000 12:07

TEST2



Graphs



CostEstimate7.xls

Cost Estimate of Desalination Intakes

Cost estimate of desalination and Intakes – model

A. Plant name: Test2		Type of Plant: SWRO				Type of Intake: Beach wells

B: Desalination plant		
Plant capacity	$m^3.day^{-1}$	30,000
Annual production	$m^3.y^{-1}$	10,100,000
Feed water salinity	$mg.L^{-1}$	35,692
Recovery	%	1
Salinity Rejection	%	0.0
Site area	m^2	10,400
Distance from Intake	m	200
Distance to brine disposal	m	650

C. Hydrogeology		
GW flow rate	$m^3.y^{-1}.m^{-1}$	1,100
Fresh water fraction pumped	%	10.0
Fraction penalised*	%	8.3
Single well capacity	$m^3.d^{-1}$	2,400
Depth of well	m	100

(continued)

Depth to dynamic water table	m	50
Length of well screen	m	10
Length of collector / Gallery	m	0

*Fraction of water pumped on account of Inland Supply.	
No of wells	24.75
Collection length m	18182
Load factor	0.922

Test2 (continued)

D. Finance

Legal currency	IS		
Foreign currency	US$		
Exchange rate	4.00		
Date of cost estimates	1/1/00		
Intert. rate – local	6%	Interest rate – foreign	5%
Monthly interest – local	0.49%	Monthly interest – foreign	0.41%

F. Unit costs of energy

Taanf Type	Weighted mean rate		Unit costs by time rates						Fraction of hours per year			
			LC IS/KWH			FC US$/KWh			%			
	LC	FC	Low	Regular	Peak	Low	Regular	Peak	Low %	Regular %	Peak %	10 %
A	0.21	0.0	0.15	0.20	0.30	0.00	0.00	0.00	40.0	30.0	30.0	10 %
B	0.22	0.00	0.20	0.25	0.40	0.00	0.00	0.00	40.0	6.0	30.0	
C	0.27	0.00	0.25	0.30	0.50	0.00	0.00	0.00	40.0	7.0	30.0	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		

E. Unit costs of chemicals

Type	Cost of 1 kg	
	LC	FC
	IS	US$
$FeCl_3$	0	2
H_2SO_4	0	1
Cl	0	1
NaOCl	0	1
$NaHSO_3$	0	1
$CaCO_3$	0	1
NaOH	0	1

G. Desalination plant and Intake components costs

Ser No.	Item	Unit	Quantity	Investments				Schedule		Interest during construction	
				LC		FC		Construction	Depreciation	LC	FC
				Unit cost	Total cost	Unit cost	Total cost				
				IS	IS	US$	US$	(Months)	(Years)	IS	US$
10	Land and site										
11	Land acquisition /rental	m^2	10,400	300.00	3,120,000	0.00	0	12	1,000	84,897	0
12	Site development / infrastructure	m^2	10,400	100.00	1,040,000	0.00	0	12	1,000	0	0
13	Contingency	%	20.0		832,000		0	12	1,000	0	0
	Sub total				4,992,000		0			84,897	0
20	Intake	$m^3.y^{-1}$	20,000,000								
21	Well drilling	m	2,475	800.00	1,980,198	0.00	0	12	35	0	0
22	Pump columns	m	1,485	0.00	0	700.00	1,039,604	12	20	28,288	23,614
23	Wells pumps and electricity	–	25	0.00	0	6,000.00	148,515	12	15	4,041	3,373
24	Wells infrastructure	–	25	15,000.00	371,287	0.00	0	12	50	0	0
25	Collector /gallery	m	10,182	0.00	0	120.00	2,181,818	18	30	92,658	77,224
26	Connecting pipeline	m	200	0.00	0	260.00	52,000	12	30	1,415	1,181
27	Low level pumping	$m^3.d^{-1}$	59,406	0.00	0	0.00	0	6	20	0	0

(continued)

28	Contingency	%	20.0		470,297		684,387	12	30	18,623	15,546
	Sub total				2,821,782		4,106,324			145,025	120,938
30	Pretreatment plant	$m^3.y^{-1}$	20,000,000								
31		$m^3.d^{-1}$	59,406								
32	Structures	$m^3.d^{-1}$	59,406	0.00	0	9.00	534,653	12	50	14,548	12,145
33	Electro-mechanical	$m^3.d^{-1}$	59,406	0.00	0	41.50	2,465,347	12	30	67,084	56,000
34	Media	$m^3.d^{-1}$	59,406	0.00	0	41.50	2,465,347	12	10	67,084	56,000
35	Chemicals	$m^3.y^{-1}$	20,000,000	0.00	0	0.00	0	0	0		
36	Contingency	%	10.0		0		546,535	12	30	14,872	12,414
	Sub total				0		6,011,881			163,587	136,559
60	Desalination plant	$m^3.y^{-1}$	10,100,000								
60	Desalination plant	$m^3.y^{-1}$	10,100,000								
61	High pressure pumps + turbine	$m^3.d^{-1}$	59,406	0.00	0	105.00	6,237,624	24	50	361,951	301,169
62	Structures	$m^3.d^{-1}$	30,000	0.00	0	30.00	900,000	24	50	52,224	43,454
63	Electro-mechanical	$m^3.d^{-1}$	30,000	0.00	0	149.00	4,470,000	24	30	259,381	215,824
64	Membranes	$m^3.d^{-1}$	30,000	0.00	0	60.00	1,800,000	6	5	22,047	18,433
65	Other	$m^3.d^{-1}$	30,000	0.00	0	0.00					
66	Contingency	%	10.0		0		1,340,762	24	40	77,800	64,736
	Sub total				0		14,748,386	0	0	773,403	643,616
70	Post treatment	$m^3.y^{-1}$	18,100,000								

(continued)

71	Infra structure	$m^3.d^{-1}$	59,406	0.00	0	0.00	0	18	50	0	0
72	Structures	$m^3.d^{-1}$	59,406	0.00	0	70.00	4,158,416	18	50	176,600	147,184
73	Electro-mechanical	$m^3.d^{-1}$	59,406	0.00	0	63.00	3,742,574	18	30	158,940	132,465
74	Media /chemicals	$m^3.d^{-1}$	59,406	0.00	0	0.00	0	6	10	0	0
75	Other	$m^3.d^{-1}$	59,406	0.00	0	0.00	0	18	50	0	0
76	Contingency	%	10.0		0		790,099	18	50	33,554	27,965
	Sub total				0		8,691,089	0	0	369,094	307,614
80	Brine disposal	$m^3.y^{-1}$	9,900,000								
81	Pipelines	m	650	0.00	0	3,300.00	2,145,000	24	30	124,468	103,566
82	Structures	$m^3.d^{-1}$	0	0.00	0	0.00	0	24	50	0	0
83	Electro-mechanical	$m^3.d^{-1}$	0	0.00	0	0.00	0	24	15	0	0
84	Other	$m^3.d^{-1}$	0	0.00	0	0.00	0	24	30	0	0
85	Contingency	%	20.0		0		429,000	24	30	24,894	20,713
	Sub total				0		2,574,000			149,362	124,280
	Grand total				7,813,782		36,131,681			1,685,369	1,333,006

Ser No.	Annual capital costs		Operation & maintenance costs		Chemicals		Energy		Annual direct costs		10%		
	LC		FC		Fraction of investment		Total annual O&M costs						
				LC	FC	LC	FC	LC	FC	LC	FC		FC
	IS	US$	(%)	IS	US$	IS	US$	IS	US$	IS	US$		US$
10													
11	192,294	0	0.0	0	0					192,294	0	0.019	0.000
12	62,400	0	1.0	10,400	0					72,000	0	0.007	0.000
13	49,920	0	0.0	0	0					49.920	0	0.005	0.000
	304,614	0		10,400	0					315,014	0	0.031	0.000
20													
21	136,582	0	0.0	0	0	0	0	0	0	136,582	0	0.014	0.000
22	2,466	85,315	0.0	0	0	0	0	0	0	2,466	85,315	0.000	0.008
23	416	14,633	3.0	0	4,455	0	0	1,897,000	0	1,897,416	19,089	0.188	0.002
24	23,554	0	1.0	3,713	0	0	0	0	0	27,269	0	0.003	0.000
25	6,731	146,954	0.5	0	10,909	0	0	0	0	6,731	157,863	0.001	0.016
26	103	3,460	0.5	0	260	0	0	0	0	103	3,720	0.000	0.000
27	0	0	3.0	0	0	0	0	0	0	0	0	0.000	0.000
28	35,519	45,532	0.0	0	0	0	0	0	0	35,519	45,532	0.004	0.005
	205,374	295,894		3,713	15,625	0	0	1,897,000	0	2,106,087	311,518	0.209	0.031
30													
31													
32	923	29,952	1.0	0	5,347	0	0	0	0	923	35,298	0.000	0.003
33	4,874	164,017	3.0	0	73,960	0	150,000	462,000	0	466,874	387,976	0.046	0.038
34	9,115	326,526	3.0	0	73,960	0	40,000	0	0	?,115	440,486	0.001	0.044
35			0.0	0	0	0	0	0	0	0	0	0.000	0.000
36	1,080	34,360	0.0	0	0	0	0	0	0	1,080	36,360	0.000	0.004

(continued)

sub total	15,992	556,855		0	153,267	0	190,000	462,000	0	477,992	900,123	0.047	0.089
60													
61	22,964	358,174	3.0	0	187,129	0	0	5,726,700	0	5,749,664	545,302	0.569	0.054
62	3,313	51,679	1.0	0	9,000	0	0	0	0	3,313	60,679	0.000	0.006
63	18,844	304,820	3.0	0	134,100	0	0	0	0	18,844	438,920	0.002	0.043
64	5,234	420,012	3.0	0	54,000	0	0	0	0	5,234	474,012	0.001	0.047
65			0.0	0	0	0	0	0	0	0	0	0.000	0.000
66	5,171	81,910	0.0	0	0	0	0	0	0	5,171	81,910	0.001	0.008
	55,525	1,216,595		0	384,229	0	0	5,726,700	0	5,782,225	1,600,823	0.572	0.158
70													
71	0	0	0.0	0	0	0	0	0	0	0	0	0.000	0.000
72	11,204	235,847	1.0	0	41,584	0	0	0	0	11,204	277,431	0.001	0.027
73	11,547	252,077	3.0	0	112,277	0	0	2,436,000	0	2,447,547	364,354	0.242	0.036
74	0	0	3.0	0	0	0	140,000	0	0	0	140,000	0.000	0.014
75	0	0	0.0	0	0	0	0	0	0	0	0	0.000	0.000
76	2,129	44,811	0.0	0	0	0	0	0	0	2,129	44,811	0.000	0.004
	24,880	532,734		0	153,861	0	140,000	2,436,000	0	2,460,880	826,596	0.244	0.082
80													
81	9,042	146,272	1.0	0	21,450	0	0	0	0	9,042	167,722	0.001	0.017
82	0	0	1.0	0	0	0	0	0	0	0	0	0.000	0.000
83	0	0	3.0	0	0	0	0	0	0	0	0	0.000	0.000
84	0	0	0.0	0	0	0	0	0	0	0	0	0.000	0.000
85	1,808	29,254	0.0	0	0	0	0	0	0	1,808	29,254	0.000	0.003
	10,851	175,527		0	21,450	0	0	0	0	10,851	196,977	0.001	0.020
	617,236	2,777,605		14,113	728,432	0	330,000	10,521,700	0	11,153,049	3,836,037	1.104	0.380

APPENDIX - E

EXAMPLES OF MEMBRANE DESIGN CALCULATIONS AND PROCESS DESIGN SPREADSHEETS FOR SWRO PLANT

Contents

Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

Hydranautics RO system design software, Version 6.4 (c) 1998 3/26/2000
Basic Design

RO program licensed to:	ALCID & CO		
Calculation created by:	S. SHAPIRA		
Project name: SEAMED31		Permeate flow:	1389.0 $m^3.h^{-1}$
HP pump flow:	2780.1 $m^3.h^{-1}$	Raw water flow:	2778.0 $m^3.h^{-1}$
Recommended pump press:	78.2 bar		
Feed pressure:	74.4 bar	Permeate recovery ratio:	50.0%
Feed water temperature:	20.0 C (68F)		
Raw water pH:	8.00	Element age:	3.0 years
Acid dosage, ppm (100%):	0.0 (none)	Flux decline,%.y^{-1}:	7.0
Acidified feed CO2:	2.1	Salt passage increase,%.y^{-1}	10.0
Average flux rate:	9.0 gfd	Feed type:	Seawater – well

Stage	**Perm: flow**	**Flow per feed**	**Vessel conc:**	**Flux**	**Beta**	**Conc: press**	**Element type**	**Elem: No.**	**Array**
	$m^3.h^{-1}$	$m^3.h^{-1}$	$m^3.h^{-1}$	gfd		bar			
1–1	1,155.4	7.9	4.6	11.1	1.07	73.3	SWC1	2100	350×6
1–2	234.7	9.6	8.2	4.6	1.02	71.2	SWC1	1020	170×6

Appendix E (continued)

	Raw water		Feed water		Permeate		Concentrate	
Ion	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$
Ca	396.0	19.8	396.0	19.6	0.8	0.0	791.2	39.5
Mg	1,280.0	105.3	1,280.0	105.3	2.5	0.2	2,557.5	210.5
Na	11,007.0	478.6	11,007.0	478.6	101.5	4.4	21,912.3	952.7
K	397.0	10.2	397.0	10.2	4.6	0.1	789.4	20.2
NH_4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ba	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sr	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CO_3	0.5	0.0	0.5	0.0	0.0	0.0	0.9	0.0
HCO_3	113.0	1.9	113.0	1.9	1.7	0.0	224.3	3.7
SO_4	3,008.0	62.7	3,008.0	62.7	6.3	0.1	6,009.7	125.2
Cl	19,491.0	549.8	19,491.0	549.6	163.4	4.6	38,818.6	1,095.0
F	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
NO_3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SiO_2	0.0		0.0		0.0		0.0	
TDS	35,692.5		35,692.5		280.7		71,104.2	
pH	8.0		8.0		6.2		8.5	

Appendix E (continued)

	Raw water	Feed water	Concentrate
$CaSO_4$ / Ksp * 100:	22%	22%	52%
$SrSO_4$ / Ksp * 100:	0%	0%	0%
$BaSO_4$ / Ksp * 100:	0%	0%	0%
SiO_2 saturation:	0%	0%	0%
Langelier saturation index	0.82	0.82	1.90
Stiff & Davis saturation index	–0.11	–0.11	0.89
Ionic strength	0.73	0.73	1.52
Osmotic pressure	26.6 bar	26.6 bar	55.0 bar

These calculations are based on nominal element performance when operated on a feed water of acceptable quality. No guarantee of system performance is expressed or implied unless provided in writing by Hydranautics.

Hydranautics (USA) Ph: (619) 901-2500 Fax: (619) 901-2578
Hydranautics (Europe) Ph: 31 5465 49335 Fax: 31 5465 49337

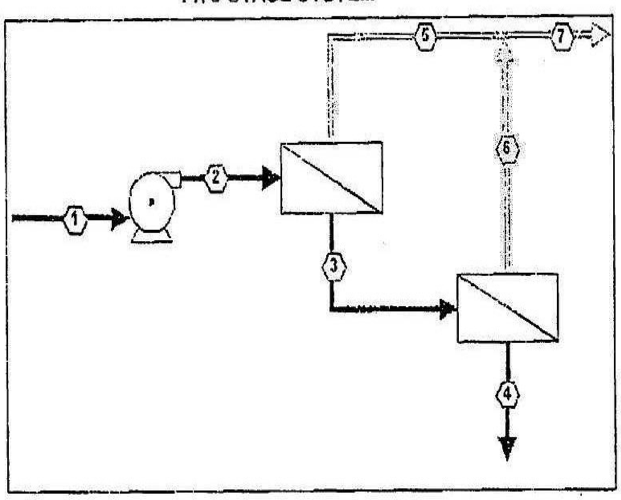
Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

HYDRANAUTICS RO SYSTEM DESIGN SOFTWARE, VERSION 6.4 (c) 1998 3/26/2000

BASIC DESIGN

SEAMED31 19-03-2000

TWO STAGE SYSTEM



		1	2	3	4	5	6	7
Flow	m3/hr	2780.1	2780.1	1624.7	1390.1	1155.4	234.7	1389.0
Pressure	bar	0.0	74.4	73.3	71.2	0.0	0.0	0.0
Salinity	ppm TDS	35692.5	35692.5	60930.5	71104.2	202.0	667.0	280.7

Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

Hydranautics RO system design software, Version 6.4 (c) 1998 3/26/2000
Basic Design

RO program licensed to:	ALCID & CO		
Calculation created by:	S. SHAPIRA		
Project name: SEAMED31		Permeate flow:	1389.0 $m^3.h^{-1}$
HP pump flow:	2780.1 $m^3.h^{-1}$	Raw water flow:	2778.0 $m^3.h^{-1}$
Recommended pump press:	78.2 bar		
Feed pressure:	74.4 bar	Permeate recovery ratio:	50.0%
Feed water temperature:	20.0 C (68F)		
Raw water pH:	8.00	Element age:	5.0 years
Acid dosage, ppm (100%):	0.0 none	Flux decline,%.y^{-1}:	7.0
Acidified feed CO_2:	2.1	Salt passage increase,%.y^{-1}	10.0
Average flux rate:	9.0 gfd	Feed type:	Seawater – well

Stage	**Perm: flow**	**Flow per feed**	**Vessel conc:**	**Flux**	**Beta**	**Conc: press**	**Element type**	**Elem: No.**	**Array**
	$m^3.h^{-1}$	$m^3.h^{-1}$	$m^3.h^{-1}$	gfd		bar			
1–1	1,155.4	7.9	4.6	11.1	1.07	73.3	SWC1	2100	350×6
1–2	234.7	9.6	8.2	4.6	1.02	71.2	SWC1	1020	170×6

Calculation of power requirement

Feed pressure, bar	74.4	Pump efficiency,%	87.0	Pumping energy, $kWh.m^{-3}$	3.08
Concentrate pressure, bar	71.2	Motor efficiency,%	97.0	Pumping power, kW	6,671.5
Permeate flow, $m^3.h^{-1}$	1,389.0	ERT efficiency,%	89.0	Recovered power, kW	2,397.6
Recovery ratio,%	50.0	ERT backpressure, bar	0.0	Power requirement, Kw	4,273.9

Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

Hydranautics RO system design software, Version 6.4 (c) 1998 3/26/2000
Basic Design

RO program licensed to: ALCID & CO
Calculation created by: S. SHAPIRA

Project name: SEAMED11		Permeate flow:	1389.0 $m^3.h^{-1}$
Hp pump flow:	2780.1 $m^3.h^{-1}$	Raw water flow:	2778.3 $m^3.h^{-1}$
Recommended pump press:	86.6 bar		
Feed pressure:	82.5 bar	Permeate recovery ratio:	50.0%
Feed water temperature:	20.0 C (68F)		
Raw water pH:	8.00	Element age:	5.0 years
Acid dosage, ppm (100%):	0.0 HCl	Flux decline,%.y^{-1}	7.0
Acidified feed CO_2:	2.3	Salt passage increase,%.y^{-1}	10.0
Average flux rate:	9.0 gfd	Feed type:	Seawater – well

Stage	**Perm: flow**	**Flow per feed**	**Vessel conc:**	**Flux**	**Beta**	**Conc: press**	**Element type**	**Elem: No.**	**Array**
	$m^3.h^{-1}$	$m^3.h^{-1}$	$m^3.h^{-1}$	gfd		bar			
1–1	1,172.4	7.9	4.6	11.2	.07	81.4	SWC1	2100	350×6
1–2	217.6	9.5	8.2	4.3	1.02	79.3	SWC1	1020	170×6

SEAMED11 (continued)

	Raw water		Feed water		Permeate		Concentrate	
Ion	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$	$meq.l^{-1}$
Ca	440.0	21.9	440.0	21.9	0.9	0.0	879.1	43.8
Mg	1,423.0	117.1	1,423.0	117.1	2.8	0.2	2,843.2	234.0
Na	12,231.0	531.8	12,231.0	531.8	113.7	4.9	24,348.3	1,058.6
K	441.0	11.3	441.0	11.3	5.1	0.1	876.9	22.5
NH_4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ba	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sr	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CO_3	0.5	0.0	0.5	0.0	0.0	0.0	1.0	0.0
HCO_3	126.0	2.1	126.0	2.1	1.9	0.0	250.1	4.1
SO_4	3,342.0	69.6	3,342.0	69.6	7.1	0.1	6,676.9	139.1
Cl	21,657.0	610.9	21,657.0	610.9	183.1	5.2	43,130.9	1,216.7
F	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
NO_3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SiO_2	0.0		0.0		0.0		0.0	
TDS	39,660.5		39,660.5		314.5		79,006.5	
pH	8.0		8.0		6.2		8.5	

	Raw water	Feed water	Concentrate
$CaSO_4$ / Ksp * 100:	25%	25%	59%
$SrSO_4$ / Ksp * 100:	0%	0%	0%
$BaSO_4$ / Ksp * 100:	0%	0%	0%
SiO_2 saturation:	0%	0%	0%
Langelier saturation index	0.91	0.91	1.99
Stiff & Davis saturation index	–0.04	–0.04	0.98
Ionic strength	0.82	0.82	1.70
Osmotic pressure	29.7 bar	29.7 bar	61.7 bar

These calculations are based on nominal element performance when operated on a feed water of acceptable quality. No guarantee of system performance is expressed or implied unless provided in writing by Hydranautics.

Hydranautics (USA) Ph: (619) 901-2500 Fax: (619) 901-2578, Hydranautics (Europe) Ph: 31 5465 049335 Fax: 31 5465 49337

Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

Hydranautics RO system design software, Version 6.4 (c) 1998

Project name: SEAMED11 19-03-2000
Water source: Seawater – well

Water analysis

pH	8.00	Turb	0.0	E. cond	61223	CO_2	2.3	H_2S	0.0
Temp	20.0 C	SDI	0.0	TDS	39661	Fe	0.0		

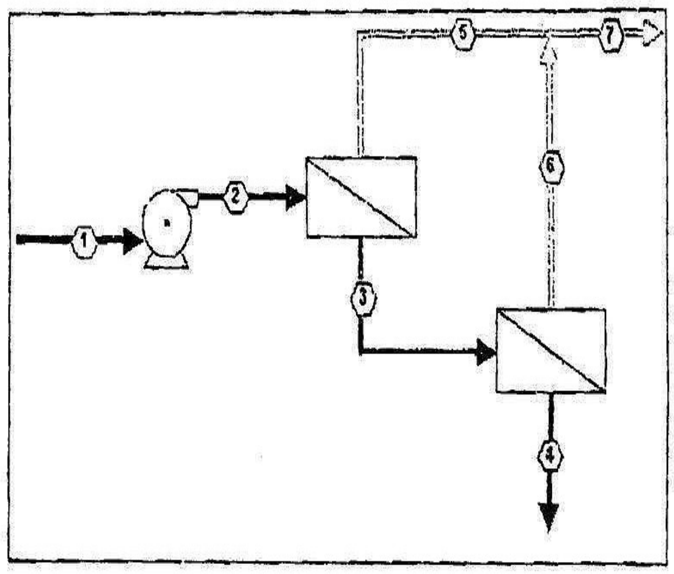
	$mg.l^{-1}$	$meq.l^{-1}$	$mg.l^{-1}$		$mg.l^{-1}$	$meq.l^{-1}$
Ca	440.0	22.0	440.0	CO_3	0.5	0.0
Mg	1,423.0	117.1	1,423.0	HCO_3	126.0	2.1
Na	12,231.0	531.8	12,231.0	SO_4	3,342.0	69.6
K	441.0	11.3	441.0	Cl	21,657.0	610.9
NH_4	0.0	0.0	0.0	F	0.0	0.0
Ba	0.0	0.0	0.0	NO_3	0.0	0.0
Sr	0.0	0.0	0.0	SiO_2	0.0	0.0

$CaSO_4$ saturation	26%
$SrSO_4$ saturation	0%
$BaSO_4$ saturation	0%
SiO_2 saturation:	0%
LSI	0.9
SDI	0.0
Ionic strength	0.819
Osmotic pressure	430.7

Appendix E: Examples of membrane design calculation and process design spreadsheet for SWRO plant

SEAMED11 19-363-2bu4

TWO STAGE SYSTEM



		1	2	3	4	5	6	7
Flow	m3/hr	2780.1	2780.1	1607.7	1390.1	1172.4	217.6	1389.0
Pressure	bar	0.0	82.5	81.4	79.3	0.0	0.0	0.0
Salinity	ppm TDS	39660.5	39660.5	68545.7	79006.5	50.8	1733.6	314.6

Appendix E: Examples of membrane design calculation and process design spreadsheets for SWRO plant

Hydranautics RO system design software, Version 6.4 (c) 1998 3/26/2000
Basic Design

RO program licensed to:	ALCID & CO		
Calculation created by:	S. SHAPIRA		
Project name: SEAMED11		Permeate flow:	1389.0 $m^3.h^{-1}$
HP pump flow:	2780.1 $m^3.h^{-1}$	Raw water flow:	2778.0 $m^3.h^{-1}$
Recommended pump press:	86.6 bar		
Feed pressure:	82.5 bar	Permeate recovery ratio:	50.0%
Feed water temperature:	20.0 ^{0}C (68 ^{0}F)		
Raw water pH:	8.00	Element age:	5.0 years
Acid dosage, ppm (100%):	0.0 HCl	Flux decline,%.y^{-1}:	7.0
Acidified feed CO_2:	2.3	Salt passage increase,%.y^{-1}	10.0
Average flux rate:	9.0 gfd	Feed type:	Seawater – well

Stage	Perm: flow	Flow per feed	Vessel conc:	Flux	Beta	Conc: press	Elem: type	Elem: No	Array
	$m^3.h^{-1}$	$m^3.h^{-1}$	$m^3.h^{-1}$	gfd		bar			
1–1	1,172.4	7.9	4.6	11.2	1.07	81.4	SWC1	2100	350×6
1–2	217.6	9.5	8.2	4.3	1.02	79.3	SWC1	1020	170×6

Calculation of power requirement

Feed pressure bar	82.5	Pump efficiency %	87.0	Pumping energy $kWh.m^{-3}$	3.40
Concentrate pressure bar	79.3	Motor efficiency %	97.0	Pumping power kW	7,397.8
Permeate flow $m^3.h^{-1}$	1,389.0	ERT efficiency %	89.0	Recovered power kW	2,670.4
Recovery ratio %	50.0	ERT backpressure bar	0.0	Power requirement kW	4,727.4

APPENDIX - F

THE ISRAEL COASTAL AQUIFER

Contents

1. THE ISRAEL COASTAL AQUIFER

The coastal aquifer extends over 120 km of Israel's Mediterranean coast from Mount Carmel in the north to the Gaza Strip in the south. The width of this aquifer varies from 3 – 10 km in the north to 20 km in the south. The total area of the aquifer is 1,800 km^2.

The coastal aquifer is composed of sandstone and sand layers of Pliocene - Pleistocene age, extending from the surface to depths of 150 - 180 m at the coastline, decreasing eastwards to zero at the Judean and Samarian foothills. Near the coast it is subdivided into sub-aquifers by thin impervious to semi-impervious layers.

Natural replenishment of the aquifer is from rainfall over the coastal plain (400.-.600 mm per year). Mean annual natural replenishment is estimated at about 300 MCM per year. In addition to natural replenishment, 90 MCM per year infiltrates to the aquifer from irrigated areas, septic tanks, and leaks from the water supply system, *etc*. The safe yield of the aquifer is estimated at 240 – 300 MCM per year. Artificial recharge of the coastal aquifer from the national water system and by stream flow retention projects; is practiced by means of 60 wells and 10 spreading basins.

Seawater intrudes into the aquifer and sub-aquifers in a wedge like form, with a thin transition zone between seawater and fresh water approximated by a sharp interface. The toe of the interface intrudes inland as a function of the fresh water level. An acceptable distance of intrusion ranging from 1,000 m to 2,000 m has been established in order to allow optimal water abstraction for beneficial uses.

Specific well yields and hydraulic transmissivities vary quite markedly due to the heterogeneity of the aquifer. Average transmissivities range from 1,000 to 2,000 m^2 per day.

Water levels declined steeply in the 1950’s and early 1960’s, until the National Water Carrier was commissioned. Mining of the aquifer in this period temporarily supported the expansion of irrigated areas that were later supplied from the NWC. In the late 1960’s and early 1970’s, water levels were partly recovered by large - scale artificial recharge and reduced pumping. Overexploitation in the 1980’s has resulted in the continued decline of water levels, this stopped in 1986. In 1992, due to exceptionally high rainfall, water levels were restored to the levels of the 1970’s.

Owing to the aquifer's heterogeneity and low transmissivity, as well as over exploitation in certain areas, water levels in some regions have declined much more than in others, creating water table depressions below sea level. In addition, the seawater - groundwater interface has moved inland in the overexploited parts of the aquifer, to an extent, which has caused a high increase in salinity of production wells. To counteract these effects, some 20 to 70 MCM per year have been artificially recharged into the aquifer, and pumpage in some areas has been substituted by water imported from the NWC.

Which policy to follow in order to balance pumping and artificial recharge over the different parts of the aquifer with aquifer yield, the position of the interface, and the residual outflow to the sea, has been the subject of many studies using detailed physical and numerical models. It has been established that the seawater - groundwater interface should be maintained at an average distance of no more than 1,500 m inland from the coastline by appropriate distribution of pumping and artificial recharge operations, and by containing water table fluctuations within predetermined levels in each part of the aquifer.

Hydrological monitoring of the coastal aquifer is based on a network of 300 special observation wells on a 2 km grid and regular monitoring of some 1700 pumping wells. In the west, a line of special multi - piped observation wells has been installed to monitor seawater intrusion in the different sub-aquifers.

Planning the management of the aquifer in the initial stages of the National Water System development in the 50's and 60's, was aided with groundwater simulation models, these models were of the Hele-Shaw type. The models represented the groundwater flow and seawater intrusion in vertical two-dimensional cross sections by the viscous flow of oils between two Perspex sheets. Processes such as the transient impact of pumping and recharge on the water table and on seawater intrusion as well as the mixture of recharged water with the indigenous aquifer water were studied.

The development of digital computing hardware and software in the 70's resulted in mathematical finite difference and finite element models to simulate groundwater flow and water level variations as well as seawater intrusion. The coastal aquifer was represented by a horizontal grid of regular 2 x 2 km squares (excluding the first coastal square, which is a 1 x 2 km cell). This division is known as USOM strips and squares. The models included additional surrounding boundary cells. The western boundary cells represented seawater intrusion by an approximate transformation. Typical data of this model are shown in the following section 12.

2. GEOMORPHOLOGY [2]

The land surface is characterized by kurkar (the local calcareous sandstone) ridges and troughs, overlain by sand dunes. Three ridges constitute the most well expressed features. The Coastal Ridge, 500 - 700 m from the shore line, the 2nd ridge, 1800 - 2000 m from the shore line and the 3rd ridge, the main one is at a distance of 4 - 5.5 km from the sea. The maximal elevations of the ridges are 40 m, 40 m and 60 m, respectively.

The troughs are stretching in between the ridges, a narrow trough in the west and a wide one in the east. This ridge and trough configuration predominates all over the Coastal Plain but changes locally because of river debouchments and their associated thick clayey deposition. The area is covered with sand dunes, the main concentration and thickness of which is in the eastern trough. The dunes cover the clays, within the trough, the loam's on the ridge flanks and the kurkar. However, the kurkar crops out of the sand mantle along the seaward flank of the coastal ridge.

[2] This section and the following sections relate to the Nitzanim area, which is an example of SWRO application in the present study.

3. HYDROGEOLOGY

3.1. General

The Pleistocene Aquifer is subdivided into four sub-aquifers, namely A (the uppermost one), B, C and D, respectively (see geological cross section - Chart 4). The subdivision is created, invariably by the clay, silt and loam intercalations within the predominantly kurkar sequence. This system of intermediate impervious to semi-impervious horizons dissipates gradually eastwards and pinches out, to be replaced by non-systematic, local, and inconsistent layers and the aquifer becomes uniform and undivided.

A brief description of the hydrogeological units is given:

3.2. Sub-Aquifer A

Concretional calcareous sandstone (irregular, platy) usually is rather friable, mixed with loose sand. The rock is predominantly continental origin with a rather low percentage of reworked and weathered macrofaunal fragments but occasionally calcified lumachelle may appear.

The bottom of Sub-Aquifer A, at a depth of 40 - 50 m along the shoreline, constitutes a continental originated clay or loam. This layer usually extends to 0.5 - 1 km eastwards (see geological cross section) and tends to slope up and merge with the 2nd trough clays. This model is not that easily determined and is supported, in places, by microfaunal evidence (Marginopora Sp., denoting the Tyrrhenian period), when the underlying beds of Sub-Aquifer B were deposited.

In places, the extension of this bottom layer is not that consistent and may wedge out and disappear at 400 m from the shoreline. That is, Sub-Aquifer A may merge, hydrologically, with the upper units of the underlying Sub-Aquifer B.

Some occasional loamy lenses may appear within this complex in addition to the bottom layer. The kurkar is usually covered by a loam mantle, a predominantly weathering product which becomes finer grained and clayey within the 1st trough. A perched water horizon may exist over these clays during the rainy season.

3.3. Sub-Aquifer B

The main and thickest aquiferous unit, approximately 60 m, is in the study area. The lithology is mainly of concretional calcareous sandstone of mainly ‘coastal’ to shallow marine origin, such as beach rock, conglomerate, and lumachelle. These beds are usually associated with the marine-estuarine basal clays.

The bottom clays are at -120 m to -110 m at the shoreline and sloping upwards; due east. The lithology changes from grey marine clay in the west; to black lagunar-estuarine clays in the east and contain pockets of Cardium Sp. in places. This clay horizon may extend 6 - 7 km eastwards, sometimes as continental originated clay and loam at its easternmost reaches.

Sub-Aquifer B attains, usually, a secondary subdivision into B1, B2 and B3 subunits. This subdivision, a product of mainly marine originated clays and silts, is somewhat inconsistent within the study area. There are differences between the north and south, however, the B3 seems to be the main subunit. B1 and B2 tend to merge into one unit several hundred metres east from the coast and with Sub-Aquifer A, throughout most of the study area.

3.4. Sub-Aquifer C

The environment of deposition is predominantly marine, therefore a high-energy bed, such as beach rock is not that common. The basal, marine originated clay is at -140 m to -160 m at the shoreline. The thickness of the kurkar beds is 20 - 40 m, but due to additional clay horizons within the B and C units, the accurate boundary between the two may be indiscernible, hence the frequent ‘thickness variations’ of these sub-aquifers. The problem may also stem of the difficulty to outline the underlying Sub-Aquifer D, let alone, the unavailability of reliable electrical logs.

3.5. Sub-Aquifer D

A marine originated unit with no shallower originated derivates. Hence, the kurkar is commonly intercalated by clay and silt and the obvious outcome is a much lower transmissivity. The top Saqie Group (or Base Kurkar), constitutes the sub aquifer bottom, the marker of which is, in places, a thin sequence of interlayered chalky, marly sandstone, chalky marl and clays. The thickness at the shoreline is up to 10 - 20 m. The proportion of kurkar increases due east, parallel to the pinching out of the intermediate fine to very fine grained horizons.

3.6. The Saqie group

A very thick complex of shales and marls, wedging out and thinning, due east, in the study area, from 2,200 m to 1,500 m, with a westward gradient (for Saqie/Base Kurkar) of 1% to 2%. The uppermost layers there belong to Yaffo Fm. Petah Tikva Mbr. (Up. Miocene-L. Pleistocene). In places, this member includes some shell and sand lenses, containing brines and, occasionally; gas traps.

3.7. Seawater intrusion

The estimate of underground seawater intrusion in the 50’s and 60’s relied on a battery of observation wells drilled at distances of 500 - 1500 m from the sea with some pipes opened at different depths in the same borehole with proper vertical separations. The monitoring was performed by sampling and by electric conductivity logging with electrodes regularly introduced into all pipes. This monitoring system has been abandoned because of seawater intrusion and structural failures.

Measuring depth to saline water underground with TDEM (Tine Domain Electromagnetic Method) is now considered to be feasible and reliable.

A TDEM measurement performed in 1995 in Strip 9, found the toe of the interface in sub aquifer B at a distance of 900 m from the coast. At a distance of 600 m, the depth to the interface was estimated by TDEM at -60 m b.s.l. A smaller intrusion is envisaged in sub aquifer C, where, in 1990, the hydraulic head was found to be 0.5 m higher than in sub aquifer B.

In Strip 10, the intrusion seems to be similar. In 1974, an observation well near the coast hit the interface at a depth of -28 m b.s.l. An extrapolation resulted in an estimate of 800 - 900 m intrusions. In 1995, a TDEM survey indicated the intrusion to be at 900 m, *i.e.* a rather small variation in the past 20 years.

An unpublished report of the Israel Hydrological Service shows seawater intrusions of: 800, 600, 600, 800 m respectively in strips 8,9,10 and 11.

There is an ongoing study in the area by the Geological Survey of Israel, which will be reported in the near future.

3.8. Available data

The following data are available for the study at present:

a) Annual Summaries for about three decades by report cells of: Total Pumping, Total Artificial recharge, End-year mean water table, Mean chlorides, Mean Nitrates
b) A complete record for some decades by wells of chlorides and nitrates
c) Estimates of Hydraulic data by strips and cells according to a previous two-dimensional flow simulation model, which is being updated at present

These include:

- Transmissivity
- Storativity
- Rain replenishment portion
- Related Rain station
- Return-flow portion from irrigation
- Sea-water intrusion and hydraulic conductivity at the sea-water interface
- Depth of aquifer bottom below sea level
- Top soil level
- Initial groundwater level above sea
- Typical artificial recharge-winter
- Typical artificial recharge-summer
- Typical pumping-winter
- Typical pumping-summer

3.9. Beach wells

Beach wells using water for cooling installations and for swimming pools were installed in Israel in the early 60’s. Data on these wells were collected from the Water Commission, Hydrological Service, TAHAL’s archive and site visits and interviews with operators. These are summarized in Section 9.1 of the Main Report.

The typical dimensions of such well are: 90 m total depth with a 20 m - 25.4 cm (10") screen. These wells are located at a typical distance of 20 m from the coastline. The discharge rate is approximately 4,000 m^3 per day and seawater is mixed with approximately 10% of fresh aquifer water. Detailed data collected for these wells are shown in the main report.

APPENDIX - G

BEACH DRAINS MANAGEMENT SYSTEM – EXPERIENCE GAINED

Contents

The attached table prepared by BMS Denmark, A/S is an overview of existing BMS projects installed since 1981, in Denmark, England, Japan, France, Spain, Sweden and U.S.A. It also shows their main design parameters. The range of these parameters as reported by BMS is as follows:

Equilibrium discharge rate: 0.5 - 2.0 $m^3.h^{-1}.m$ drain (for hydraulic conductivity varying between 2 – 10 x 10^{-4} $m.sec^{-1}$).

Length of drain:	200 - 800 m
Total capacity:	100 - 1,400 $m^3.h^{-1}$
Distance from mean shoreline:	5 – 10 m
Invert elevation:	-1 - 2.5 m (a.m.s.l.)
Drain diameter:	113 - 450 mm.
Drain material:	- Epoxy cemented filter sand around perforated PVC pipe. Flexible corrugated slotted pipe with filter sand and geotextile cover. - Flexible perforated corrugated PE pipe with geotextile stocking, in a bed of fine sand.
Typical Costs:	For a facility of 300 $m^3.h^{-1}$ with a length of 300 m, construction cost including material, pumps *etc.*: US$85,000. Investigation, evaluation, design, supervision and royalty: US$90,000. Operation and maintenance: US$3,000 per year.
Reduction of salinity:	Up to 40% as a result of inflow of fresh water from the inland aquifer.

These values will be used in the present study for the hydrodynamic and economic comparative evaluation of this method.

The construction of a subsurface coastal drain is shown in Figure G.1.

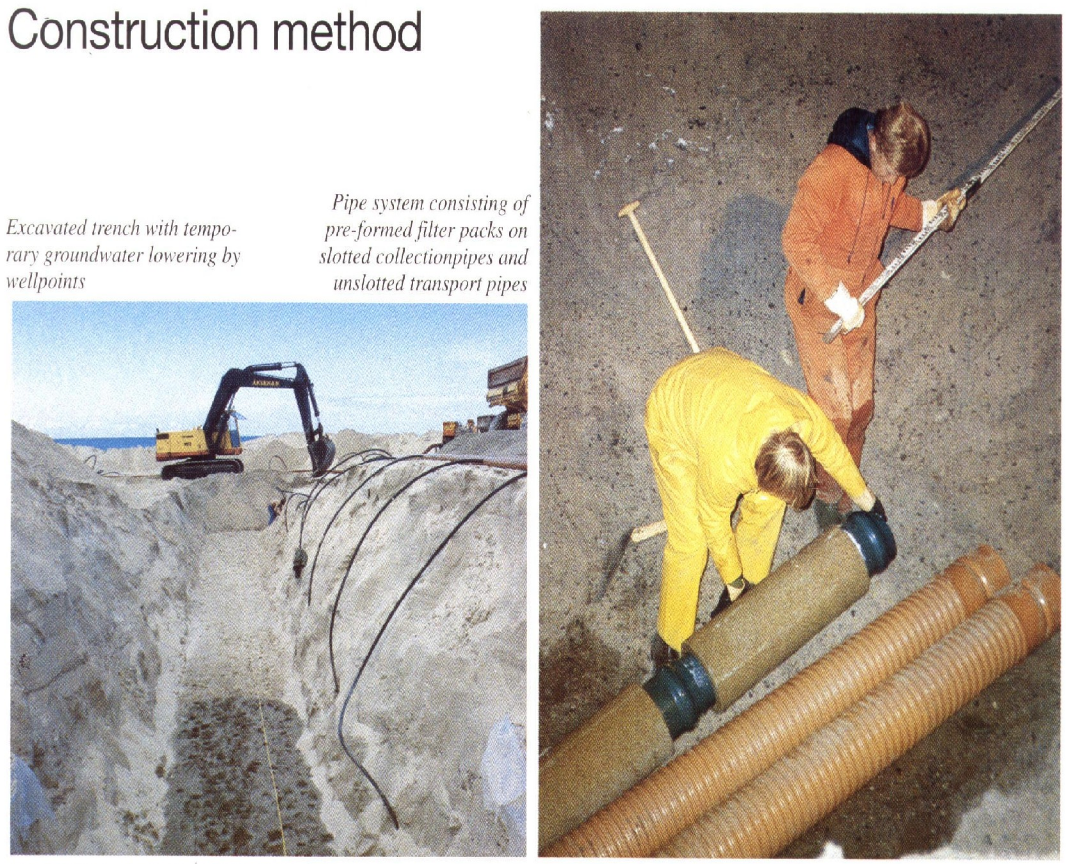


Figure G.1: Construction of coastal drains

Overview of BMS projects 2000-01-10

Project	Year installed	Period of operation	Length of System	Drain material	Drain diameter & Invert El. (MSL)	Installation method	Tide range
Hirtshals W, Denmark	1981	Since 9/81	200 m 656’	1	315 mm 185/200 –2.5 m	Backhoe W.pts.	–1.5 m
Hirtshals E, Denmark	1983	8 months 1983	200 m 656’	1	200 m –2.0 m	Backhoe W.pts.	–1.0 m
Thorsminde, Denmark	1985	1/85 – 4/91	500 m 01640’	1	200 mm –2.0/–2.5	Backhoe W.pts.	1.5 m
Sailfish Point Stuart, FL, USA	1988	7/88 – 8/96	177 m 580’	3	50/450 mm –2.4 m	Backhoe W.pts.	0.8 m
Enoe Strand, Denmark	1994	Since 7/94	600 m 1969’	5	113 mm –1.8 m	Plough	0.5–1.0 m
Towan Bay, UK	1994	Since 9/94	180 m 591’	6	300 mm	Backhoe W.pts.	7 m
Codfish Park, Nantucket I, MA, USA	1994	Since 1/95	357 m 1170’	7	300 mm –2.1 m	Trench machine	1.0–1.5 m
Lighthouse S, Nantucket I, NA, USA	1994	Since 1/95	309 m 1170’	7	300 mm –2.1 m	Trench machine	1.0–1.5 m
Lighthouse N, Nantucket I, NA, USA	1994	Since 1/95	405 m 1330’	7	300 mm –2.1 m	Trench machine	1.0–1.5 m
Chigasaki-Naka Beach, Japan	1996	5/96 – 9/96; 7/97 –	180 m 600’	10	300 mm –2.3 m	Trench machine Sheet wall	1.6 m
Riumar I, Ebro Delta, Spain	1996	Since 10/96	300 m 985’	11	160 mm –2.3 m	Backhoe W.pts.	0.2–0.4 m

BMS project (continued)

Hornbaek W, Denmark	1996	Since 12/96	450 m 1410’	11	160 mm –0.8 m	Plough	0.2–0.4 m
Hornbaek E, Denmark	1996	Since 12/96	530 m 1650’	11	160 mm –1.5 m	Plough	0.2–0.4 m
Ystad, Sweden	1998	Since 3/98	200 m 656’	11	160 mm –1.5 m	Plough	–1.0 m
Hitotsumatsu Beach, Japan	1998	Since 6/98	800 m 2490’	11	300 mm -2.4 m	Trench machine	–2.0 m
Les Sables d'Olonne, France	1999	Since 4/99	300 m 985’	11	160-215 -280- mm -1.2 m	Trench machine	3.4 m mean
Riumar Il, Ebro Delta, Spain	1999	Since 12/99	300 m 985’	11	160 mm –2.0 m	Backhoe W.pts.	0.2–0.4 m

BMS project (continued)

Initial/final Beach slope		Pump arrangement	d_{so}/U Sand grain size	Pump capacity installed	Flow rate $m^3.h^{-1}.m^{-1}$		Approximate draw down C drain	Littoral conditions	Comments
Initial	Final				Initial	Final			
1:20	1:20	[2]	.026/1.7	400 $m^3.h^{-1}$	2.0	1.0			25,000 m^3 sand harvested each year to renourish other beaches
1:25	1:20	[2]	.02/1.3	100 $m^3.h^{-1}$	0.4	0.15			Width maintained. Erosion rate: 7 m per year
1:25	1:30	[2]	0.35/1.7	700 $m^3.h^{-1}$	1.7	1.1			Experimental system, width increased 25 m
1:25	1:15	[2]	0.3/3	340 $m^3.h^{-1}$	1.5	0.60	0.8 m	4)	Width increased 20–25 m during operation
1:15		[2]	0.25/2.3	300 $m^3.h^{-1}$	0.4	0.1	1.0 m		Width increased 3 m August 1996. Maintained
1:45		[2]	0.2/1.7	200 $m^3.h^{-1}$	1.27	1.0			General accretionary trend. Exposed seawall footing safeguarded
1:45		Low vac. wet well (3–5'HG)	1.5/4.2	700 $m^3.h^{-1}$	1.7		0.3 m	8)	Decreases in shoreline width due to storm events. Shoreline erosion rate in the treated areas has been reduced compared to untreated areas
1:6		[2]	0.8/3.2	1400 $m^3.h^{-1}$	1.8		0.3–1.3 m	8)	
1:6		[2]	0.4/3.7	1400 $m^3.h^{-1}$	3.2		0.9–1.8 m	9)	
1:10		[2]	0.5/4	500 $m^3.h^{-1}$	2.8				Temporary shut down due to damage Repaired and reactivated. Shoreline stabilized. Beach level increased.

BMS project (continued)

1:20		[2]	0.2/1.4	290 $m^3.h^{-1}$	0.5		1.0		Width maintained after severe storm event Oct. 97
1:10		[2]	0.3/2	170 $m^3.h^{-1}$	0.1		0.5		Width increased 0–5 m, May 1997
1:20		[2]	0.3/2	325 $m^3.h^{-1}$	0.3		1.0		
1:15		[2]	0.3/3	240 $m^3.h^{-1}$	0.8		1.0		Accretionary trend on the lea side of the 90 metres long groyne
1:20		[2] 2 separate wet wells	0.25/2	2 × 300 $m^3.h^{-1}$					Accretionary trend. Beach level increased. 200 metres foreshore treated by 4 drain structures in parallel
1:70		[2]	0.25/3	250 $m^3.h^{-1}$					Accretionary trend and substantial foreshore dry up in the drain zone.
1:20		[2]	0.25/1.6	400 $m^3.h^{-1}$					No measurements at present

[1] Epoxy cemented filter sand around PVC perforated pipe
[2] Gravity wet well with pressure discharge pipe
[3] Horizontal well points with epoxy cemented sand filter attached to PVC pipe
[4] Inlet/mole south of system can add 1 knot to littoral current
[5] Flexible perforated corrugated pipe with filter sand and geotextile cover (at bottom side)
[6] Perforated PVC pipe with gravel wrapped in geotextile
[7] Flexible PE perforated corrugated pipe with geotextile stocking
[8] Tide induced littoral current less severe to 3 knots max.
[9] Tide induced littoral current less severe to 1–2 knots max.
[10] Flexible perforated corrugated pipe with filter gravel 90 m and without filter gravel 90 m
[11] Flexible perforated corrugated pipe with geotextile stocking and filter gravel

APPENDIX - H

PRACTICAL CONSIDERATIONS IN THE DESIGN OF BEACH WELL STRUCTURE AND COMPONENTS – THE GREEK EXPERIENCE

I. Zachrias

Contents

The usage of Beach wells in desalination plants is today, a widespread adopted technique for acquiring seawater due to the significant advantages incorporated. The most important amongst them is the natural filtration, which significantly reduces the operation and maintenance costs of the plant and minimizes the potential risk of biofouling of the system (Edlinger, Gomila, 1995). However, in order to achieve the maximum benefits from the advantages of this technique, special attention should be paid to the site-specific characteristics and in the design and construction phases of the plant (Heyden, 1985).

First, the water demand that is to be covered from the plant should be taken into account to calculate the necessary diameter of the Beach well and the capacity of the pump in case of a single well. For a multi-well system, the demand will determine the number of wells. Then the hydrogeology of the area should be examined in order to decide the boreholes' size and structure. Particularly, if the wells are constructed in solid rock formations (calcareous or metamorphic), no geomaterial or long casing may be required. The fact that there are not many fine particles encountered in this type of rock that can cause problems in the pumping equipment and in the quality of the acquired water results in avoiding placing of special geomaterial filters around the borehole. Additionally, the casing of the well extends only to the depth of the submersible pump in order to protect it.

Nevertheless, a typical structure of a Beach well in an unconfined aquifer near the coastline incorporates several important components that contribute to its proper operation. Very often, the constructed Beach well penetrates an upper permeable layer of sediment such as sand and gravel and ends at a lower impermeable layer such as silt, clay, metamorphic rocks, *etc*. In such a case, there is a need for a casing, which is usually made of PVC or marine bronze to avoid corrosion (Figure 1), (Heyden, 1985). The casing is slotted with openings, which vary in size according to the surrounding geologic formation to avoid blocking them. The pumped water flows through these slots and rises to the surface. Further, the casing's length varies according to the thickness of the aquifer, the water demand, and the water level fall. In particular, there is a commonly used equation for calculating the casing's length, which takes into consideration all the aforementioned parameters:

$$L = \frac{Q}{FU} \qquad \text{(Lekkas, 1984)}$$

where, L is the length of the casing, Q is the pumping discharge, F represents the area of slots and U is the permissible velocity of the flowing water into the borehole.

Moreover, the thickness of the casing should be at least 6 mm in order to provide a high bearing capacity, which will preserve its condition in the long term. Regarding the openings, these should be small enough to detain 90 – 100% of the material that surrounds the casing. The common practice is to use openings of 0.5 to 5 mm and no smaller because then there are increased hydraulic friction losses and a high risk of blockage which is a serious problem in the operation of the plant (Lekkas, 1984).

Outside the casing there are geomaterials intended to protect the equipment and preserve water quality. The geomaterials, filter the water incoming into the borehole by detaining the fine particles, protect the casing from mass sliding, and allows the water to flow towards the pumping equipment. These geomaterials comprise gravel of various sizes, preferably rounded and anti-erosive. Thus, siliceous or quartz gravels are usually chosen which are resistant to erosion and do not deteriorate the water quality. The size of the gravels that should be used in each case depends on the type of the surrounding formation and it should fulfil several important requirements such as:

- The porosity of the geomaterial should not be significantly higher than the finer material of the surrounding formation to avoid infiltration of particles that may endanger the pumping equipment
- The size of the geomaterial's particles should not be smaller than the casing openings (especially of those in the first geomaterial layer, right outside the casing), in order to avoid their intrusion into the borehole
- If a borehole has penetrated soil layers with different particle sizes and consequently different permeability, then different types of geomaterial should be used for each soil layer (Lekkas, 1984)

Usually, the first layer of geomaterial that is placed just outside the casing comprises medium to coarse gravel (2 - 3 mm), in order to avoid the intrusion of particles into the borehole through the casing's slots, as previously mentioned, while the outer layer comprises fine to medium gravel (0,7 - 1,2 mm) so as to achieve higher filtration capacity (Figure 1).

If the borehole has penetrated solid rock formations, then the usage of geomaterials may not be necessary and the implementation of just a simple casing to protect the pumping equipment may be enough.

The ceiling of the borehole usually comprises coarse compacted sand in the first few metres (1 - 2 m) and above this layer there is a concrete slab on which the well's head is based (Figure 1). A riser pipe goes through the well's head, into the borehole's casing and reaches a submersible pump at the bottom of the borehole. In this pipe, the pumped water rises towards the surface where it is discharged into the network of pipes that directs it to the next stages of the desalination process.

All the pumping equipment is made of marine bronze to avoid corrosion and it has a capacity that covers the demand. The discharge rate of pumping is also determined to avoid saltwater intrusion problems into the aquifer (Ergil, 1999). Additionally, the flowing velocity of the pumped water should not exceed a critical value (usually >0.03 m.sec^{-1}) where the friction powers become great, resulting in increased energy consumption and blockage danger for the casings' openings.

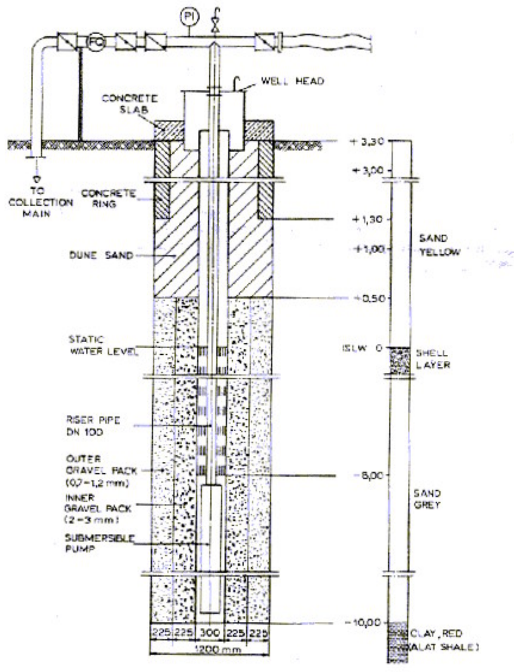


Figure H.1: Beach well structure in a desalination plant

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