INVESTIGATION OF MARINE COMPONENTS OF LARGE DIRECT SEAWATER INTAKE & BRINE DISCHARGE SYSTEMS FOR DESALINATION PLANTS, TOWARDS DEVELOPMENT OF A GENERAL DESIGN APPROACH

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Declaration:

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the owner of the copyright thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

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ABSTRACT

This investigation focused on the marine components of large direct seawater intake and brine discharge systems for seawater desalination plants, with the main aim to provide an overall design approach for these components.

Due to its complexity, an overall and systematic design approach, addressing all the components (feedwater requirements, plant technology, marine structures and environmental issues) is required to ensure an optimum design. A literature review was done on the various desalination technologies, the main components of a seawater desalination plant, as well as the physical, hydraulic, operational and environmental issues regarding seawater extraction facilities, marine pipelines and discharge structures (diffuser).

In order to obtain practical input to the development of an overall design approach, information regarding the marine structures of ten of the largest existing seawater desalination plants throughout the world were obtained and compared with each other and the available technologies.

By way of example, the recently designed marine components of a new seawater reverse osmosis desalination plant in Namibia were reviewed and, as part of this thesis, alternative conceptual concepts which will include two additional components (sump and brine reservoir) were designed. The alternative design was compared with the actual design in order to determine the feasibility of the alternative in terms of operation and cost and subsequently provide input to the overall design recommendations.

Furthermore, from the literature review it seems that there are still significant uncertainties regarding the required performance of a brine (dense) outfall and this required more attention in terms of environmental and hydraulic performance. Based on the Namibian plant, the diffuser configuration was analysed in terms of its hydraulic and environmental performance and subsequently some general guidance with specific respect to a brine diffuser was developed, which in turn formed part of the overall design approach for the marine components.

Finally, the design approach for seawater intake structures, brine outfalls and the connecting marine pipelines is provided in the form of flow diagrams.

SAMEVATTING

Hierdie ondersoek handel oor die mariene komponente van groot en direkte toevoer van seewater en die sout-uitvloeisisteme van ontsoutingsaanlegte van seewater. Die doel is om ‘n oorsigtelike ontwerpbenadering vir hierdie component te verskaf.

As gevolg van die kompleksiteit, is ‘n oorsigtelike en sistimatiese benadering, wat al die komponente (vereistes vir toevoerwater, tehnologie by die aanleg, mariene omstandighede en omgewingsfaktore) in ag neem noodsaaklik om die beste ontwerp te verseker.

‘n Literêre oorsig is gedoen ten opsigte van die tegnologie van verskeie ontsoutingsmetodes, die hoofkomponente van ‘n seewater-ontsoutingsaanleg, asook die fisiese, hidrouliese, operasionele en omgewingskwessies rakende die fasiliteite om die seewater te onttrek, die mariene pyplyne en die strukture vir die afvloei.

Ten einde die optimum ontwerp te ontwikkel, is inligting oor die tegnologie en strukture van tien van die grootste bestaande onsoutingsaanlegte in die wêreld bekom, bestudeer en vergelyk Hulle is met mekaar vergelyk, asook met beskikbare tegnologie.

As ‘n voorbeeld is die nuut ontwerpte mariene komponente van die nuwe ontsoutingsaanleg in Namibië, waar ontsouting d.m.v. omgekeerde osmose gedoen word ondersoek en as deel van hierdie tesis, is ‘n alternatiewe konsep, wat twee bykomende komponente – ‘n opvangput en reservoir vir die afloop – ontwerp. Hierdie alternatiewe ontwerp is met die werklike aanleg vergelyk om die uitvoerbaarheid van die onderneming en die koste daaraan verbonde te toets. Dit is gebruik as aanbeveling vir die oorhoofse ontwerp.

Uit die literêre oorsig blyk dit dar daar nog groot onsekerheid is oor die vereistes van die (digte) waterafloop en dat meer aandag aan die omgewings- en hidrouliese aspekte gegee moet word. Met die Namibiese aanleg as voorbeeld, is die struktuur van die spreiers t.o.v. hidrouliese werkverrigting en die omgewing ontleed. Voortspruitend daaruit is algemene riglyne vir ‘n spesifieke spreier vir afloopwater ontwikkel, wat op sy beurt weer deel vorm van die oorhoofse ontwerp vir mariene komponente.

Laastens is die ontwerp vir die strukture vir seewater-invloei, die afloopwater en die mariene verbindingspyplyne as vloeidigramme aangetoon.

ACKNOWLEDGEMENTS

I wish to extend my thanks to my supervisors; Mr DE Bosman and Mr G Toms, who aided me in this study, providing a lot of guidance and assistance. Furthermore, thanks to my employer WSP Africa Coastal Engineers, making available the funds and resources to complete this thesis. I am also very grateful to Mr WAM Botes who taught me the importance of protecting our marine environment and providing me with the tools to do so. Finally, Areva Resources Namibia who made design information available of their project in Namibia, Trekkopje as well as Steve Christie, senior engineer at the Perth desalination plant who provided detailed information on the operational procedures of the plant.

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GLOSSARY

µg/L (micrograms per litre)	Micrograms per litre; a measurement describing the amount of a substance (such as a mineral, chemical or contaminant) in a litre of water. It is expressed in terms of weight per volume. One µg/L is equal to one part per billion
Beneficial use area	Desired uses of the marine and estuarine areas
Biocide	A chemical (e.g. chlorine) used to kill biological organisms
Brine	Water that contains a high concentration of salt. Brine discharges from desalination plants may also include constituents used in pre-treatment processes, in addition to the high salt concentration seawater
Bromide	An element that is present in desalinated seawater
Coagulation	A pre-treatment process used in some desalination plants. A substance (e.g., ferric chloride) is added to a solution to cause certain elements to thicken into a coherent mass, so that they may be removed
Coastal area	The part of the land affected by its proximity to the sea, and that part of the sea affected by its proximity to the land.
Cogeneration	A power plant that is designed to conserve energy by using "waste heat" from generating electricity for another purpose
Concentrate	Water that contains a high concentration of salt. Concentrate discharges from desalination plants may include constituents used in pre-treatment processes, in addition to the high salt concentration seawater
Conventional treatment	A method of treating water, which consists of mixing, coagulation-flocculation, sedimentation, filtration, and disinfection. Similar to direct filtration with the addition of flocculation and sedimentation
Deaeration	Removal of oxygen. A pre-treatment process in desalination plants to reduce corrosion
Desalination	Desalination is the process of removing dissolved salt and other minerals from seawater to create freshwater
Diffuser	The offshore end (part) of an outfall, consisting of a main pipe (with or without tapers) with discharge ports at specific distances apart, designed to provide an even distribution of port flows along the diffuser.
Dilution	The lessening in concentration of a substance due to mixing with water
Direct seawater intake	Open water intake extraction water directly from the sea
Discharge	A return stream from the desalination plant that is released back into the environment through dilution and mixing
Disinfection	Water treatment which destroys potentially harmful bacteria
Distillation	A process of desalination where the intake water is heated to produce steam. The steam is then condensed to produce product water with low salt concentration
Ecosystem	A community of plants, animals and organisms interacting with each other and with the non-living (physical and chemical) components of their environment
Eddies	The movement of a stream of water in which the current doubles back on itself causing a type of 'whirlpool'. This is typically caused by promontories along a coastline or due to counteractions from driving forces such as wind shear and an ambient current

Electrodialysis	Most of the impurities in water are present in an ionized (electrically-charged) state. When an electric current is applied, the impurities migrate towards the positive and negative electrodes. The intermediate area becomes depleted of impurities and discharges a purified stream of product water. This technology is used for brackish waters but is not currently available for desalting seawater on a commercial scale
Environmental impact	A positive or negative environmental change caused by human action
EPA	United States Environmental Protection Agency
Estuary	A partially or fully enclosed body of water which is open to the sea permanently or periodically, and within which the seawater can be diluted, to an extent that is measurable, with freshwater drained from land or a river. The upstream boundary of an estuary is the extent of tidal influence.
Euphotic zone	The euphotic zone is the depth of water body in an ocean that is exposed to sufficient sunlight for photosynthesis to occur.
Far-field dilution	When an effluent plume is transported away from the initial mixing zone, dispersion, entrainment and mixing with sea water is brought about by currents, turbulence, eddies and shears, a process generally referred to as secondary dilution which together with the chemical/biological 'dispersion' of non-conservative substances and the decay of certain organisms, can be described as the "far-field dilution process
Feedwater	Water fed to the desalination equipment. This can be source water with or without pre-treatment
Filtration	A process that separates small particles from water by using a porous barrier to trap the particles and allowing the water through
GRP	Glass Reinforced Polyester/Plastic
HDPE	High Density Polyethylene
Head loss	The drop in the sum of pressure head, velocity head, and potential head between two points along a path
Hydraulic grade line	The height to which the water would rise in a piezometer tube attached vertically to the water conveyance pipeline
Indirect seawater intake	Intake water filtered through seabed (e.g. via beach wells)
Infiltration Gallery	A method used for seawater intake. Perforated pipes are arranged in a radial pattern in the sand onshore below the water level. Water in the saturated sand enters the perforated pipes
Initial dilutions for dense plume	The dilution of the wastewater plume generated by jet momentum and the negative buoyancy effect that occur which causes the plume to descend on the seabed
Initial dilutions for buoyant plume	The dilution of the wastewater plume generated by jet momentum and the positive buoyancy effects that occur between the outlet ports of a marine outfall's diffuser and the sea surface
Intake	The physical facilities through which the seawater enters the plant
Marine discharge	Discharging wastewater to the marine environment either to an estuary or the surf zone or through a marine outfall (i.e. to the offshore marine environment)
Marine Environment	Marine environment includes estuaries, coastal marine and near-shore zones, and open-ocean-deep-sea regions.
Marine outfall pipeline	A submarine pipeline originating onshore, which conveys wastewater from a head works to a submerged discharge location on or near the seabed beyond the surf zone (i.e. to the offshore marine environment). Also referred to in the literature as a long sea outfall/pipeline and ocean outfall/pipeline.

Mean sea level	The average elevation of the sea surface for all stages of the tides over a long period
Membrane desalination	Use of membranes to remove salts from seawater
Meteorological conditions	The prevailing environmental conditions as they influence the prediction of weather
mg/L	Milligrams per litre; a measurement describing the amount of a substance (such as a mineral, chemical or contaminant) in a litre of water. One milligram per litre is equal to one part per million
Micro-filtration	A method of water filtration, using a pressure-driven membrane process, which includes particle filters that reject particles larger than 1.0 micron in size. Provides a less refined effluent than ultra-filtration
Micro-layer	The upper few millimetres of the ocean. Fish eggs are sometimes concentrated in the micro-layer
Mitigation	The process of preventing damage or repairing an area after construction or creating environmental improvements, (sometimes in a different location)
Multi-effect Distillation (MED)	A form of distillation. Evaporators are in series, and vapour from one series is used to evaporate water in the next one. This technology has several forms, one of the most common of which is the Vertical Tube Evaporator (VTE)
Multi-stage Flash Distillation (MSF)	A form of distillation. The intake water is pressurized and heated. It is discharged into a chamber maintained slightly below the saturation vapour pressure of the water, and a fraction of the water content flashes into steam. The steam condenses on the exterior surface of heat transfer tubing and becomes product water. The unflashed brine enters a second chamber, where brine flashes to steam at a lower temperature. Each evaporation and condensation series is called a stage
MWQG	Marine Water Quality Guidelines
Nearshore discharge	Diluting and mixing the concentrate with a large flow of water and returning it to the near shore area
Ocean Thermal Energy Conversion (OTEC)	A solar, ocean thermal desalination approach where electricity is produced by using the temperature differential between cold, deep waters and warm, shallow surface waters. Water at the ocean surface (at about 70°F) is used to heat liquid ammonia, which vaporizes at this temperature in a vacuum chamber. The ammonia vapor is used to turn a turbine to produce electricity. The vapour is then condensed by using cold water pumped up from the ocean depths (at about 35°F)
Offshore	Within the context of ocean outfalls, this is the zone in the sea in which wave action has an insignificant effect on water circulation and shoreline processes (erosion and accretion)
Offshore discharge	A discharge to the offshore areas
Pollution	The direct or indirect alteration of the physical, chemical or biological properties of the natural environment, including the marine environment, so as to make it less fit for any beneficial purpose for which it may reasonably be expected to be used, or to make it harmful or potentially harmful to the welfare, health or safety of human beings or to any aquatic or non-aquatic organisms
Potable	Water that does not contain pollution, contamination, objectionable minerals or infective agents and is considered safe for domestic consumption
PP	Polypropylene
Product Water	The desalinated water delivered to the water distribution system

Reverse Osmosis (RO)	A process of desalination where pressure is applied continuously to the feedwater, forcing water molecules through a semi-permeable membrane. Water that passes through the membrane leaves the unit as product water; most of the dissolved impurities remain behind and are discharged in a waste stream
Rhodamine-B dye	A fluorescent red basic xanthene dye used in the marine environment to determine transport and dispersion patterns
Saline water	Water that contains a significant concentration of dissolved salts (NaCl)
Salinity	Generally, the concentration of mineral salts dissolved in water. Salinity may be measured by weight (total dissolved solids - TDS), electrical conductivity, or osmotic pressure. Where seawater is known to be the major source of salt, salinity is often used to refer to the concentration of chlorides in the water.
SDI	Swartz's Dominance Index: used to evaluate benthic community assemblages and defined as the minimum number of species comprising 75% of the total abundance in a given sample
Secondary dilutions	The further dilution that occurs after initial dilution when a wastewater plume is transported away from the discharge area
Stagnant stratified conditions	The absence of currents and with stratification of the seawater (density gradient between the surface and the bottom)
Stagnant un-stratified conditions	The absence of currents and with no stratification of the seawater
Stratification	When denser seawater underlies lighter sea water causing a vertical density gradient in the water column, depending on the vertical temperature gradient between warmer upper water layers and colder deeper water layers and the salinity gradient
Surf zone	Also referred to as the 'breaker zone' where water depths are such that the incoming waves collapse and breakers are formed
Suspended solids (SS)	The term "Suspended solids" refers to small solid particles which remain in suspension in water as a colloid or due to the motion of the water. It is used as one indicator of water quality. It is sometimes abbreviated SS, but is not to be confused with settleable solids, also sometimes abbreviated SS, which contribute to the blocking of sewer pipes.
Seawater Reverse Osmosis (SWRO)	A process of desalination where pressure is applied continuously to seawater, forcing water molecules through a semi-permeable membrane. Water that passes through the membrane leaves the unit as product water; most of the dissolved impurities remain behind and are discharged in a waste stream
Thermal desalination process	Involves the heating of seawater, generating water vapour, which in turn is then condensed to produce fresh water.
Total Dissolved Solids (TDS)	Total salt and calcium carbonate concentration in a sample of water, usually expressed in milligrams per litre (mg/L) or parts per million (ppm). The state-recommended Maximum Contaminant Level (MCL) drinking water standard for total dissolved solids is 500 mg/L, the upper MCL is 1,000 mg/L, and the short-term permitted level is 1,500 mg/L
Total dynamic head	The summation of hydraulic head (elevation, pressure, and/or friction losses) that a flow of water must overcome to move forward
TSS	Total Suspended Solids
Turbidity	A measure of suspended solids concentration in water
Ultra-filtration (UF)	A membrane filtration process that falls between reverse osmosis (RO) and micro-filtration (MF) in terms of the size of particles removed
Ultraviolet	The use of ultraviolet light for disinfection

treatment (UV)	
Vacuum Freezing (VF)	A process of desalination where the temperature and pressure of the seawater is lowered so that the pure water forms ice crystals. The ice is then washed and melted to produce the product water. This technology is still being developed, and is not yet commercially competitive
Vapour Compression	A form of distillation. A portion of feedwater is evaporated, and the vapour is sent to a compressor. Mechanical or thermal energy is used to compress the vapour, which increases its temperature. The vapour is then condensed to form product water and the released heat is used to evaporate the feedwater

1 INTRODUCTION

During the 20th century, the growth rate of the world's Required Water Use was twice the growth rate of its Population. Since it is estimated that the world's population will double within the next 50 years, the demand for fresh water will increase accordingly whereas the natural water resources will become even scarcer. As the demand for fresh water increases, the need to manage existing natural resources more effectively, as well as the need to desalinate seawater also increases. Therefore, it is critical to optimize the design of seawater desalination facilities in order to provide a reliable and consistent source of fresh water, which will also generate economic opportunities without a significant impact on the environment.

Extensive research has been conducted over the years with regards to desalination technologies and subsequently the optimization of desalination plants. However, limited information, specifically with respect to overall guidelines for the design of the marine components for large scale desalination plants could be accessed. Although general information and studies is available for specific components, an overall development guideline would be helpful to ensure all the relevant issues are addressed and the marine structures optimally designed.

Taking the above into account, this study aims to provide overall guidance for designing a seawater intake facility, brine outfall and marine pipelines of large desalination plants. The available literature together with reports on existing facilities was reviewed and subsequently a design approach was developed which can contribute towards specific design guidelines in future for the design of the marine components.

Although a number of the design criteria for seawater intake and brine discharge facilities are very site specific, variations on the intake and discharge systems for a new facility in Namibia were designed and compared with the original design in order to determine the feasibility and subsequently contribute to the overall design approach. In addition, the diffuser configuration in general for a brine effluent was also investigated in order to provide general guidance on optimizing a brine diffuser.

2 OBJECTIVES OF INVESTIGATION

The objective of this investigation is to develop an overall design approach for the design of large scale offshore seawater intake and brine discharge structures for desalination plants from reviewing available literature, existing facilities and assessing the feasibility and optimization of certain components. Therefore, the aim of this thesis is to assist designers with the basic information and guidance to develop the marine components of large seawater desalination plants.

3 METHODOLOGY

The main purpose of this study was to contribute towards the development of local design guidelines for the design for direct seawater intake and brine discharge components of seawater desalination plants. The methodology of the study was as follows:

- Obtain, evaluate and conclude on available literature concerning:
  - The concept of various desalination technologies;
  - The main components of seawater desalination plants;
  - Design components and considerations for a seawater intake structure, brine outfall and marine pipelines; and
  - Physical, hydraulic and environmental design criteria for brine outfalls

- Review a number of existing large scale seawater desalination plants worldwide, focussing on:
  - Desalination process (i.e. technology, different preliminary and primary treatment processes);
  - Configuration of intake structures, marine pipelines and brine discharge and
  - Operational performance

- Review the Trekkopje marine intake and discharge facility under construction in Namibia, especially in terms of the preliminary treatment design and proposed operations as well as the brine discharge operations.

- Propose an alternative concept/design for the preliminary treatment of the intake seawater and the discharge of the brine, based on the conclusions of the literature research in order to provide input towards the overall design concept.

- Analyse the typical brine effluent diffuser configuration and subsequent guidelines and recommendations as part of the overall design approach.

- Finally, contribute towards the development of design guidelines for the marine components of direct seawater intake abstraction facilities and brine outfall structures for seawater desalination plants.

4 LITERATURE STUDY

In order to develop an overall design approach for designing the marine components, available literature of all the individual components and considerations which need to be addressed throughout the entire design phase were obtained and reviewed. These were then summarized and arranged according to the overall design process of large scale seawater intake and brine discharge systems.

4.1 DESALINATION TECHNOLOGIES

It is very useful for marine and coastal engineers to understand the different desalination technologies since the type of technology influences the required feedwater volume and quality as well as the brine effluent constituents and discharge rate.

Desalination refers to the treatment process in which salt and other minerals are removed from brackish groundwater or seawater in order to produce suitable fresh water for industrial or domestic uses. Desalination plant capacity around the world has more than doubled over the last two decades due to the competitive cost of desalinated seawater compared to conventional freshwater resources. Furthermore, development of improved and refined desalination technologies is expected to continue and lower the future cost of seawater desalination even more.

According to the *Magazine of the Scientific Chronicles (2005),* Aristotle observed the principle of distillation in the 4th century. In the 1960's, thermal processes were developed to desalinate seawater and investigations were undertaken on desalination processes utilising reverse osmosis. In 1978, the first seawater desalination plant using reverse osmosis was in operation in Jeddah, Saudi Arabia.

Currently, desalination facilities operate in more than 120 countries worldwide, with the greatest number of desalination facilities in the Middle East, followed by the United States. According to the American Membrane Technology Association *(AMTA, 2007)*, the global capacity from all desalination facilities was almost 10 million m^3/day in 1993 (refer to Figure 4.1). Ten years later, in 2003, the global capacity had nearly doubled, with slightly less than 18 million m^3/day from Middle East plants.

Due to major breakthroughs in membrane technologies in the early nineties, there was a subsequent significant increase in the installation of reverse osmosis desalination plants worldwide as shown in Figure 4.1 *(AMTA, 2007)*. However, although reverse osmosis plants are more numerous, thermal plants (MSF systems) presently account for over 85% of the worlds desalinated water volumes.

A great amount of research and informative literature is available with regards to desalination technologies. The following sections is summarise some of the various investigations which were found to be the most informative, comprising research done by (*BECK, 2002*), Buros (*BUROS, 1982* and *BUROS, 2000*) and Wangnick *(WANGNICK, 2004)*.

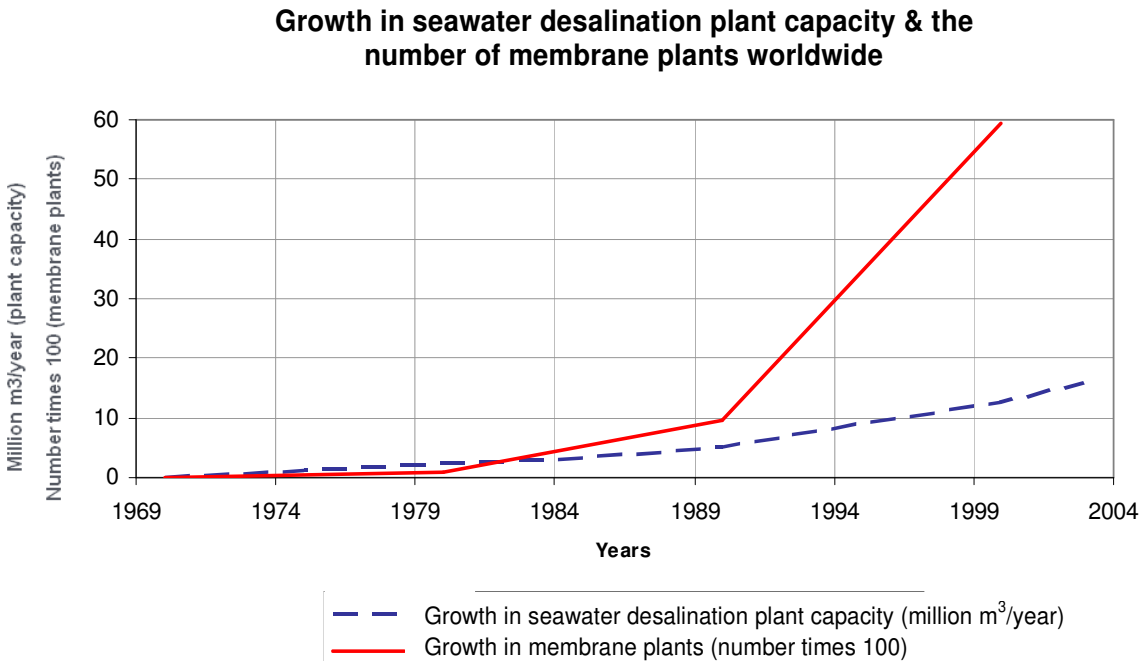


Figure 4.1: Desalination plant capacity and number of membrane desalination facilities worldwide

Currently, the major available desalination processes are thermal (distillation) and membrane technologies as shown in Table 4.1. Several other desalination processes have been developed, but although valuable under special circumstances, these processes (some of which are listed in Table 4.1), have not achieved the level of commercial success that distillation and membrane desalination processes have.

Table 4.1: Available desalting technologies

Major Processes	***Thermal***	• Multi-stage flash distillation (MSF) • Multiple-effect distillation (MED) • Vapour compression (VC)
	Membrane	• Electrodialysis (ED) • Reverse osmosis (RO)
Minor Processes		• Freezing • Membrane distillation • Solar humidification
Hybrid facilities		• Multi-effect distillation with reverse osmosis • Multi-stage flash with reverse osmosis
Co-generation		• Facility that produces both electric power and desalted seawater

4.1.1 Thermal processes

(BUROS, 2000)

Thermal processes involve the heating of seawater to generate water vapour, which in turn is then condensed to produce fresh water. Throughout this processes, a brine waste stream is produced which needs to be discharged. The following three thermal technologies are commercially available:

Multi-Stage Flash Distillation (MSF): Initially, seawater is heated in a vessel, then transferred to another vessel where the ambient pressure is lower, causing the water to immediately boil. Only a small percentage of this water is converted to water vapour, since boiling will continue only until the water cools. MSF processes are generally built in units that produce approximately 4 000 to 57 000 m^3/day fresh water. The inherent efficiency of MSF, combined with the relatively low maintenance demand of these plants makes them particularly suited to the large-scale applications required in the Middle East. Figure 4.2 illustrates the processes of a typical MSL unit.

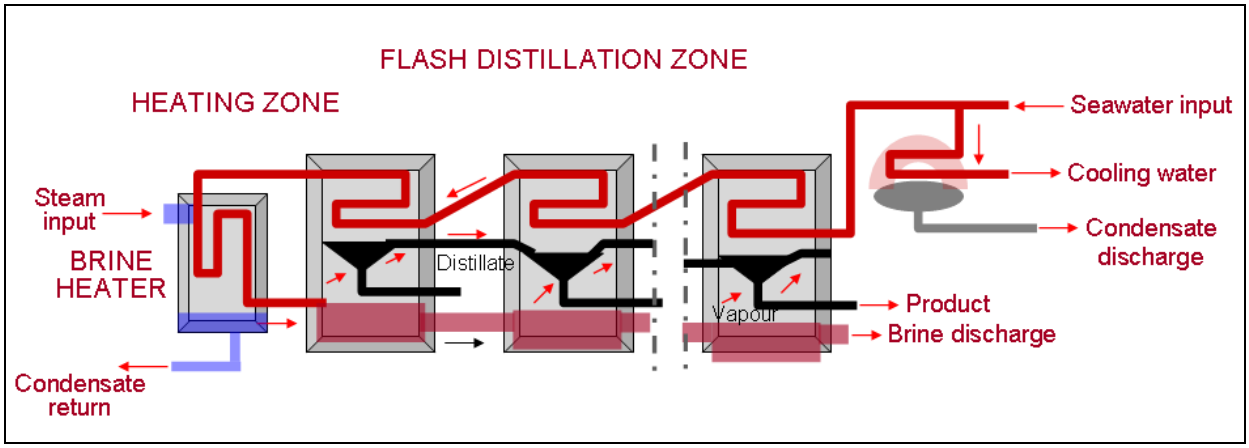


Figure 4.2: Schematic diagram of a typical MSF process

Multi-Effect Distillation (MED): Similar to MSF, MED occurs in a series of vessels, utilising the principles of condensation and evaporation at reduced ambient pressure. The MED process allows seawater to undergo boiling without the need to supply additional heat after the first effect by means of multiple condensation of steam in various vessels. Although the installed capacity of units using the MED process relative to the world's total capacity is still small, its numbers and popularity have been increasing.

Vapour Compression Distillation (VC): The VC distillation process is generally used in combination with other processes (e.g. MED process) or by itself for small and medium size seawater desalination applications. The heat source for evaporating the seawater originates from the compression of vapour, instead of the direct exchange of heat from steam produced in a boiler.

4.1.2 Membrane processes

Membrane technologies make use of membranes to separate the salts from seawater and are used in two commercial desalination processes, namely electrodialysis and reverse osmosis. A membrane is a flat surface with selective permeability which removes salt and other impurities from the seawater.

Electrodialysis (ED): The ED process *(BUROS, 2000)* uses an electrical potential (voltage) to move salts through a membrane and leaving the fresh water behind as product water. The process is schematically illustrated in Figure 4.3.

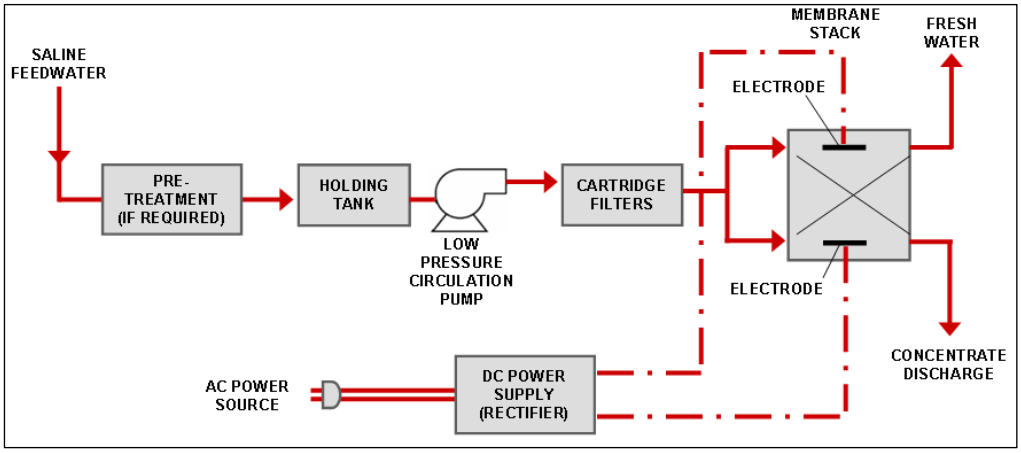


Figure 4.3: Diagram of Electrodialysis process

The raw feed water is pre-treated, preventing materials that could harm the membranes or clog the channels in the cells from entering the membrane stack. Feed water is then circulated through the stack with a low-pressure pump. Afterwards, post-treatment is carried out to stabilise the water and prepare it for distribution.

Reverse Osmosis (RO) - (BUROS, 1982, BUROS, 2000, BECK, 2002): In principle, a high pressure is applied to saline water, forcing it through a membrane as illustrated in Figure 4.4. Freshwater is produced during this process, since only water molecules can pass through the membrane, thus leaving the salts behind.

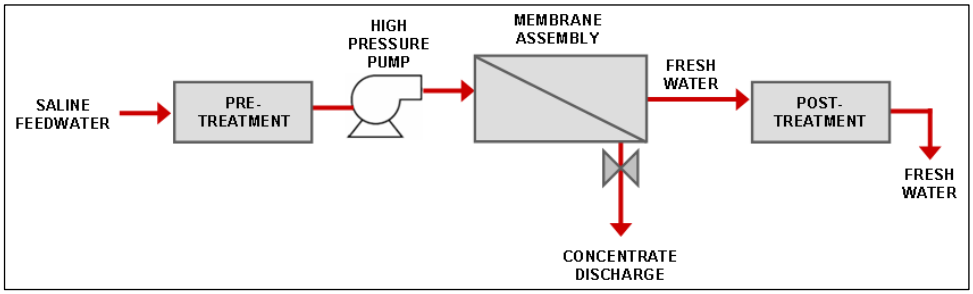


Figure 4.4: Basic components of a reverse osmosis plant (BUROS, 1982)

Initially, the feed water is pre-treated to remove particles such as sand, shells or seaweed to protect the membranes and usually consists of fine filtration and the addition of acid or other chemicals to inhibit precipitation and the growth of micro-organisms. The feed water is then pumped into a closed vessel where it is pressurized against the membrane by means of a high pressure pump. Subsequently, fresh water is generated when a portion of the feedwater passes through the membranes, leaving behind the salts. The remaining feedwater which didn't pass through the membranes increases in salt content and is discharged.

Depending on the salt content of the feed water, pressure and type of membrane, the amount of the brine stream can vary from 20 to 70 percent of the feed flow. Therefore, the quality of the fresh water produced depends on the pressure, the concentration of salts in the feed water and the size of the membranes.

Although the reduced cost of RO systems over the last decade has increased the use of RO for seawater desalination, RO plants use electric energy to create the high pressure pumping and can therefore be expensive to treat seawater with very high salinities such as those in the Middle East. According to a study which was undertaken by Mahmoud Abdel-Jawad (ABDEL-JAWAD, 2001), the electrical and mechanical energy consumption (kWh/m^3) for a reverse osmosis plant could be double than that of a thermal MSF distillation plant. However, in addition to the electrical and mechanical energy requirements, thermal technologies (e.g. MSF plants), also requires thermal energy.

Therefore, comparing membrane technologies with thermal technologies in terms of energy consumption, thermal desalination technology would only be more feasible when integrated with a power plant which could provide the desalination plant with the required thermal energy. Based on Abdel-Jawad's study, Table 4.2 roughly indicates the energy requirements per cubic metre of product water for the various desalination technologies.

Table 4.2: Energy consumption for various desalination technologies

Desalination technology	MSF	MED	VC	RO	ED
Electrical/mechanical energy consumption (kWh/m^3)	3.7 – 4.2	1.8 – 2.3	1.6 – 1.8	6.1 – 7.5	2.6 – 5.5
Electrical equivalent for thermal energy consumption (kWh/m^3)	11.6 – 12.5	7.9 – 11.4	12.8 - 13	NA	NA
Total equivalent energy consumption (kW/m^3)	15.3 – 16.7	9.7 – 13.7	14.6	6.1 – 7.5	2.6 – 5.5

4.1.3 Hybrid facilities

Hybrid desalination plants are a combination of a distillation plant (MSF or MED) and an RO plant operating in conjunction, where the distillation and the membrane plants together provide the desired desalinated water demand. These systems usually are combined with the intent to obtain some advantage over either process alone.

4.1.4 Co-generation (Co-location)

(BUROS, 2000, BECK, 2002 and WANGNICK, 2004)

Co-generation plants consist of a power plant and a desalination plant and are popular in the Middle East and North Africa. Power is generated and the energy source is then used to desalinate seawater, which is normally done by means of the distillation process. The intake facility serves to provide cooling water for power generation as well as feedwater for the desalination process.

Co-generation systems have great economic benefits, since energy is the largest operating cost in any desalination process and the fuel consumption could be significantly less for dual-purpose plants compared to two separate plants. However, the units are permanently connected and therefore the desalination plant cannot function when the turbine is not operating due to repairs or other reasons. Co-generation plants have normally the following advantages:

- Compared with the use of two separate intake structures, the overall entrainment, impingement and entrapment of marine organisms are reduced if one seawater intake is shared;
- The combined use of the same intake and outfall structures minimizes the disturbance of the ocean floor habitat during construction (therefore, eliminates the need for separate intake structures, intake pipelines and bar screening facilities);
- Construction costs for the intake are reduced, since new surface intake structures can amount to 20% of the construction cost;
- The overall power demand for the desalination facility is reduced;
- The salinity concentration of the effluent is diluted when it is blended with the power plant's cooling water; and
- The temperature of the thermal discharge from the power plant is reduced by the brine discharge.

4.2 MAIN COMPONENTS OF A SEAWATER DESALINATION PLANT

The main components of a reverse osmosis seawater desalination plant, where the feedwater is extracted via a direct, offshore subsurface intake type are schematically illustrated in Figure 4.5 and genrally comprise of the following components:

Intake headworks/structure: The function of the intake headworks is to extract the required amount of feedwater (seawater) for the desalination plant. The type of intake works will depend on the required abstraction rate, site specific physical and environmental conditions as well as construction costs. In the case of a direct intake, the intake headworks is usually a structure on the seabed which accommodates some form of screening.

Intake pipelines (part of extraction system): If feedwater is extracted from an offshore intake structure, the feedwater is transferred via a culvert, tunnel or pipeline from the seawater intake structure (headworks) to land.

Preliminary treatment (part of extraction system): To prevent coarse objects and debris from entering the intake pipelines, bar screens are normally placed at the intake head. Normally it is more effective to remove the finer objects at a sump on land with coarse and fine screens.

Pump station: Seawater is normally pumped to the desalination plant for pre-treatment. The pump station is designed to provide sufficient "head' to pump the required feedwater rate to the desalination plant.

Pre-treatment plant: For a reverse osmosis plant, the seawater first passes through a pre-treatment process. This process removes suspended solids and other matter that may harm the reverse osmosis membranes. This pre-treatment is typically achieved by filtration. The pre-treatment process also includes chemical dosing to prevent scaling and fouling of the reverse osmosis membrane.

Desalination plant: Membranes separate the salt from the seawater.

Post treatment plant: The water is normally disinfected and fluoridised before introduced to the water supply network.

Water transfer pipeline: Transporting the fresh water from the desalination plant facility to the water supply network.

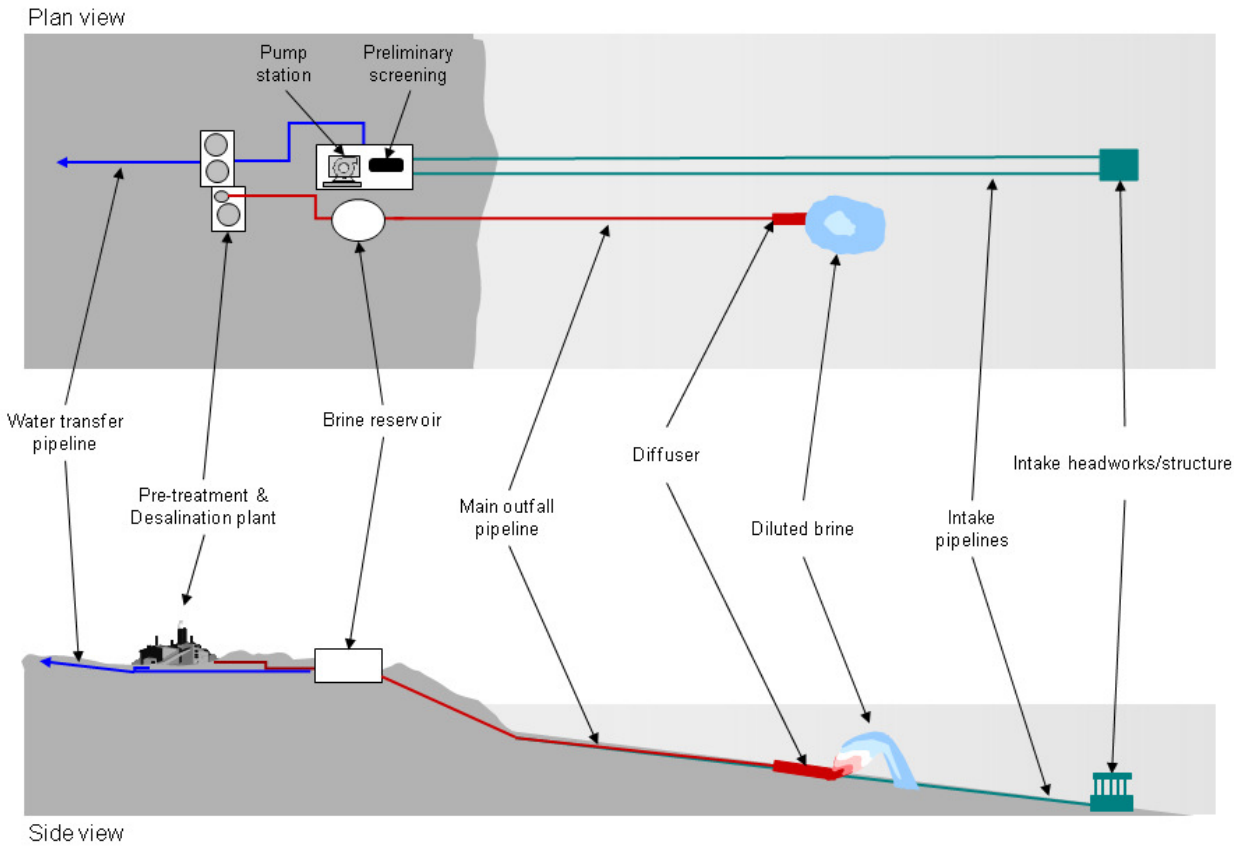


Figure 4.5: A direct intake type with a RO desalination process

Brine reservoir/tank (part of discharge system): A reservoir for the outfall system will normally be required to ensure the brine is discharged intermittently at the design flow rate in order to achieve the required dilutions.

Main outfall pipeline (part of discharge system): Brine from the pump station or reservoir is transported through the main outfall pipe out to the diffuser. The main pipe should be designed to flow full, maintain scouring velocities and minimize unnecessary head loss.

Diffuser (part of discharge system): The diffuser is the seaward section of the outfall through which brine is discharged through either a series of discharge ports along its length, or a single port, depending on the ambient conditions and required dilutions.

4.3 SEAWATER INTAKE SYSTEM

The primary aim of a seawater intake system is to provide the required flow rate of clean seawater to the desalination plant, while keeping the environmental impact to a minimum.

4.3.1 Intake works location selection

The location of the intake works is more site specific than any other component of a desalination facility and can make up for approximately 20% of the capital cost of the entire facility in certain cases.

Although access and proximity to the desalination plant is essential, the environmental requirements, physical characteristics, coastal processes and the ease of constructability should be considered as well. Table 4.3 list the various aspects which should be considered for a desalination plant with specific reference to a direct seawater intake structure *(Desalination Issues Assessment Report, 2003)*:

Table 4.3: Aspects relevant to intake location selection for a direct seawater intake

Beneficial uses of surrounding waters and beach area
Environmental sensitive areas
Recreational areas
Port demarcated areas (i.e. *vessel navigation)*
Industrial use (i.e. *proximity existing industrial ocean outfalls)*
Physical characteristics of the intake
Intake type
Required water depth
Required flow
Water quality
Meteorological conditions
Oceanographic conditions (i.e. seabed slope, bathymetry)
Environmental processes
Waves (i.e. *construction constrains, intake structure stability, turbidity at intake works)*
Currents (i.e. *construction constrains, intake structure stability)*
Sediment processes
Environmental impacts
Pollution
Fouling
Aesthetic considerations (i.e *could change character and appearance of beaches*)
Marine biology (i.e. *Impact on shore and benthic marine organisms in the area of the intake, Entrainment issues (area rich in marine species))*

4.3.2 Feedwater flow rate & quality

The required flow rate of feedwater depends on the essential required use and on the desalination process. For instance, the required seawater flow rate for a reverse osmosis plant is normally twice the plant production capacity. However, the required seawater flow rate could be more than ten times the production capacity for a distillation production of thermal processes, ensuring adequate feedwater for both the desalination process and cooling water requirements. The required capacity of

a desalination plant is also determined by the required freshwater demand, which ranges from domestic uses to mining and chemical industrial processes.

Investigations have to be carried out on the quality of the feedwater in order to determine all constituents (e.g. suspended solids, dissolved solids, temperatures, organic, inorganic, etc.) as well as seasonal and diurnal variations, since the feedwater quality directly affects the pre-treatment process design as well as the overall plant operations. The required feedwater quality will depend on the specific requirements of the desalination plant type and design. However, desalination plants operate most efficiently if the feedwater characteristics remain relatively constant.

If the brine discharge point is within close proximity of the intake, ongoing water quality sampling at the intake location is required to verify that the brine is diluted sufficiently and does not impact the salinity of the feedwater.

Since certain algae and seaweed can be toxic to humans or potentially harmful by decreasing the oxygen levels in the water, a direct subsurface seawater intake point is normally 20 to 25 metres deep *(MISSIMER, 2008)*, depending on site specific conditions, to ensure the feedwater will not be impacted by algae near the water surface and the suspended sediment near the seabed.

4.3.3 Intake Types

Seawater intake types range from screened wells onshore, to large surface water intakes along the shore and to offshore intake structures. Each type varies in design, power consumption and environmental considerations. Intake types are generally divided into two main groups, which are direct intakes and indirect intakes as shown in Figure 4.6.

Selecting the most feasible type of intake structure will mainly depend on the following:

- The required feedwater volume and quality;
- Site-specific coastal and physical processes;
- The seabed bathymetry;
- Geophysical conditions;
- Environmental considerations;
- Cost of construction; and
- Future maintenance costs.

A direct seawater intake refers to an intake structures which extracts seawater directly from the ocean. This type of structure can either be constructed below the surface (sub-surface) at an offshore location, or as an open channel (surface intake) protected from waves by groynes or in a natural bay as illustrated in Figure 4.7.

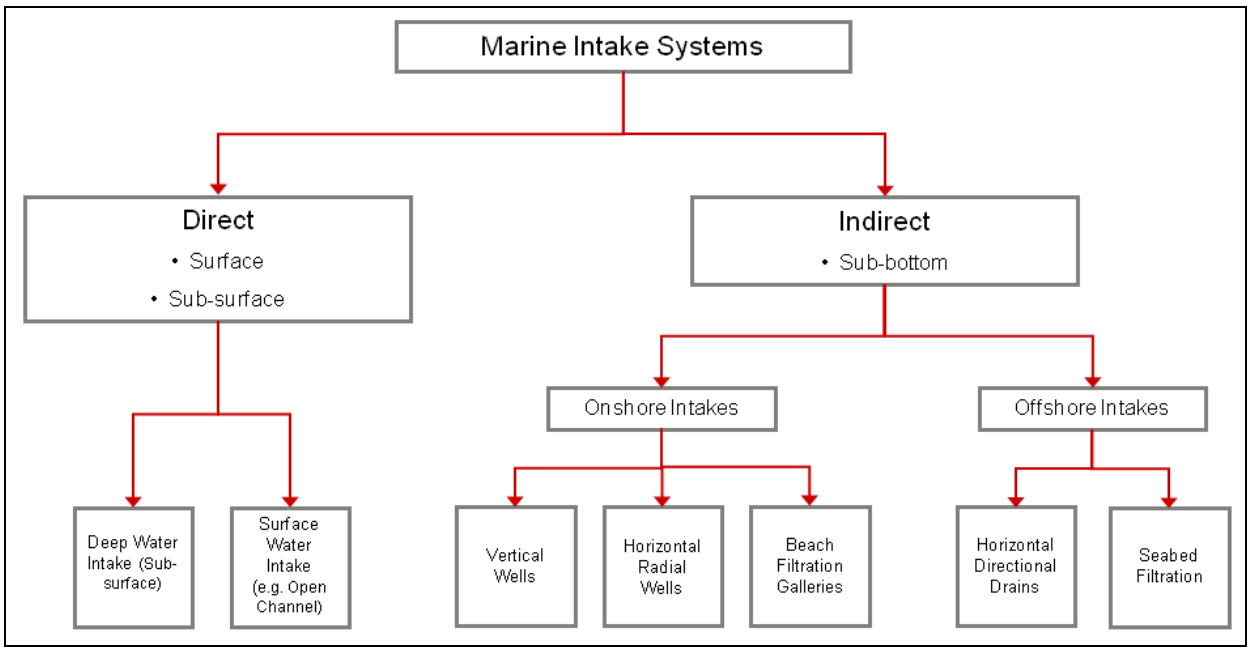


Figure 4.6: Different Marine Intake Types (PANKRATZ, 2008)

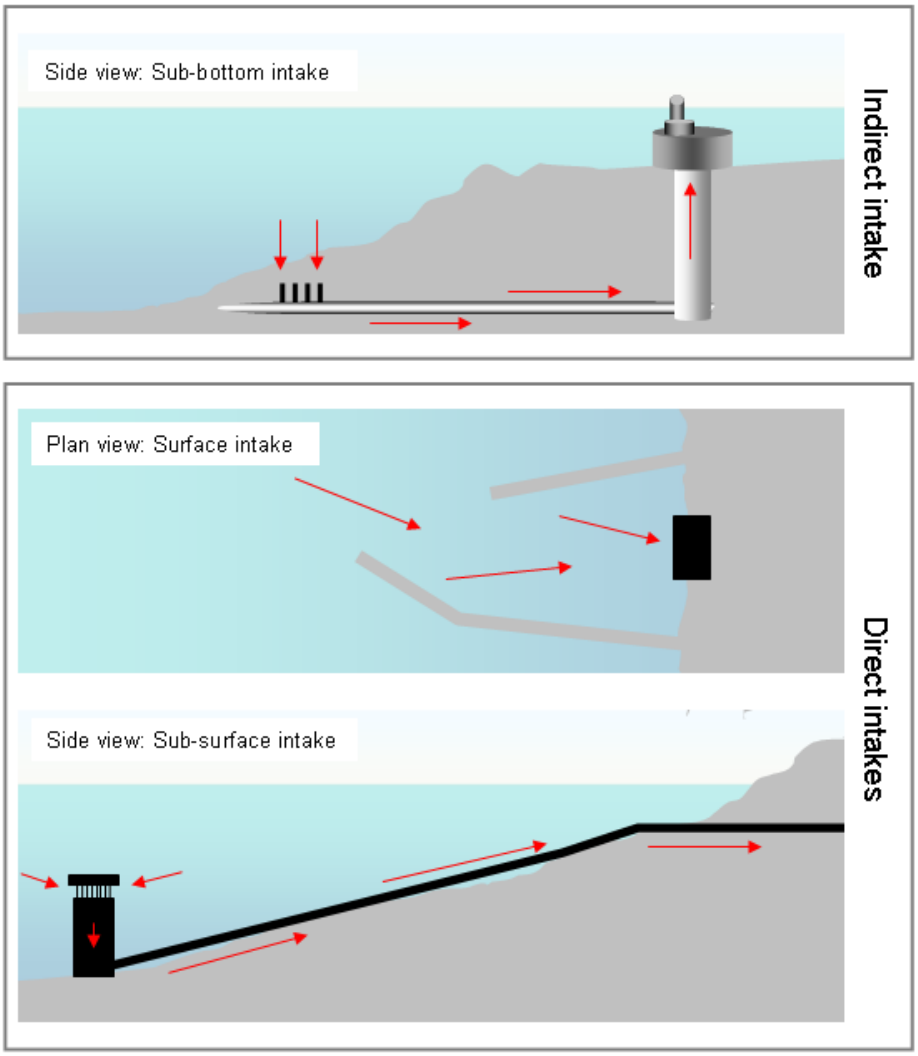


Figure 4.7: Schematic illustration of direct and indirect intake types

Indirect intakes, also referred to as sub-bottom intake structures, extract seawater from a point below the seafloor or beach. Various sub-bottom intake types are available, depending on the local geotechnical conditions and specific project specifications. The indirect intake types include horizontal and vertical wells, beach filtration galleries, horizontal drains and seabed filtration systems. Figure 4.8 illustrates the main three types of seawater intakes.

Table 4.4 lists the main advantages and disadvantages of the different intake types which are discussed in more detail in the following sections:

Table 4.4: Main advantages and disadvantages of different intake types

	Indirect	Direct: Surface	Direct: Sub-surface
Advantages	No preliminary treatment only minor primary treatment of feedwater required After construction, minor impact on coastline Not affected by wave action and turbidity No entrainment or entrapment of marine organisms	Extract large volumes of feedwater Maintenance operations more simple and accessible	Extract larger volumes of feedwater than indirect intakes, but less than direct surface intakes Minor impact on the coastline during construction and none after construction
Disadvantages	Limited feedwater extraction volumes High impact on coastline during construction	Major impact on natural coastline Mitigation measures required against entrainment and entrapment of marine organisms Intake requires protection against waves and turbidity and therefore expensive construction costs (breakwaters) Subject to oil pollution (oil spills)	Mitigation measures required against entrainment and entrapment of marine organisms Marine pipelines require protection (or buried) in surf-zone Expensive maintenance operations

4.3.3.1 Direct intakes

Direct intakes can be designed to extract seawater at very high flow rates, therefore having significantly wider applications for large SWRO desalination plants than indirect intakes. However,

extensive preliminary treatment is normally required which involves coarse and fine screening at the intake headworks, or on land in a sump.

Furthermore, pre-treatment is required to remove marine biofouling and suspended matter which ranges from single and double stage multimedia filtration in combination with clarification or dissolved air flotation to ultrafiltration *(PETERS & PINTO, 2008)*. As a result, considerable increases in the overall treatment costs occur for these additional required processes.

From an environmental point of view, one of the main concerns of a direct intake is the entering of marine life and other debris into the desalination system. Not only does this have a negative impact on marine life, it also affects the operation of the desalination plant (e.g. fouling of desalination membranes).

The design, environmental requirements and operational aspects of direct intakes are discussed in more detail in Section 4.3.4.

4.3.3.2 Indirect intakes

The main principle of sub-bottom intakes is that the open seawater is separated from the point of intake by a geologic unit and therefore entirely dependent on site-specific hydrogeological conditions. The main available sub-bottom technologies comprise Beach wells (Horizontal and Vertical), Infiltration galleries and Seabed filtration system.

The advantage of sub-bottom intake facilities is that the source water collected is already pre-treated via the slow filtration through the seabed/sand formations from where the source water is extracted. During operation, sub-bottom intakes have generally little impact on marine life. Although these intakes are very environmentally friendly during operation, the cost implications and geological conditions have to be taken into consideration.

Beach wells

Beach Wells are well-known and commonly used in the industry and subsequently the materials for beach wells are readily available. However, beach wells are not suited for large intake volumes and the maintenance costs are quite substantial when a number of wells are required.

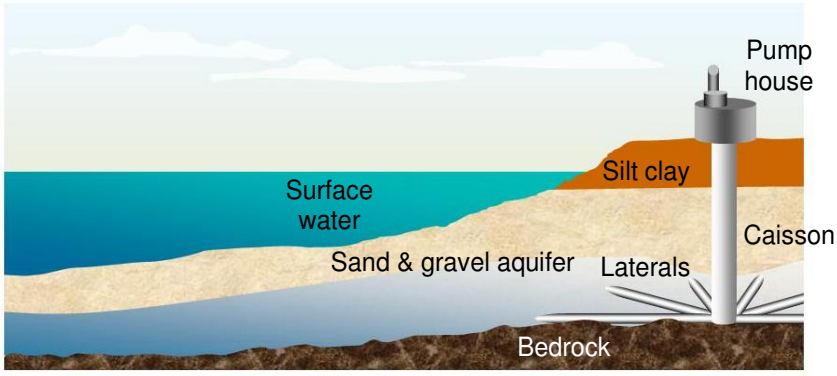


Figure 4.8: Schematic of Horizontal (Ranney) beach well

Horizontal beach wells consist of a caisson structure (or large capacity sump) and water well collector screens (laterals), which extends horizontally from inside the caisson, into the surrounding seawater aquifer. Large horizontal intake wells are called "Ranney" wells and illustrated schematically in Figure 4.8.

The wells usually have to be located within close proximity to the ocean, on the seashore and a significant beach area that could be environmentally sensitive or normally used for recreational purposes, will have to be sacrificed in order to construct a number of wells for a desalination plant that requires great volumes of feedwater.

Vertical beach wells are generally shallower than horizontal intake wells. The wells should be located as close to the coastline as possible and the extraction rate for individual vertical wells is in the order of 4 000 m^3/day.

A vertical beach well consists normally of a non-metallic casing, a well screen and a vertical turbine pump or stainless steel submersible pump as illustration in Figure 4.9.

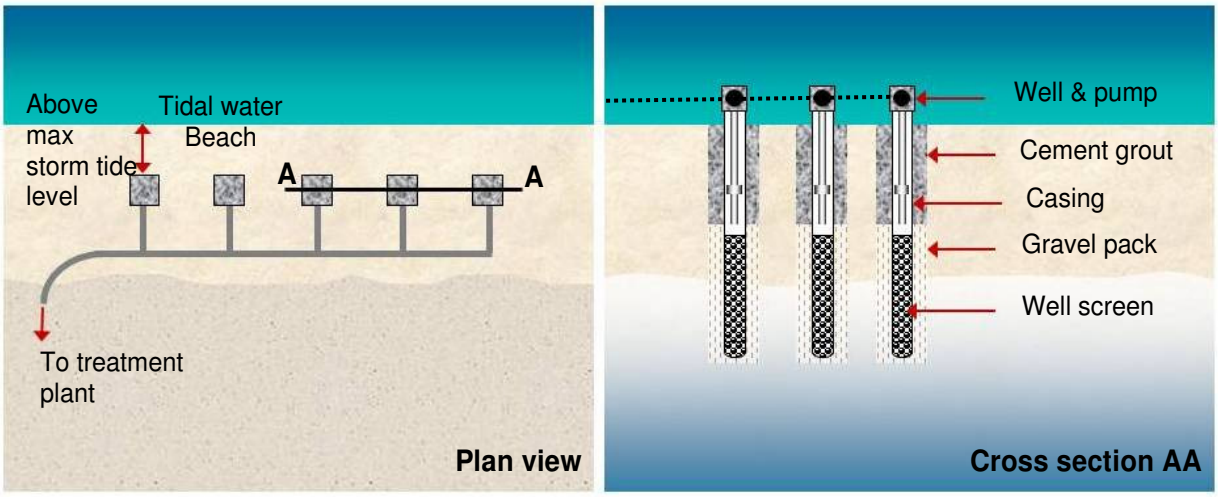


Figure 4.9: Schematic of Vertical Beach Wells

Although the vertical beach wells are normally less expensive than the horizontal wells, its yield rate is significantly less than that of horizontal wells.

Infiltration galleries

Infiltration galleries are also sub-surface seawater collection systems, based on the same concept as horizontal beach wells and normally constructed where horizontal or vertical wells cannot be used due to unfavourable hydro-geological conditions. *(PANKRATZ, 2008)*

These systems are typically shallow in depth and constructed with perforated pipes that discharge the collected seawater into a watertight chamber, from where the water is pumped to the desalination facility. In general, the extraction rate of a single collection well is between 800 and 8000 m^3/day.

Seabed filtration systems

A seabed filtration unit is not a filter itself, but a device for making the seabed serve as a natural sand filter. The filter system is located below the seabed and connected to a series of intake wells located on the shore.

These systems are normally the most expensive sub-bottom intake system and the additional operations and maintenance costs for seabed filtration systems makes this even more expensive than any other type of sub-bottom intake system. Therefore, large seawater desalination plants with capacities over 20 000 m^3/day does not normally make use of seabed filtration systems. *(PANKRATZ, 2008)*.

Environmental consideration regarding Indirect (Sub-bottom) intake systems
Fresh water aquifers could be negatively impacted by a nearby sub-bottom seawater intake and especially seabed filtration systems can cause a significant impact on the ocean floor and benthic marine organisms.

Beach wells usually have to be located within close proximity to the ocean on the seashore or beach areas. Subsequently, depending on the required feedwater flow rate for the desalination plant, a substantial area of beach habitat (environmentally sensitive beaches) or recreational beaches will have to be sacrificed. In addition, large beach strips are normally disturbed during the excavation and construction phase, causing visual and aesthetic impacts.

Typically, beach well water has very low dissolved oxygen (DO) concentrations and a reverse osmosis (RO) treatment process does not increase the DO concentration. Therefore the RO systems' brine stream will have low DO concentrations which will possibly not comply with acceptable marine water quality standards/guidelines and result in oxygen depletion and significant stress to marine life. If this is the case, the brine stream will require re-aeration, which could significantly affect the production costs of a desalination plant.

4.3.4 Direct intakes: Design considerations

4.3.4.1 Physical design considerations

A seawater desalination feasibility study was undertaken by Melbourne Water in Australia *(MELBOURNE WATER, GHD)* for a proposed desalination facility. Some of the main design requirements which they addressed and should be considered when designing a direct intake structure are summarized below.

The feedwater should normally be consistent in salinity and have low suspended solids. In order to ensure that the required feedwater quality is extracted, the following aspects should be taken into account:

- The seawater intake should be located in deeper open water which is unaffected by inflows from the land;
- The intake depth should also be sufficient to prevent the contamination of the feedwater in terms of possible oil spills or algae;
- The distance from the intake head (intake opening in the intake structure) to the seafloor should be adequate to prevent seafloor sediments from being drawn into the intake pipes;

- The opening size in the intake structure should be designed to control the local velocity of the intake water to avoid entrainment of debris or marine life, which can be ensured by designing the intake head cross section area large enough to reduce inflow velocities;
- Coarse screens are normally required at the intake head to keep out large objects and large marine life;
- Provision should be made for periodic chlorine dosing "shock dosing" to suppress marine growth; and
- The intake should be located upstream from the brine discharge location with respect to the prevailing current.

In terms of the structural stability of the intake structure, it is important to keep the inlet deep enough to avoid any interaction with boats and shipping (e.g. hazard to navigation). Furthermore, the intake structure should also be deep enough in order to minimize general wave stresses on the intake as well as air intake during stormy conditions.

Since constructing the connecting pipes or tunnels between the intake structure and the shore is expensive, it is more economically feasible to locate the intake structure as close to the shore as possible, subsequently minimizing the intake pipe lengths. Therefore, sites where deeper water is relatively close to shore are preferred, while locating the intake in deep open ocean water will provide the best intake water quality, this might lead to more difficult, expensive and time consuming construction methods.

4.3.4.2 Hydraulic design

The following design guidelines, which are specified in the Coastal Engineering Manual (*EM 1110-2-3001, 1995)*, should be taken into account to ensure the optimum hydraulic performance of a seawater intake structure:

- The water flow path between the inlet openings in the intake structure and the flow pipe/duct to shore should be streamlined to obtain water velocities increasing gradually;
- Abrupt changes in flow cross section areas should be avoided in order to minimize turbulence and consequent power loss;
- The flow section between a rectangular intake opening and a round duct (pipe or tunnel) is particularly important*:* The transition should be made over a flow distance equal to one or more conduit diameters, and
- Model tests are of great value in determining the direction of the flow path between the intake opening and the duct.

4.3.4.3 Environmental design considerations

(EPA 1985, 2001 and 2004, DESALINATION ISSUES ASSESSMENT REPORT 2003 and SYNTHESIS PAPER)

The most significant environmental concern of open seawater intake facilities is the impingement and entrainment of marine life. Recent studies have shown that the impact of marine impingement and entrainment on the natural environment is significantly more difficult to determine than the effect of the brine discharge stream.

Impingement occurs when large marine life is trapped in or against the intake screens in the intake openings due to the velocity and force of the water flowing through them. The Environmental Protection Agency (EPA) suggests that velocities at the intake screen should be 0.15 m/s or less in order to minimize impingement.

Entrainment occurs when very small and microscopic organisms (e.g. phytoplankton, zooplankton, eggs and larva) are pulled through the screens and into the abstraction system. Certain marine species can survive impingement and can be returned to the sea; however entrainment is much more difficult to control and organisms entrained into the process equipment are considered to have a high mortality rate.

The impacts of impingement and entrainment effects depend upon a number of factors, including the extraction flow rate, locations of intake and local ocean conditions. The following technological measures as well as design considerations can reduce impingement and entrainment associated with direct intake systems *(EPA, 2001)*:

- Intake location
- Intake velocity
- Velocity cap
- Screening methods
- Physical barriers
- Behavioural systems

Direct intakes: Location

An open seawater intake should be located in a region that minimizes the impact on the aquatic ecosystem. Nearshore coastal areas are generally the most biologically productive areas due to the light which penetrates to a certain depth. The feedwater intake should be placed at a depth and location of least likely impact to reduce the potential for entrainment and impingement.

Velocity

The velocity of water entering an intake structure exerts a direct physical force against which fish and other organisms must act to avoid impingement or entrainment. EPA considers velocity to be one of the more important factors that can be controlled to minimize adverse environmental impact at cooling seawater intake structures. The appropriate velocity thresholds should be based on the maximum current velocities of which the local fish species can tolerate without getting forced into the intake pipes. Although EPA recommends a threshold of 0.15 m/s, this threshold may not be sufficiently protective for certain species and alternative requirements may be developed for specific areas/circumstances.

Two velocities are of importance in the design of intake structures: the approach velocity and the through-screen velocity. The approach velocity is the velocity measured just in front of the screen face or at the opening of the cooling water intake structure in the surface water source. This velocity has the most influence on an aquatic organism and its ability to escape from being impinged or entrained by the seawater intake structure.

The through-screen velocity is the velocity measured through the screen face or just as the organisms are passing through the opening into another device (e.g., entering the opening of a velocity cap). This velocity is always greater than the approach velocity because the net open area is smaller.

Velocity Cap

In order to prevent the entrainment of fish, the vertical entrance of an offshore intake structure can be "capped" (refer to Figure 4.10). A cover (velocity cap) is placed over the entrance of the intake structure, therefore converting vertical flow to horizontal flow and reduces fish impingement. Velocity caps have been found successful in reducing impingement, but not the entrainment of eggs and larvae, thus velocity caps are often used in combination with other devices.

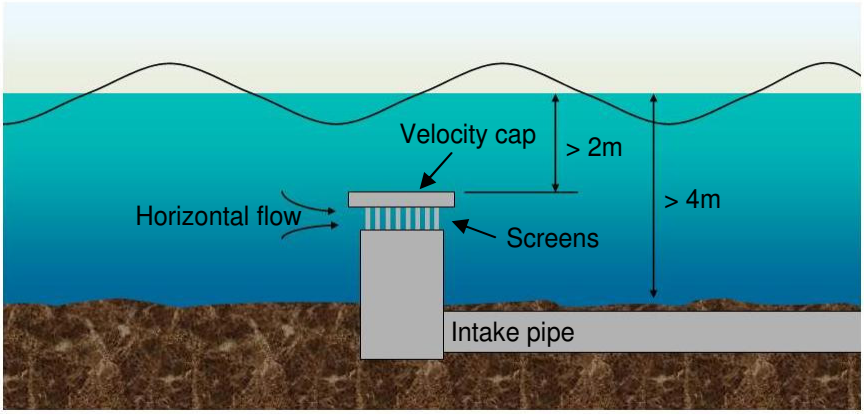


Figure 4.10 Intake with a velocity cap headworks

In the case of rectangular intake openings in the intake structure, the United States Environmental Protection Agency suggests that the optimum width of the horizontal entrance should be approximately 1.5 times greater than the height to ensure uniform flow and minimize entrainment.

Physical Barriers and screening methods

- *Travelling Water Screens* consist of revolving wire mesh panels with 6 to 9.5 mm openings to prevent the intake of debris or marine organisms. The wire mesh panels revolve out of the flow and a high-pressure water spray removes accumulated debris which is further disposed of. The panels can be located directly onshore, or at the end of a long channel or intake pipe that extends beyond the surf zone.

- *Fine mesh screens* are typically installed on a conventional travelling screen with openings ranging from 0.5 mm to 5 mm. The screen is designed to exclude eggs, larvae and juvenile fish from open intakes. However, fine mesh screens could result in substantial operational problems due to the increased amount of debris that will be removed along with the marine life.

- The *Ristroph screen* is a modification of the conventional travelling water screen. Instead of revolving wire mesh panels preventing the intake of debris or marine organisms, the Ristroph screen panels are fitted with fish buckets that collect fish and then lift them out of the water before debris is removed with a high pressure spray. Ristroph screens were reported to

improve impingement survival up to 80% in contrast with the conventional screens which average at about 15%. However, these screens do not affect the entrainment of marine organisms

- *Passive Screens* comprise slotted screens, which are constructed of trapezoidal-shaped "wedgewire" and aligned on a horizontal axis with openings from 0.5 mm to 10 mm. The cylindrical shape dissipates the through slot-velocity allowing organisms to escape the flow field. As indicated in Figure 4.11, passive screens are normally constructed of much larger pipes than the ultimate intake pipe in order to reduce flow velocities to less than 0.15 m/s. These screens can reduce impingement and entrainment (up to 80%) of marine organisms effectively. The screens are normally cleaned with an air backwash system when debris accumulates and adequate counter-current flow is needed to transport organisms away from the screen.

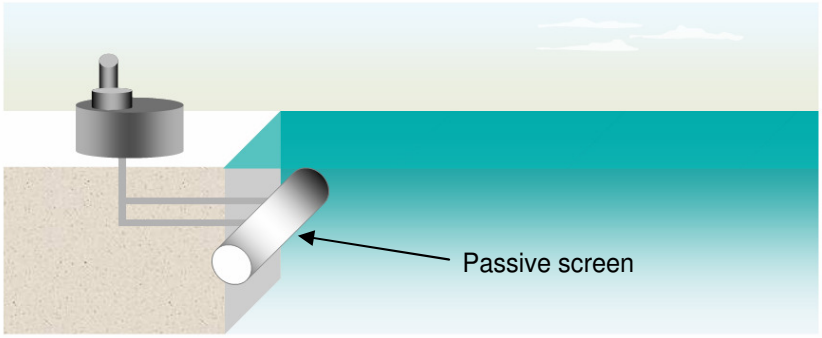


Figure 4.11: Passive screen seawater intake

- *Louvers* are a series of vertical panels placed perpendicular to the intake approach flow. A new velocity field is created parallel to the face of the louvers which carries fish away from the intake and towards a fish bypass system. Louvers are also classified as behaviour barriers since these rely on a fish's ability to recognize the new flow field and swim away.

- *Angled travelling screens* work similar to louver systems but consist of flow-through screens rather than solid panels.

- *Filter barriers* reduces intake impingement as well as entrainment by placing a full-depth porous filter fabric at the entrance of the intake structure. The openings of the fabric can range between 0.4 mm to 5 mm and it is suspended by a floating boom and anchored to the seabed. The system is designed to provide enough surface area in order to maintain the through-flow velocity low enough to avoid impingement of fish.

- *Fish net barriers* are wide-mesh (4 mm to 32 mm) nets used primarily for the prevention of impingement of fish, but not effective in reducing entrainment.

Behavioural systems

Behavioural fish avoidance systems rely on a fish's instinct to swim away from synthetically generated light, bubbles, sound or hydraulic regime. Various technologies have been developed over the years such as light barriers, sound barriers or air bubbles. However, according to the

Environmental Protection Agency of the USA, these systems have been generally ineffective up to date and not frequently used in practise.

Environmental design guidelines (www.epa.gov/)

EPA (Environmental Protection Agency in the USA) developed regulations for the location, design, construction and capacity of cooling water intake structures, reflecting the best technology available for minimizing negative environmental impact. These can also be applied to open-water intakes for seawater desalination plants.

Cooling water intake structures cause adverse environmental impact by entraining large numbers of fish and shellfish or their eggs. EPA specifies three rulemaking phases addressing cooling water intakes:

- Phase I rule, promulgated in 2001, covers *new facilities*
- Phase II rule, promulgated in 2004, covers large *existing electric generating plants*
- Phase III rule, promulgated in 2006, covers certain *existing facilities* and *new* offshore and *coastal oil and gas extraction facilities*

In addition to the EPA guidelines, the US Army Corps of Engineers also provides design guidelines for the planning and design of hydroelectric power plant stations which should be taken into account when designing a seawater abstraction system. The relevant documents are listed in Table 4.5 and can be downloaded from the EPA and US Army Corps websites. *(www.epa.gov/ and www.hnd.usace.army.milTECHINFO/)*

Table 4.5: EPA and US Army corps guideline documents

Environmental Protection Agency			
	Date	**Document ref**	**Title**
Phase I – New Facilities			
Final Rule	18-Dec-01	40 CFR Parts 9, 122, et al.	National Pollutant Discharge Elimination System: Regulations Addressing Cooling Water Intake Structures for New Facilities; Final Rule
Technical development document	09-Nov-01	EPA-821-R-01-036	Technical Development Document for the Final Regulations Addressing Cooling Water Intake Structures for New Facilities
Phase II – Large Existing Electric Generating Plants			
Final Rule	09-Jul-04	40 CFR Parts 9, 122 et al.	National Pollutant Discharge Elimination System—Final Regulations To Establish Requirements for Cooling Water Intake Structures at Phase II Existing Facilities; Final Rule
Technical development	12-Feb-04	EPA 821-R-04-007; DCN	Technical Development Document for the Final Section 316(b)

document		6-0004	Phase II Existing Facilities Rule
Phase III – Certain existing facilities and new offshore and coastal oil and gas extraction facilities			
Final Rule	16-Jun-06	CFR Parts 9, 122, 123, et al.	National Pollutant Discharge Elimination System; Establishing Requirements for Cooling Water Intake Structures at Phase III Facilities; Final Rule
Technical development document		EPA-821-R-06-003	Technical Development Document for the Final Section 316(b) Phase III Rule
U.**S. Army Corps of Engineers**			
Engineer Manual	30-Apr-95	EM 1110-2-3001	Planning and Design of Hydrolectric Power Plant Structures

4.3.4.4 Operation and Maintenance

(MELBOURNE WATER & EPA 2000)

Maintenance of marine structures is in principal more expensive and difficult than land-based structures. Physical processes which greatly determine the frequency, method, accessibility (divers, supporting vessels, etc.) and cost of maintenance includes the wave climate (seasonal), wind regime, currents, water depth, turbidity and distance offshore. Furthermore, the availability of certain marine plant and the risk involved with dangerous techniques which divers could be required to perform also influences the maintenance cost.

Discarded fishing nets and marine organisms (e.g. masses of dead kelp) can completely block single intake structures with static screens (i.e. without removal systems). Periodical cleaning of the intake screens is required to prevent blockage and subsequently cause increased velocities and energy requirements of the intake water. Furthermore, the following mitigation measures, some of which is recommended by the EPA, can be taken to prevent/minimize marine growth and clogging of the intake:

- Piled structures can be constructed around single intakes to protect the intake from large debris;
- Some form of air blasting system (installed through pipes placed in the intake screen system) may be periodically used to assist in screen maintenance to prevent marine build up
- The construction of several separate intake risers could reduce the risk of total blockage
- If seawater at proposed site contains a high concentration of suspended silt and fine sand, a sedimentation settling basin can be constructed upstream of the pump station in order to remove harmful particles to the pumps, pipes and desalination plant.
- A macerator located upstream of the intake pump station can grind any marine growth
- Provision should be made for chlorine dosing pipes to prevent marine growth inside he intake pipelines.
- Provision to seal off pipe entrance in order to clean pipes and intake structures

The seawater at the intake should be sampled on a regular basis in order to monitor the seawater quality which could affect the desalination process.

4.4 MARINE PIPELINES

If feedwater is extracted from an offshore intake structure, the feedwater is transferred via a tunnel or pipeline from the seawater intake to land. The brine waste stream is transported from the headworks (or brine holding tank pump station) through the main outfall pipe out to the diffuser. The following section summarizes the general considerations which have to be taken into account when designing a sea pipeline.

4.4.1 Hydraulic design

The hydraulic study is based on the hydraulic energy balance for the complete system by comparing the specific energy between any two points in the system and taking into account all friction and fitting losses between two adjoining points. This balance ensures continuity of flow. *(Chadwick, A & Morfett, J, 1998)*

The energy principle is represented by the Bernoulli equation (refer to Appendix A) and the continuity equation for steady incompressible flow is used for brine discharge.

The friction losses in pipes depend on the pipe diameter, mean flow velocity, wall roughness, density and viscosity of the fluid. The Darcy-Weisbach, Colebrook-White (refer to Appendix A) formulae are the generally accepted formula to determine friction losses in pipes

The Local head loss h_L (transition losses) occurs where the flow velocity changes in magnitude or direction. All local losses due to friction and turbulence at inlets, converging sections and bends in the pipe should be determined. *(Refer to Appendix A for equation).*

4.4.2 Structural design

4.4.2.1 Physical processes

Information on the relevant physical processes *(DWAF, 2004)* which have to be investigated together with the possible impact on the design are summarized below.

- *Bathymetry:* A bathymetry survey provides water depth contours, the slope of the seafloor, protruding reefs and offshore sandbars. An echo-sounder survey from a survey boat can be used for this purpose. The bathymetry will determine the pipeline length and subsequently the construction costs required to reach the necessary water depth.

- *Seabed physiography* determines the method therefore also affects the cost of construction and a side scan sonar survey can be used for this purpose. Large regions of exposed rock affect the trench excavation through the breaker zone and high points might require some offshore blasting.

- *Sub-seabed conditions*: If a pipe is buried in a trench, the cost of excavation of the trench depends on the sub-seabed composition. Seismic surveys obtain information from beneath the sea-floor and exploratory drilling is normally required to verify results of such a survey. Further geotechnical investigations required in detail design include soil analysis, rock analysis and seismic stability.

- *Sediment movement*: Large seasonal changes in sand depth can occur in the surf zone due to wave-induced turbulence and currents, especially during storms and these can affect the stability of the seafloor. Therefore, the required installation depth of a pipeline in the surf zone should be below the seabed erosion profile.

- *Waves*: Wave data are crucial for detailed structural design of the pipeline, the development of a feasible construction method as well as forces to which exposed parts of a marine pipeline will be subjected during its lifetime.

- *Tides*: Tidal variations also have to be taken into account for the hydraulic design with respect to the available head (pressure or gravity).

4.4.2.2 Pipe stability

The stability of a pipeline resting on the seafloor is affected by the forces caused by waves and currents, the resistance from the seafloor and the physical characteristics such as outside diameter, weight, length, etc. of the pipeline.

In order to determine the wave forces, offshore wave data should be obtained and transformed to the nearshore area using numerical wave refraction, shoaling and breaking model (e.g. SWAN). Since the water depth varies along the pipeline and because the pipeline may traverse regions of wave concentration and dissipation, it is necessary to determine the wave conditions at a number of positions along the pipeline.

Currents impose a load on a marine pipeline which is additional to that of waves and can be obtained by a flow meter or by numerical flow modelling of the region.

The hydrodynamic forces acting on the pipe can be represented by the Morrison equations. These equations are functions of time since the orbital velocity and acceleration change throughout a wave cycle. *(Refer to Appendix B for equations to calculate the forces on a pipeline and the stability of a pipeline resting on the seabed.)*

The pipeline stability can be determined using the:

- Det Norske Veritas (DNV) 'Rules of Submarine Pipelines 1996' and
- DNV's 'On-bottom stability design of submarine pipelines' of October 1988 (Also referred to as DNV 1996 and 1988 respectively).

4.4.2.3 Material selection

Marine growth inside a seawater intake pipeline could have a major impact on the hydraulics and subsequently the operation of a desalination plant. Therefore pipe materials which are more resilient to marine growth such as HDPE should be considered. However, all of the following criteria should be considered when selecting the most optimum material*(MOSTERT, 2009)*:

- Design life of pipeline
- Financial aspects related to:
  - The cost of the pipe
  - Special financial requirements (e.g. Deposit payment)
  - Availability of local support
- Importance of required inside diameter, depending on:
  - The design flow
  - Hydraulic losses
  - Available head (pumps or gravity)
- Installation of pipeline:
  - Special installation requirements/methodology (e.g. bottom tow method requires resilient seabed material)
  - Cost of installation
  - Handling of pipes
  - Installation above or below seafloor (method and soil conditions)
- Availability of components
  - Availability of pipes
  - Manufacturing ability
  - Fittings and bends
- International acceptance
- External and internal longterm protection requirements
- Maintenance
- Testing procedures
- Support to local industry

Table 4.6 summarizes the strengths and weaknesses of the available materials which are most commonly used for marine pipelines *(MOSTERT, 2009)*.

Table 4.6: Material selection

Pipe material	Max dia (m)	c* (m/s)	ks** (mm)	Advantages	Disadvantages
HDPE	SA – 1.2 Eur – 2.0	Approx 500	0.003 to 0.015	• Corrosion & Chemical resistance • Abrasion resistance (thick walls) • Relatively light weight • Flexible • No stress cracking • Ability to install seamless pipeline • Proven technology • Non toxic	• Lack of UV resistance • Not suited for suspended applications – prone to sag/stretch/shrink • Thick walls – fusion/weld/melt conditions much more difficult and critical • Fittings and connections could be difficult to install underwater • Required weighting

				• Tough • Impact strength • Many jointing options • Ease of fabrication • Proven quality standards	• Creep • Not situated for rocky areas
PP		Approx 600		• Ability to withstand high temperatures • Corrosion and chemical resistance • Abrasion resistance • Relatively light weight • Flexible • No stress cracking • Ability to install seamless pipeline	• Limited range of sizes • Limited expertise – manufacture/installation • Limited range of jointing options • Inadequate specifications • Inadequate training for installers • Imported for Special Applications
GRP	2.5	Approx 600	0.06	• Lightweight (especially large dia) • Corrosion resistance • Ease of handling and installation • Easy jointing • Ease of fabrication • Aggressive environments/media • Higher pressures (up to PN32) • Low co-efficient of friction • Low wave celerity	• Positively buoyant in water • Not resistant to negative pressures without trench preparation • Require careful handling • Compacted sand backfill around pipe is required for support the pipe from deforming and cracking when negative pressures occur inside or from overburden pressures • Proper dewatering of trench excavations required • Cannot use as a subsea pipe where there are waves • Cannot be laid directly on bedrock without specific bedding preparation. • Require concrete thrust blocks when using bell&spigot • Subsea GRP pipe needs to be buried • Bedding and backfilling construction costs more than that of a continuous jointed pipe such as steel or HDPE
Steel	Unlimited - 4	830 to 1000		• High strength (245 – 450 MPa) • Can be laid on bedrock • No specific and inexpensive bedding and backfilling • Many skilled labour available for welding pipes • Can be installed as a continuous pipe length • Specials pipes such as diffusers can be fabricated • Bedding and backfilling construction costs of steel pipes less than that of GRP pipes • Can be installed as a subsea pipes	• Protection against corrosion (need to keep cathodic protection operational for lifespan of pipe) • High cost in comparison with other pipe materials • Temporary works and construction of steel marine pipelines are more expensive than GRP and HDPE pipe materials • Heavy to handle, especially in large (e.g. 1 m diameter) sizes

				at a site having waves • Can be installed as a subsea pipe directly on bedrock	
Ductile Iron Pipe		1000 - 1200		• High tensile strength • High impact strength • High pressure • High ring bending strength • High beam strength • High circumferential tensile strength • High design safety factors • Increased pressure headroom • Every pipe pressure tested	• Imported • Prone to exchange rate fluctuations • Long lead times • Not labour intensive friendly • Only standard range of fittings – use for steel fabrication
Concrete			0.15	• Strength increase with time • Soil conditions, bedding and load not problematic • Constant internal shape and size as well as rugged abrasion resistant surface	• Corrosion of steel reinforcement when used in marine environments • Heavy to handle, especially when used as a subsea pipe • Locally available pipes would need to be redesigned if used as a subsea pipe • Cannot be laid directly on bedrock in a trench or in the sea without specific bedding preparation. • Cannot be used as a subsea pipe where there are waves as installation requires flat water conditions • Subsea concrete pipe needs to be buried for stability • Bedding and backfilling construction costs in a subsea application are more than that of a continuous jointed pipe suchas steel or HDPE making it not feasible as a subsea pipe

* Wave celerity for 6 to 16 Bar working pressure

** Roughness (new pipe)

4.4.3 Installation methodology

The main concerns regarding the construction of marine structures in general, include the constructability (design specifications), the contractor experience and resources, site specific conditions during construction (e.g. storm, exposed coastline, rock), the access of construction plant, construction materials and the site area available for storage of materials.

The following installation methodologies for offshore marine pipelines are the main technologies. The choice depends on site specific conditions (e.g. exposed coastline, bedrock, etc.), pipe length and material, as well as the contractor's experience and marine construction plant available. *(DIXON, 2007)*

Lay-barge (conventional)

The method of using a lay barge involves a specially designed vessel with facilities to weld pipe lengths (normally 24m) into a continuous pipeline and at the same time lowering it onto the sea bed. The pipeline is static relative to the seabed and the barge moves forward as it increases the length of the pipeline. Most lay-barges are held in position by anchors however some of the modern lay-barges make use of dynamic positioning systems.

S-lay and J-lay

The S-lay derives its name from the suspended shape of the pipe at the end of the vessel, a gentle "S" from the stringer (a profiled support for the pipeline as it enters the water) to the seabed. In order for the pipe to hold its shape, the pipe is held under tension. S-lay operations can be undertaken in water depths ranging from 10m to 500m, but have been laid in depths up to 2000m.

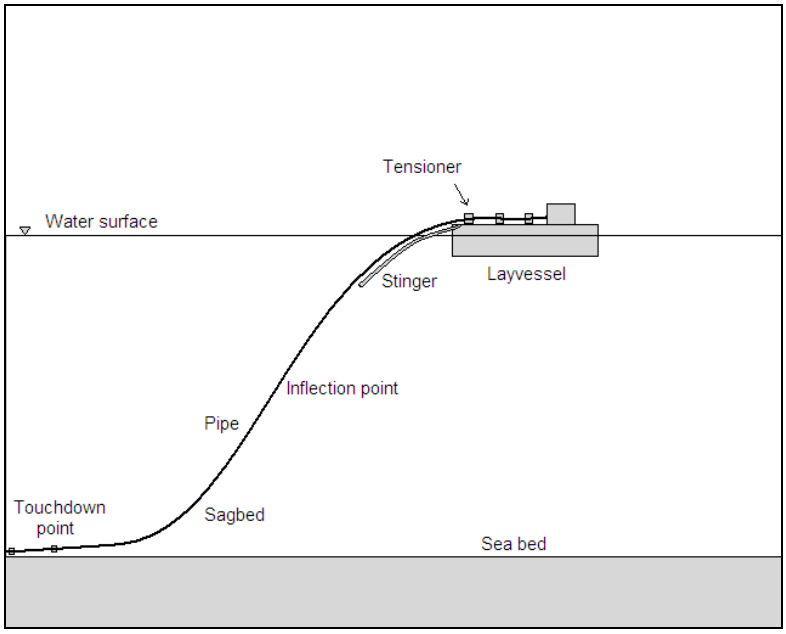


Figure 4.12: Schematic illustration of S-lay pipeline installation method

The J-lay takes its name from the suspended shape of the pipe at the end of the barge. The curve is similar to a catenary and develops lower stress levels in the pipe compared to an S-lay. Unlike the S-lay, tension is not required to maintain the installation shape. Deeper water is required for a J-lay operation and generally used for deep water pipelines.

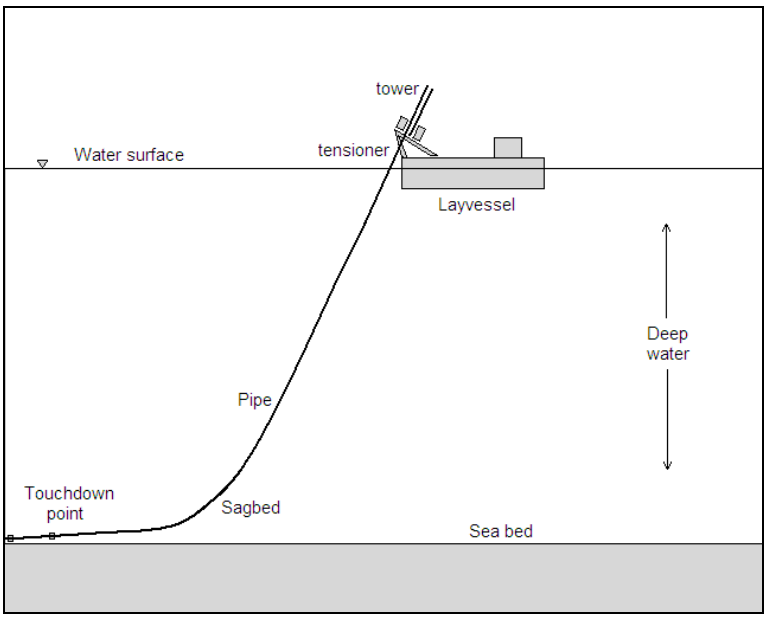


Figure 4.13: Schematic illustration of J-lay pipeline installation method

Surface Tow

A long length of pipeline is floated into position and then gradually sunk to the seabed. The pipeline is floated using its own buoyancy (i.e. closed ends with air in the empty pipe) or additional buoyancy can be attached to the pipeline if required. This method must be properly engineered to prevent damage to the pipeline due to:

- "Snaking" in response to waves, currents and wind – this method requires very calm sea conditions
- Buckling of the pipeline during the ballasting procedure, when water runs to one end of the pipeline causing it to sink sharply.

Near Bottom Tow

The pipeline is made slightly negative buoyant by attaching short lengths of chain at frequent intervals to the pipeline. Once in position, the pipeline is ballasted to the seabed by filling it with water. Since the pipeline is closer to the seabed than during the Surface Tow Method, the risk of buckling is reduced.

S-Curve Float and Sink

This method is primarily used for the installation of HDPE pipelines. Pipeline lengths are towed on the surface until it is in position. The shore floating end of the pipeline is then pulled to the seabed at which point water is introduced at the submerged end. Tension is applied at the off shore end to maintain the pipeline in position and control the curvature of the pipeline and thus the stresses that develop. The air in the pipeline is then released from the offshore end in a controlled manner and the pipeline sinks to the seabed in an "S" curve.

Bottom Tow

This method involves dragging the pipeline into position along the seabed. The bottom-tow method can either be "pull offshore" or "pull onshore". Pull offshore is when the pipeline is assembled onshore and pulled by a pulling barge or vessel offshore. Pull onshore is when the pipeline is pulled onshore from a laybarge moored some distance offshore

4.4.4 Maintenance and operation requirements for marine pipelines

Seawater intake pipes: In order to ensure sustainability, it is normally best to construct two or more separate intake pipelines for large desalinisation plants. Subsequently the maintenance and cleaning operations can be performed on one intake pipe, while seawater can still be abstracted through the other pipe.

Brine outfall pipe: It is standard practise to discharge effluent intermittently in cases where the discharge flow are less than the design flow in order to ensure that scour velocities are maintained in the main pipeline and the required port velocities are achieved to meet the required dilutions and adhere to marine water quality guidelines. For this purpose, a brine storage tank and air valves will be required.

In general, pipes and fittings will have to be monitored on a regular basis and provision should be made for periodically flushing/cleaning of the marine pipes by means of e.g. pipe pigging. Note that this is more critical for the intake than the discharge pipes in terms of feedwater quality. However, corrosion in flanges and "stagnant" areas (i.e. dead ends) in pipelines will be more severe in the brine pipelines due to the more saline seawater.

According to The Professional Divers's Handbook *(BEVAN, 2005)*, a Pig is a device which can be used to clean the inside marine pipelines and remove sediment deposits (Wirebrush Pig) or hard deposits such as mussels (Scraper Pig or Mandrel Pig) which could obstruct or retard the flow through a pipeline. These devices are inserted into the pipeline and travel throughout the length of a pipe driven by the product or feedwater flow. Although the optimum performance speed of is specified by the manufacturer, the speed that the pig should travel through a pipe normally ranges from 0.5 m/s to 4 m/s.

Normally for seawater intake pipelines, pigging systems are designed as "One way" open systems which mean that the pig will have to be removed from the pipeline at the end of each run. Subsequently this could lead to problems with both safety and process interruption since the pipeline need to be opened at the pig receiving end and the pig removed and replaced at the other end of the pipe. Therefore, it is necessary to ensure that marine fouling is minimized as much as possible by mans of other mitigation measures such as chlorine dosing, maintaining high flow velocities and coating the inside of the pipes with anti-foul paints. In general, once marine fouling has been allowed to accumulate to a great extend, removal becomes complex and expensive.

The latest technologies for pigging systems are highly sophisticated and the equipment has a global positioning system fixed on it in order to determine the exact location of the pig inside the pipeline while moving. Along with the GPS positioner there are also other instruments such as an internal camera that takes live video of the pipe condition inside while the pig is moving as well as a thickness gauge that constantly measures the thickness of the wall of the pipe as the pig moves.

It is best practise to make provision to re-circulate seawater in the intake pipes during extended plant shutdowns (i,e, say more than 2 weeks), to prevent intake of the stagnant water in the pipe that could be toxic (stagnant water in a pipe will tend to cause die-off of marine life).

4.5 BRINE DISCHARGE

A desalination plant will consist of a feedwater stream, a product water stream, and a waste stream of concentrated reject solution (refer to Section 4.2 of this thesis). The various discharge options for the brine are briefly summarised in Table 4.7. However, only the components of an offshore ocean outfall were investigated in detail and the findings provided in this Section.

Table 4.7: Advantages and disadvantages of various discharge options

Discharge option	Advantages	Disadvantages
Offshore ocean outfall: Offshore, submarine ocean outfalls are submerged offshore pipes that discharge the concentrated reject solution (brine) directly to sea	• Most common • Least expensive • Discharge large volumes	• Significant negative impact on marine environment
Co-locating with an Existing Power or Wastewater Treatment Plant	• Brine stream normally diluted to environmental requirements due to large amount of power plant discharge in comparison with desalination discharge • Discharge large volumes	• Possible impact on marine environnement
Discharging to the Influent of a Wastewater Treatment Plant	• No negative impact on marine environment, but possible negative impact on rivers	• Smaller discharge volumes • Pre-treatment of brine might be required
Discharge to evaporation ponds	• No negative impact on marine environment, but possible impact on surrounding natural environment	• Smaller discharge volumes • Large land requirements make evaporation ponds uneconomical for many developed or urban areas
Discharge to confined aquifers	• No negative impact on marine environment, but possible negative impact on aquifer	• Expensive • Difficult to ensure other groundwater resources remain uncontaminated
Discharge in rivers that flow into an estuary	• No negative impact on marine environment, but possible negative impact on estuary	• Small volumes • Impact on salinity to be assessed

The chemical constituents and physical behaviour of a brine discharge pose a threat to marine organisms. Brine can kill organisms in the short terms, but may also cause more subtle changes in the community assembly over longer time periods.

4.5.1 Brine composition

The composition of the brine effluent is very critical, since the type of constituents and their concentrations will directly affect the required dilutions which have to be achieved and subsequently the entire outfall design. Table 4.8 provides the typical effluent properties of RO and MSF desalination plants as assessed by Latermann and Höpner *(LATTERMANN & HÖPNER, 2007)*:

Table 4.8: Typical effluent properties of RO and MSF seawater desalination plants

	Reverse Osmosis (RO)	Multi-stage Flash Distillation (MSF)
Physical properties		
Salinity	Up to 65 000 – 85 000 mg/L	About 50 000 mg/L
Temperature	Ambient seawater temperature	+5 to 15°C above ambient
Plume density	Negatively buoyant	Positively, neutrally or negatively buoyant depending on the process, mixing with cooling water from co-located power plants and ambient density stratification.
Dissolved oxygen (DO)	If well intake used: typically below ambient seawater DO because of the low DO content of the source water. If direct intakes used: approximately the same as ambient seawater DO concentration	Could be below ambient seawater salinity because of physical deaeration and use of oxygen scavengers
Biofouling control additives and by-products		
Chlorine	If chlorine or other oxidants are used to control biofouling, these are typically neutralized before the water enters the membranes to prevent membrane damage.	Approximately 10-25% of source water feed dosage, if not neutralized
Halogenated organics	Typically low content below harmful levels	Varying composition and concentrations, typically trihalomethanes
Removal of suspended solids		
Coagulants (e.g. iron-III-chloride)	May be present if source water is conditioned and the filter backwash water is not treated. May cause effluent coloration if not equalized prior to discharge.	Not present (treatment not required)
Coagulant aids (e.g. polyacrylamide)	May be present if source water is conditioned and the filter backwash water is not treated.	Not present (treatment not required)
Scale control additives		
Anti-scalants	Typically low content below toxic levels	Typically low content below toxic levels
Acid (H_2SO_4)	Not present (reacts with seawater to cause harmless compounds, i.e. water and sulfates; the acidity is consumed by the naturally alkaline seawater, so that the discharge pH is typically similar or slightly lower than that of ambient seawater.)	Not present (reacts with seawater to cause harmless compounds, i.e. water and sulfates; the acidity is consumed by the naturally alkaline seawater, so that the discharge pH is typically similar or slightly lower than that of ambient seawater.)
Foam control additives		
Antifoaming agents (e.g. polyglycol)	Not present (treatment not required)	Typically low content below harmful levels
Contaminants due to corrosion		
Heavy metals	May contain elevated levels of iron, chromium, nickel, molybdenum if low-quality stainless steel is used.	May contain elevated copper and nickel concentrations if inappropriate materials are used for the heat exchangers

Cleaning chemicals		
Cleaning chemicals	Alkaline (pH 11-12) or acidic (pH 2-3) solutions with additives such as: detergents (e.g. dodecylsulfate), complexing agents (e.g. EDTA), oxidants (e.g. sodium perborate), biocides (e.g. formaldehyde)	Acidic (pH 2) solution containing corrosion inhibitors such as benzotriazole deravates

Seawater reverse osmosis plants typically operate with a recovery of 30 to 60 percent of product water from the feedwater. The salt concentration of the waste stream can be quantified by the concentration factor (CF) given by:

$$CF = 1/(1-\text{recovery})$$

Therefore, a reverse osmosis plant operating at 50% (0.5) recovery will have a concentrated reject solution with twice the salt content as the feedwater. *(The salinity of seawater is relatively constant at about 25 -35 ppt.)*

Chemicals used throughout the desalination process may also be discharged with the brine. Most of these chemicals are applied during pretreatment to prevent membrane fouling which normally includes chlorine and other biocides. Anti-scalants, such as polyacrylic or sulfuric acid, are also added to prevent salt deposits from forming on piping. Coagulants, such as ferric chloride and polymers, could also be added to the feedwater to bind particles together.

In addition to the above, a number of chemicals are used to clean the RO membranes. Membranes are normally cleaned every three to six months with typical industrial soaps and acids such:

- Hydrochloric Acid
- Citric Acid
- Sodium Hydroxide – pH 11 to 12
- Trisodium phosphate, EDTA and Sodium Hydroxide pH 11 to 12

When the membranes are cleaned, the first rinse, which contains a majority of the cleaning solution, is typically neutralized and disposed of in local treatment systems. Subsequent rinses, however, are often discharged into the brine.

During the desalination process, heavy metals such as copper, lead and iron could also be introduced to the brine stream. This could be due to corrosion of the desalination equipment or following maintenance of the plant. Table 4.9 shows in more details of the typical composition of a RO plant which was investigated by Strategen *(STRATEGEN, 2004)*.

Table 4.9: Typical composition of concentrated seawater from RO plant

Parameter	Seawater* (inflow)	RO seawater return	Backwash water**	Final discharge		Comments
				Pre-dilution	Port-dilution***	
Flow	x	~ 0.5 x	~ 0.05 x	~ 0.6 x	n/a	Constant design flow, no peaks expected, base load operation
Temperature (°C) dedicated intake	20 – 24	22 – 26	20 – 24	22 – 26	20 – 24	Pre-dilution discharge is 1 – 2°C above ambient seawater temperature. *RO process raises temperature 1 – 2°C*
Temperature (°C) using cooling water	27 – 36	29 – 38	27 – 36	29 – 38	20.3 – 24.3	Cooling water up to 13°C above ambient.
pH	~ 8	~ 6 - 7	~ 6 - 7	~ 6 - 8	~ 8	Reduction in pH due to the dosing feedwater by sulphuric acid during pretreatment
TDS (salinity) (mg/L)	35 900	63 537	35 900	61 195	36 450	TDS concentration increases as result of RO process
TSS (mg/L)	10	0.5	275	24	10.3	Increase in TSS due to $FeCl_3$ dosing. Solids in TSS comprise mainly $Fe(OH)_3$
Iron (inlc. In TSS) (mg/L)	1.38	0	55	5	1.5	Increase in iron concentration due to $FeCl_3$ dosing
Chloride (mg/L)	19 393	34 323	19 393	33 058	19 690	
Free chlorine (mg/L)	-	0	0	0	0	Neutralised
Sulphate (mg/L)	3 154	5 572	3 154	5 367	3 202	Sight increase in sulphate concentration due to dosing of H_2SO_4
Biron (mg/L)	4	5.9	4	6	4	Biron rejection of 71% in membranes is assumed

* Average values for normal seawater

** Based on continues discharge of filter backwash water through a storage tank

*** At the edge of the initial mixing or dilution zone based on an arbitrary dilution of 50 and will vary for each design

The environmental effects of brine disposal can be minimized by reducing the amount of chemicals which are applied during the pre-treatment processes, by either man-made or natural filtration processes. For instance, ultrafiltration can replace coagulants, which effectively removes silt and organic matter from feedwater, as well as some of the guesswork involved in balancing the pre-treatment chemicals.

4.5.2 Location selection

Table 4.10 lists aspects relevant to the outfall location selection *(Desalination Issues Assessment Report, 2003)* which is very similar to selecting the most optimum intake location. It should be

ensured that there is adequate distance between the seawater intake and brine discharge point and the brine is dispersed sufficiently. Otherwise, the brine will have an impact on the quality of the intake water.

Table 4.10: Aspects relevant to outfall location selection

Beneficial uses in water and beach areas
Environmental sensitive areas (refer to Table 4.11: Scale of sensitively of Marine Habitats)
Recreational areas
Port demarcated areas (i.e. vessel navigation)
Industrial use
Physical characteristics
Required water depth (i.e. effected by the sea level and required dilutions)
Meteorological conditions
Oceanographic conditions (i.e. seabed slope, bathymetry)
Environmental processes
Waves (i.e. *construction contraints, structure stability*)
Currents (i.e. *construction contraints, structure stability*)
Environmental impacts
Pollution
Marine biology

The sensitivity of the marine habitat subsystems to the effect of desalination plants is site specific. Table 4.11 arranges the marine habitat subsystems in the sequence from lowest to highest sensitivity according to a paper written by Einav and Harussi, *(EINAV & HARUSSI & PERRY, 2002).*

Table 4.11: Marine habitat sensitivity to desalination plants

Scale of sensitivity	Marine habitats	Comments
1	High-energy oceanic coasts, rocky or sandy with coast-parallel current	• By provision of oxygen, nutrients and energy, good conditions of biodegradation are provided. • Energy input prevents local accumulations. • Rapid water exchange prevents damages by high salinity and elevated temperature.
2	Exposed rocky coast	• "Good" water exchange, even in small niches.
3	Mature shoreline (sediment mobility)	• Sediments mobility prevents local accumulation of particle-adsorbed matter. This type of coast does not show bays and lagoons of long water and sediment residence times.
4	Coastal upwelling	• The danger of stagnant beach-near water is greater than in case 1. • Conditions change seasonally. • Seasonally the water contains nutrients, plankton and suspended solids, which is not desireable for a desalination plant. • The sensitivity of the sub ecosystem, however, is lower than in the forthcoming classes.
5	High-energy soft tidal coast	• There are large intertidal areas and large sediment surfaces susceptible to adsorption and accumulation. • The water exchange and the sediment mobility, however, are high.
6	Estuaries and estuary-similar	• Similar to case 5, additionally high nutrient input which favours

		biodegradation. Because of turbidity and seasonal water quantity changes not desirable for desalination plants.
7	Low energy sand-, mud-, and beach rocks-flats	• Sensitive because of high individual numbers at low species numbers. • Loads may accumulate because of adsorption to large surfaces and because of evaporation. Limited water exchange.
8	Coastal sandbanks	• Sandbanks mostly continue into the intertidal zone and change into area types like 7, exposed to wind and dust. • The sandbanks themselves are flooded occasionally. • Degradations work only during the rare inundation periods. • Solar stress.
9	Fjords	• Enclosed deep water bodies of limited exchange. • Danger of thermo-clines and oxygen deficits in the depth. • Shelter and breeding areas of sea animals.
10	Shallow low-energy bay and semi-enclosed lagoon	• Similar to 7, but exchange is still lower. • Load consequences add to natural stress factors like high and changing salinities, changing water level, solar irradiation.
11	Algal (cyanobacterial) mats	• Wide intertidal areas at very low beach slopes. • To give algal mats a relatively high position within a sensitivity scale is among others a question of precaution. • At one hand algal mats are very sensitive to salt, irradiation, dryness and even oil. • At the other the sensitivity to other stress factors is unknown.
12	Seaweed bay and shallows	• The category shares the sensitivity of 10 and bears additionally the sensitivity of the seaweed and the animals which feed from plants, look for shelter and use seaweed for breeding (e.g. dugongs and turtles).
13	Coral reefs	• Coastal coral reefs receive coastal discharges. • Coral reefs are the basis of a species rich community of which the species have different sensitivity. • To range the reefs among the subsystems of highest sensitivity means to regard the community members of the highest sensitivity, e.g. fish schools.
14	Salt marsh	• Salt marshes share the sensitivity of the supratidal areas of category 7 and exhibit in addition the sensitivity of the macrophytes and of the animals which inhabit the salt marshes. • Under arid climate, occasional flooding and precipitates limit degradation periods. Influence of loads adds to stress by salt, dryness, irradiation, dust and grazing.
15	Mangal (mangrove flats)	• Sensitivity is assumed to be close to category 14. • The rapid decline of mangrove areas in the past argues for a high sensitivity to many impacts.

4.5.3 Environmental and dilution requirements

In 2005, Areiqaat and Hohaned investigated the optimization of the negative impact of desalination plants on the ecosystem *(AREIQAT & MOHAMED, 2005*). From the results of their assessment the following procedures which should be undertaken to assess and minimize the negative impact of the brine discharge on the natural environment are summarized below:

- Baseline data collection
  - Hydrodynamic field measurements: Including water levels, current flow velocities and directions and flow discharges.
  - Water quality measurements: Including residual chlorine, dissolved oxygen, ambient seawater temperature and salinity, pH and ammonia.

  - Biological survey: In order to evaluate the ecosystem in the area
- Develop numerical hydrodynamic flow model: Model simulates the flow pattern in the plant vicinity and configuration of the intake and outfall of the plant should be developed and calibrated with the field measurements. The model will reproduce the flow pattern which will be used as an input for the water quality model.
- Develop numerical water quality model: Simulate the water quality of the waters around the plant, as influenced by the discharges from the power and desalination plant.
- Evaluation of the water quality results: Model results evaluated against the water quality standards

4.5.3.1 Dilution calculation

The term dilution describes the process of reducing the concentration of effluent constituents by mixing the effluent with uncontaminated ambient seawater and therefore achieving acceptable concentration levels for maintaining ecosystems functioning and recreational human activities (e.g. swimming).

The required initial dilution for the concentration of conservative constituents can be estimated by the conservation of mass as follows *(DWAF, 2004)*:

$$\mathbf{S = (C_e - C_b) / (C_g - C_b)}$$

Where:

S = Required dilutions
C_e = Concentration of constituent in wastewater
C_b = Concentration of constituent in receiving marine environment (ambient concentration)
C_g = Recommended concentration (guideline)

For example if wastewater with a total Chromium concentration of 0,13 mg/ℓ is discharged into a receiving marine environment and the guideline value for Chromium is 0,008 mg/ ℓ, then the dilution of the effluent that is required is 0.13 divided by 0.008 or a factor of 16.

As long as the minimum dilution for salinity and other relative constituents can always be achieved, then the discharge will be environmentally acceptable. Normally the aesthetics, colour, temperature and dissolved oxygen of the receiving environment are not impacted by the desalination process.

Since chemicals that are used throughout the desalination process as well as chemical that is used to clean the membranes may also be discharged with the brine (refer to section 4.5.1), the MATD (minimum acceptable toxicant dilution) value for the brine stream including all chemicals should be determined. During a MATD test, an effluent sample is "created" and then tested with respect to its impact on specific sensitive marine organisms, in order to determine the required dilutions.

(DWAF, 2004)

It is recognised that point source discharges, such as municipal and industrial wastewater, and diffuse sources, such as urban stormwater run-off, can be complex mixtures that may contain unknown compounds which may act together to increase or ameliorate the toxic effects to the

receiving marine environment (RSA DWAF, 2003b). Rather than attempting to identify all the chemicals in a sample or where the toxic effects of specific chemicals are not known, toxicity tests (bioassays) using living organisms provide a useful means of determining the potential toxicity of wastewater to the marine life. In the case of complex mixtures, toxicity testing of the waste stream (also referred to as the Whole Effluent Toxicity [WET] test) is therefore very important (ANZECC, 2000a; US-EPA, 2002b).

The US-EPA (2002b) recommends that wastewater toxicity tests consist of a control and five or more concentrations of wastewater (i.e. a range of wastewater dilutions). These tests are used to estimate:

- *LC50, i.e. the wastewater concentration which is lethal to 50% of the test organisms in the time period prescribed by the test, or*
- *No-Observed-Adverse-Effect Concentration (NOAEC)*

Minimum Acceptable Toxicant Dilution (MATD), which lies between a dilution with a response which is not significantly different from a control test (NOAEC) and the highest observed effect dilution.

Since brine is typically twice as saline as the feedwater, it has greater density than the receiving marine environment and the plume will form a downward trajectory after discharge. The brine plume will tend to sink and spread along the ocean floor where mixing is much slower than at the surface, subsequently inhibiting dilution.

Accordingly, the required dilutions will be determined from the results of the MATD test together with calculations based on the conservation of mass.

4.5.3.2 Policies & guidelines

The Department of Water Affairs (DWA) has formulated an operational policy for the disposal of land-derived water containing waste to the marine environment of South Africa. The operational policy recognises a number of ground-rules that will be applied by DWA when considering a license application to discharge wastewater into the sea *(DWAF, 2004)*.

For the discharge of brine, the following groundrules are specifically important:

- *Groundrule 9: The surfzone is a sensitive area, and disposal of wastewater to the surfzone should be avoided, except where legitimate motivation can be provided.*
- *Ground rule 10: Discharge through a marine outfall is an option, provided the suitability is properly assessed.*
- *Groundrule 11 and 13: The South African Water Quality Guidelines for coastal marine waters (or site specific objectives, if appropriate) must be met beyond the initial mixing zone.*
- *Groundrule 19: Seawater, used in an industrial process on land, is classified as 'industrial wastewater', which requires a licence for disposal.*

On this basis the brine discharge is considered an industrial wastewater discharge.

- *Groundrule 24: For the disposal of wastewater to the marine environment, potential impacts on the receiving environment must be investigated.*
- *Groundrule 26: Recognised numerical modelling techniques must be applied in the scientific and engineering assessment and design of a marine disposal system, as and where considered appropriate.*

Any waste stream which is discharged to the marine environment has to comply with the South African water quality guidelines for the coastal marine waters, which varies for different beneficial uses of the receiving environment.

Different beneficial uses require different water quality guidelines. The main beneficial uses are categorised as recreational use (e.g. swimming), industrial use (e.g. ports) and the natural environment. For example seawater that is fit for maintaining the natural environment, is not necessarily also fit for swimming. Similarly, a dynamic open coastline has a higher capacity for accepting waste than a sheltered bay. The most stringent ("target") water quality guidelines for coastal marine waters are presented in Table 4.12. These guidelines apply to all marine areas, except for the faecal coliform and E-coli guidelines, which apply in areas used for contact recreation and collection of filter feeders.

Table 4.12: South Africa marine water quality guidelines *(DWAF, 1995a & 1995b & 1995c)*

Turbidity and colour	Turbidity and colour acting singly or in combination should not reduce the depth of the euphotic zone by more than 10 per cent of background levels measured at a comparable control site.
Suspended solids	The concentration of suspended solids (SS) should not be increased by more than 10 per cent of ambient concentrations.
Temperature	The maximum acceptable variation in ambient temperature is + or - 1°C.
Salinity	Salinity should lie within the range 33 to 36 units.
pH	The pH should lie within the range 7, 3 to 8,2.
Dissolved oxygen	Dissolved oxygen should not fall below 5mg/l (99 per cent of the time) and below 6 mg/l (95 per cent of the time).
Dissolved nutrients	Nutrient levels should not cause excessive or nuisance aquatic plant growth or reduce the dissolved oxygen concentrations below recommended levels (nitrite, nitrate, ammonium, reactive phosphate and reactive silicate).
Inorganic constituents	Levels should not exceed: Ammonia 600 µg N/l (NH_3 plus NH_4^+) Cyanide (CN) 12 µg/l Fluoride (F) 5 000 µg/l

	Arsenic (As) 12 µg/l Cadmium (Cd) 4 µg/l Chromium (Cr) 8 µg/l Copper (Cu) 5 µg/l Lead (Pb) 12 µg/l Mercury (Hg) 0,3 µg/l Nickel (Ni) 25 µg/l Silver (Ag) 5 µg/l Zinc (Zn) 25 µg/l Synergism (the interactive effect of numerous compounds) should also be taken into account.
Faecal coliform or E-coli (recreation)	Maximum acceptable count per 100ml: • 100 in 80% of the samples • 2000 in 95% of the samples
Faecal coliform or E-coli (filter-feeders)	Maximum acceptable count per 100ml: • 20 in 80% of the samples • 60 in 95% of the samples

A wastewater stream that is discharged into the sea adheres to these guidelines by a process of dilution which is defined in section 4.5.4.5.

The entire concept of achievable initial dilution is based on the assumption that the receiving water is continuously moving and that 'clean' water is always available for entrainment and subsequent dilution of the wastewater plume. Note that this is not always the case (e.g. estuaries and the surf zone).

4.5.4 Hydraulic and environmental design

4.5.4.1 Energy principle

The hydraulic study is based on the hydraulic energy balance for the complete system by comparing the specific energy between any two points in the system and taking into account all friction and fitting losses between two adjoining points. This balance ensures continuity of flow. (*CHADWICK & MORFETT, 1998)*

The energy principle is represented by the Bernoulli equation (Appendix A) and the continuity equitation for steady incompressible flow is used for brine discharge. Although the hydraulic energy balance can be solved manually, computer programs should be used to optimise a multi-port diffuser since the hydraulic analysis is an interactive procedure. Numerous runs are normally required to ensure:

- Even port flow distribution along the diffuser;
- The main pipe velocities are maintained to prevent deposition;
- The Froude number of all the ports are greater than one to prevent seawater intrusion; and
- Initial dilutions are achieved.

4.5.4.2 Outfall configuration

(*LE ROUX, 2005*)
The optimisation of the main pipe diameter depends on the present and future flow conditions and available or practical head (gravity or pumps). The design should ensure:

- A main pipe velocity of greater or equal to 0.7 m/s is maintained to prevent deposition of solids and provide scouring abilities;
- The maximum flows are discharged with available gravity head or cost-effective pumping, taking into account the increase in roughness during the lifetime of the outfall and all losses at fittings in the main pipe and diffuser.

4.5.4.3 Diffuser configuration

(*DWAF, 2004*)
For the optimisation of the diffuser, the following criteria must be met:

- The diffuser section should be placed perpendicular to the prevailing ocean current. If there is no one prevalent current direction, then a T-port diffuser can be used.
- Design flows must be discharged satisfactorily through the ports to ensure continuity of flow. Generally, the total cross-sectional areas of the ports should be less than 0.7 times the cross-sectional area of the main pipe at any point in the diffuser. A port diameter of less than 75 mm is susceptible to blockage.
- Maintain sufficient flow in each port to prevent the intrusion of seawater by gradually increasing port sizes towards the end of the pipe and ensure the densimetric Froude Number for each port is greater than unity.
- Ensure an even distribution of flows, through all the diffuser ports, because the flow is directly related to the achievable initial dilution, and the worst performing port will be considered as representative of the performance of the diffuser.
- Maintain scouring flows within the diffuser section.
- Optimum dilution will be obtained with diffusers discharging between 45º and 60º to the horizontal and with alternate ports directed in opposite directions.
- The distance between any two ports must be such that the plumes do not merge during the rise of the buoyant plumes.

A single port outfall is a submerged pipe with a single efflux opening typically applied in situations where:

- Ambient conditions favour rapid dilution;
- There is a very low dilution requirement; and
- Bathymetry or bottom stability precludes a diffuser.

A multi-port diffuser consists of a header pipe containing two or more ports (with or without risers) which inject a series of turbulent jets at high velocity into the ambient receiving water body. These ports or nozzles may be connected to vertical risers attached to an underground pipe or tunnel or they may simply be openings in a pipe lying on the bottom. Multi-port diffusers are typically used where:

- Maximizing dispersion is imperative;
- Effluent flow rates are greater than 1 mgd;
- Bathymetry is not extreme and

- Underwater slope stability is good.

Intermittent flow could cause sediment ingress into the diffuser pipe during times of no-flow. Appropriate port valves could be used to prevent this problem.

4.5.4.4 Achievable dilutions

Brine behaviour on discharge could vary according to local conditions (i.e. bottom topography, current velocity, and wave action) and discharge characteristics (i.e., concentration, quantity, and temperature). Dilution modelling is required to determine the achievable initial dilutions.

In the case of a buoyant effluent, the initial dilution (near field region) is the process in which a plume rises from the diffuser or an open ended pipeline to the surface of the sea. When the plume rises, seawater is entrained into the plume and thus the dilution is created. The exit jet velocity and momentum, ambient currents, buoyancy, momentum and density structure of the seawater, influence the magnitude and extend of "mixing" of the dilution.

Since the brine density is greater than the surrounding ambient water density at the discharge level, the plume is negatively buoyant and will tend to sink towards the ocean floor. The only influencing parameters of the initial dilution (S_i) of a dense plume discharged from a diffuser or an open end pipeline are the momentum flux of the jet, the angle of discharge, the ambient currents and the density structure of the receiving water column. Therefore, the achievable dilutions for a brine plume are much more limited than for a buoyant plume.

Furthermore, a buoyant effluent plume is transported away from the initial mixing zone, and entrainment and mixing with sea water is brought about by currents, wave action, eddies and shears at or near the water surface, a process generally referred to as secondary dilution which together with the chemical/biological 'dispersion' of non-conservative substances and the decay of certain organisms, can be described as the "far-field dilution process. However, since a brine plume is more dense than seawater, secondary dilution only occurs on the seabed where currents are more sluggish and dilution very limited.

Note that the design/configuration of a diffuser can only influence the plume behaviour in the near field region and subsequently the initial dilutions. Therefore, diffusers should be designed to ensure the required dilutions are achieved in the near filed region where strong initial mixing occurs.

Numerous prediction theories and techniques are available. The choice of the technique ('model') to be applied should take the following into account:

- Confidence in the 'accuracy' of the dilution prediction estimates;
- Project/client requirements and specifications and
- The control, which the engineer has on the technique ('model') and the thorough understanding of the theories that are applied.

WAMTech model *(BOTES, 2009)*

- The methods developed for dilution estimates for various ambient and diffuser conditions by The United States Environmental Protection Agency (US-EPA, 1985) were used as a basis

for the WAMTech model to predict dilutions. Refer to US-EPA (1985), a "plume element" is followed (for each modelling time step) as it gains mass due to entrainment of the ambient water, thus the characteristics of a continuous plume in a dynamic (flowing environment) can be described. In the program, the entrained mass is added to the mass of the element, calculating the mass of a new element. The density of the new element is the averages of the previous values and the entrained values, weighted by their relative masses. Horizontal momentum is conserved and the new density creates a vertical acceleration (buoyancy) on the plume element. The segment length is changed in proportion to the total velocity to conserve mass and the radius of the plume is changed to correspond to the new mass and density. The output of the plume path (trajectory) is given as values in the x- and y-planes together with the radius of the plume to provide for the visual output of the geometry of the plume.

- The hydraulic analysis of the program is based on the hydraulic energy balance for the complete system (multi-port diffuser and main pipeline) by comparing the specific energy between any two points in the system, taking into account all friction and local fitting losses between two adjacent points. Provision was also made for T-port configurations.
- The model output includes interactive visual trajectories of the plumes for all the ports of the diffuser and standard graphical outputs of the entire range of diffuser characteristics.
- A far field dilution prediction technique (An analytical prediction method developed by Brooks (1960)), for conservative and non-conservative substances has also been linked to the model for an assessment of achievable dilutions for compliance with environmental quality objectives at distant locations.
- Procedures were developed for the simulation of temporal scenarios and actual measured environmental data, for provision of a more realistic statistical overview, taking into accounts the continuously varying receiving conditions (currents and stratification) as well as varying effluent flows.
- Output options were developed to couple the data generated by the near-field model (time-series of detailed plume characteristics (rise height, plume thickness, horizontal distance and initial dilution) to 2-D and 3-D far-field hydrodynamic models, for the computation of the advection-dispersion of the effluent in the far-field domain.

Roberts & Toms *(WRC, 1992)*

With this method the achievable dilutions are determined by the following formula given by Roberts & Toms (*Guide for the Marine Disposal of Effluents through Pipelines*), formulated especially for dense effluents:

For Stagnant conditions and jet at 60°:

$S_t = 0.38F$

$S_i = 1.03F$

$S_{av} = 1.74S_t$ or $1.74S_i$

Where:

S_t = terminal height dilution on the centre line of the plume

S_i = impact dilution at the collapse of the plume to the sea bed at the centreline of the plume

F = Densiometric Froude Number (Refer to Appendix A for equation)

S_{av} = Average dilutions across the jet

For a 60° jet discharging at velocity u perpendicular to a cross-flow velocity of U_a m/s:

$S_t/F = 0.8(u_rF)^{1/2}$

$S_i/F = 2(u_rF)^{1/2}$

Where:

$u_r = U_a/u$

The nearfield formulations in the Roberts & Toms formula were derived by empirical means including laboratory tests as well as dimensional analysis *(ROBERTS & TOMS, 1987)*.

CORMIX model

In addition to the above methods, CORMIX v5.0 could be used to provide additional information on the region of impact (sacrificial zone), since the CORMIX model predicts the horizontal distance from the discharge point where the plume reaches the seafloor. The CORMIX systemcomprise of a collection of flow models which was developed from over 200 years, in order to simulate the physics of the mixing zones. The model incorporates a collection of jet-integral approaches for determining the near-field prediction for stable conditions, length scale approaches for determining the near-field prediction for unstable conditions, as well as integral and passive diffusion approaches to simulate various scenarios of near-field and far-field mixing zones.

Taking into account the numerous prediction theories and techniques available, CORMIX v5.0 is recommended highly by Doneker (*(DONEKER, JIRKA & HINTON, 2006)* for open ocean outlets due to the:

- International confidence in the accuracy of the prediction estimates of CORMIX;
- Project/client requirements: CORMIX v5.0 allows for detailed mixing analysis of dense discharges (e.g. concentrates form desalination facilities);
- The option to include detailed ambient density and current profile specification and
- The interactively displayed output results in graphical format.

4.5.5 Operation and Maintenance

(MELBOURNE WATER & DWAF 2004)

Normally the initial and future operation of the desalination plant will result in a significant increase in flow rates. Provision should be made to close and open (e.g. flange) the diffuser ports in order for the outfall system to achieve optimum hydraulic performance and required dilutions for initial and future flow rates. Should the brine composition and concentrations, or the flow rates vary from the initial design specifications, the outfall system must be re-assessed and subsequently the required operational or physical changes made.

In order to assess the impacts of the brine discharge on the marine environment, baseline data should be gathered at the site prior to construction. Subsequently, once the plant operations commences, monitoring should be done on a regular basis for a period of at least one year but preferably for up to three years. Financial provision should be made for the following monitoring programs as part of the overall project costs to confirm the accuracy of the predicted environmental impact of a desalination plant's brine discharge:

Pre-Operational Monitoring

- Ambient salinity and other constituent concentrations at the discharge location

- Marine organisms in the area of the outfall
- A long-term inventory of marine organisms in the microlayer

Pre-discharge Monitoring

- The brine stream should be monitored before discharge, to make certain the salinity and other constituent concentrations in the brine is at the required level, and subsequently ensure the necessary regulations are met in order to protect the marine environment and human health.

Post-discharge Monitoring

- Light penetrating tests in the water column in order to determine any impact on the benthos
- Measurements of impacts on habitat in the microlayer
- Measurements of impacts on fish in the water columns
- Plume trajectory evaluation of depth, temperature, salinity, and density
- Nontoxic dye tests to measure dilution
- Sampling of sediments
- Measurements of salinity at various offshore sampling locations

4.6 CONCLUSIONS OF LITERATURE STUDY

As mentioned previously, available literature of the marine components of large seawater intake and discharge systems for seawater desalination plants was obtained and reviewed in order to formulate an overall design approach.

As introduction, the currently available desalination technologies, together with each technology's typical feedwater requirements and discharge composition were summarized in Section 4.1. The type of desalination technology influences the required extraction volumes and quality, as well as the brine stream's volume and salinity. Therefore, it is important for engineers to understand the various technologies and subsequently the impact that the intake and discharge systems would have on the plant, and vice versa. Although thermal technologies still generate the largest volume of desalinated seawater, there are currently more RO membrane plants worldwide and growing exponentially. Therefore, this thesis tends to focus more on the typical marine components for RO plants.

The main marine components for offshore intake and discharge systems for large scale seawater desalination plants, include an offshore intake headworks, marine pipes which transports the feedwater to the desalination plant and a brine outfall pipeline with diffuser which discharges the brine effluent beyond the surf zone and ensures suitable dilution.

A number of different designs incorporating various shapes and sizes, screening technologies, materials, operational and maintenance procedures and location of the intake and brine outfall with regards to the coastline, plant and each other has been developed. However, from the literature study, it was concluded that the location of the marine components, as well as the detailed design requirements and operational procedures are very project and site specific. Therefore, the literature study rather aims to provide a summary of the required information, studies, regulations and considerations (physical as well as environmental) which need to be taken into account when designing the marine components, together with the available technologies currently available, than specifying which type of technology or configuration would be the most feasible.

5 ASSESSMENT OF EXISTING DESALINATION PLANTS

Selecting the most feasible type of intake structure and outfall system depends greatly on the type of desalination plant, the required feedwater volume and quality as well as site-specific environmental requirements and physical processes. In order to get a better understanding of the type of technologies used in practice and how they affect the marine components, nine seawater desalination plants were selected worldwide and assessed. The specific plants were selected on the following criteria and their locations shown in Figure 5.1:

- Selection of case studies representative of membrane, thermal and combination desalination technologies;
- Large capacity plants;
- Plants located worldwide;
- Combination of intake structures (mostly Direct intakes) and brine discharge types and
- Assorted preliminary –, pre-treatment and operational processes

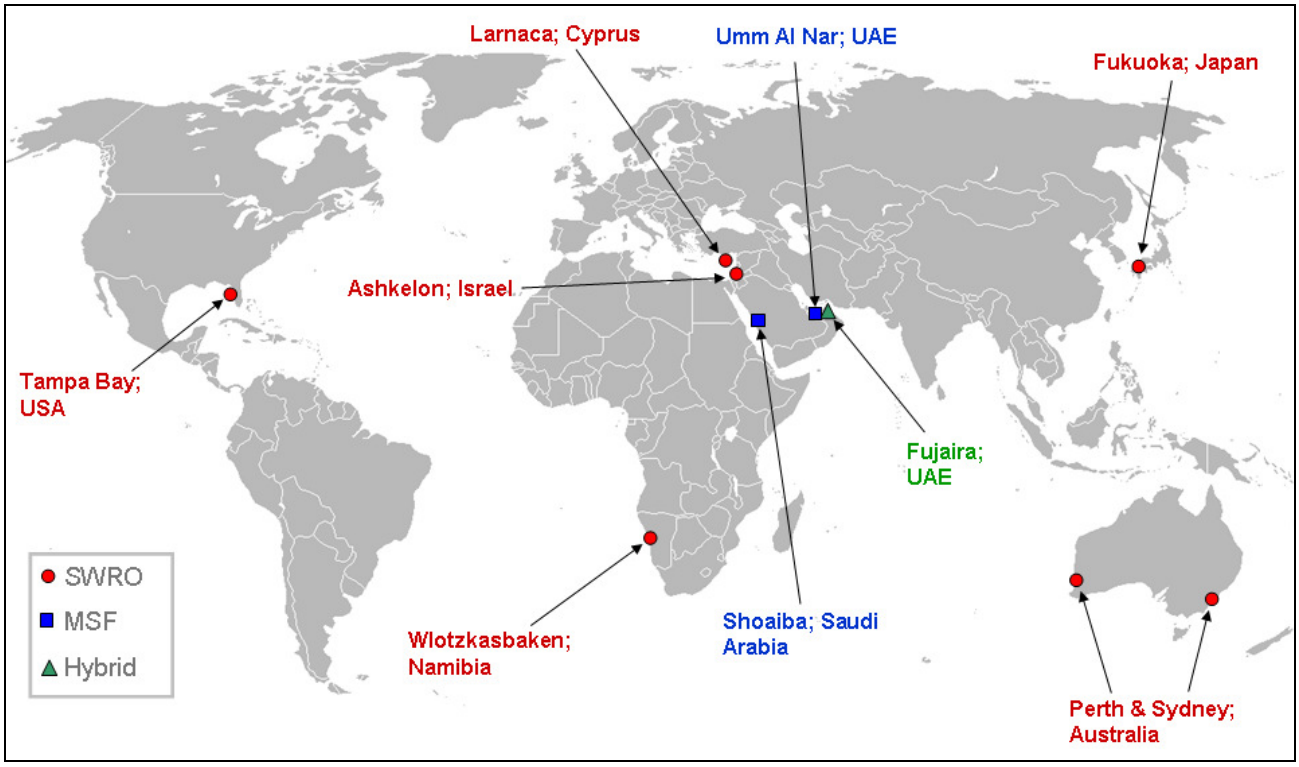


Figure 5.1: Location of desalination plants investigated

5.1 CYPRUS: LARNACA SWRO DESALINATION PLANT

In 2001 a seawater reverse osmosis desalination plant was completed on the South-East coast of Cyprus near the Larnaca airport. The plant was located approximately 500 metres from the shoreline due to environmental as well as land ownership concerns. *(www.water-technology.net)* Table 5.1 lists general information of the project:

Table 5.1: Cyprus - Project information

Client	Water Development Department of the Ministry of Agriculture, Cyprus
Owner operators	Larnaca Desalination Partners and IDE Technologies and Oceana Advanced Industries JV
Plant/Process type	Seawater Reverse Osmosis
Total plant output	54,000m³/day, 18 million m³/year, 0.6 m^3/s
Construction commenced	December 1999
Construction completed	March 2001
Project cost	US$ 47 million

Figure 5.2: Locality and layout of Larnaca Desalination Plant, Cyprus

A direct sub-surface intake structure was installed at a water depth of 11m and approximately 1.1km offshore to ensure clean seawater feed for the desalination plant as listed in Table 5.2. The entire intake pipeline consisted of 2 pipe sections which were assembled together with a flange connection. Four vertical pumps located onshore deliver the seawater to a flocculation chamber before it is pumped to the plant's membranes.

Table 5.2: Cyprus - Seawater intake configuration *(STANIMIROV, 2009)*

Offshore distance	1.1 km
Intake type	Direct sub-surface
Intake depth	11m water depth
Number of intake pipes	1 (consist of two sections)
Intake pipe material	HDPE (SDR26 / PN6.4)
Intake pipe diameter (OD)	1.2 m

After passing through the reverse osmosis process, the brine collects in an equalization tank where any foaming is broken down before it flows under gravity to a storage facility located 750 metres inland from the shore. From there it is discharged 1.5 km offshore at a water depth of 18m.

The brine stream concentration is almost double that of seawater and contains certain chemicals used during the pre-treatment process and for cleaning the membranes. The impact to the marine environment is mitigated by setting limitations to the type and amount of chemicals used in the pre-

treatment process and the membrane maintenance processes. Furthermore, the diffuser configuration was designed in order to achieve the optimum dispersion of the salts.

Table 5.3: Cyprus - Brine outfall configuration *(STANIMIROV, M)*

Offshore distance	1.5 km
Discharge depth	18 m
Outfall pipe material	HDPE (SDR26 / PN6.4)
Outfall pipe diameter (OD)	1.0 m

Monitoring is done on a regular basis to determine any changes to the marine environment and subsequently take remedial measures if required. After the first five years of operation, monitoring indicated that the effects are negligible and the impact limited to an area with a radius of 200 metres around the point of discharge. *(TSIOURTIS, 2004)*

The pipe sections were manufactured in a factory in Norway (Pipelife Norge) and towed by tugboat to Larnaca. Only the first 300 m (closest to the shore) of the pipeline were trenched, while the rest was placed directly on the seabed. During construction, approximately 120 000m^3 of sand and rock was removed during the under-sea excavation. The pipes were towed from Norway to Cyprus and installed during the spring of 2000. A submersible pontoon, developed by Oceana was used for attaching the concrete weights on the pipelines. Refer to Appendix C for photographs of the plant and pipelines during construction.

5.2 JAPAN: FUKUOKA SEAWATER DESALINATION PLANT

During 2005 the largest seawater desalination plant in Japan was completed and is located on the Northern area of Hakata Bay in the Fukuoka District (general details listed in Table 5.4).

Table 5.4: Fukuoka - General project information

Facility name	Mamizu Pia Seawater Desalination Facility
Plant/Process type	Seawater Reverse Osmosis
Total plant output	50 000 m^3/day, 0.6 m^3/s
Construction commenced	2001
Construction completed	June 2005
Project cost	US$ 440 million

The overall system comprises of an indirect sub-bottom intake (infiltration intake) constructed approximately 1.2 km offshore (outside the bay) and produces clean seawater for the reverse osmosis membrane process at maximum rate of 103 000 m^3/day. The brine is mixed with treated water from a nearby water treatment works before being discharged on the Southern shore in the Hakata Bay as illustrated in Figure 5.3. Although infiltration intakes which can extract at such a high rate are very expensive, the system was selected due to the relative low maintenance requirements and sustainability with regards to marine fouling of the sub-surface pipes.

Figure 5.3: Locality and layout of Fukuoka Seawater Desalination Plant, Japan

In order to collect clear seawater, the water level of the intake tank is lowered below sea level by onshore intake pumps. The difference in water level creates a flow which causes the sea water to flow into the intake pipe. The seawater is drawn up at a very slow rate to ensure sand is not pumped up *(HAMANO, 2004).*

Table 5.5: Fukuoka - Seawater intake configuration *(HEATON, 2005)*

Offshore distance	1.2 km
Intake type	Gravity infiltration (pipe network is buried 3 m under a sandy seabed)
Intake depth	11.5 m water depth
Number of intake pipes	1
Intake pipe material	HDPE
Intake pipe diameter	Branch pipe: 0.6 m Main pipe: 1.8 m

The layout of the intake system is illustrated in Appendix C and was designed to collect seawater without installing structures which could affect the surrounding environment. Also refer to Appendix C for additional photographs and treatment procedures.

The sand layer by itself acts as a biological treatment filter system and prevents marine organism from entering the intake pipes. Furthermore, the system presents no hazard to shipping navigation and is not affected by wave action.

After the filtration process, the brine stream, which has about double the salt concentration than that of seawater, is diluted 50:50 with secondary treated municipal sewage before being discharged to the sea in order to minimize the effect on the marine environment.

5.3 SAUDI ARABIA: SHOAIBA DESALINATION PLANT

Saudi Arabia is one of the world's largest producers of desalinated water, meeting approximately 70% of the country's drinking water, urban and industrial requirements with desalinated water.

In 2003, the second phase of a Multi-Stage Flash Distillation (MSF) facility in Saudi Arabia near Shoaiba was completed *(Refer to Table 5.6)*. At that time, the facility was ranked as the largest in the world. The desalination plant comprises a desalination facility, storage tanks, a pump station as well as pipelines to transport the product water.

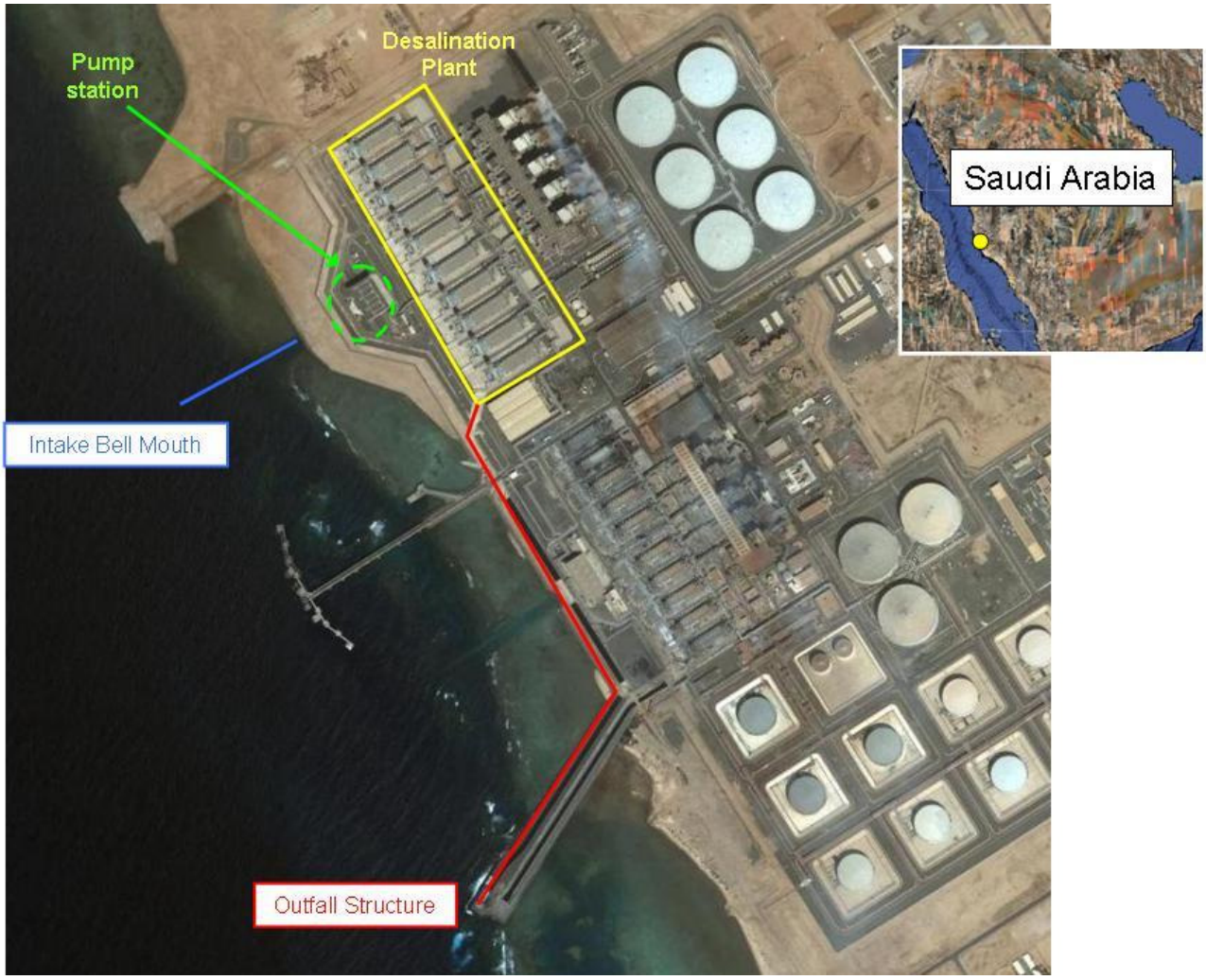


Figure 5.4: Locality and layout of Shoaiba Desalination Plant, Saudi Arabia

A power station was constructed adjacent the desalination plant which forms part of an integrated operation, providing the desalination facility with steam to heat its seawater distillers, while reducing its own demands for cooling as shown in Figure 5.4.

Table 5.6: Shoaiba - General project information *(www.water-technology.net)*

Client	Saline Water Conversion Corporation (SWCC)
Plant/Process type	Multi-Stage Flash Distillation (MSF)
Total plant output	Phase 1: 74,000m³/day, 0.9 m^3/s Phase 2: 450,000m³/day, 5.2 m^3/s
Construction commenced	1997
Construction completed	March 2003 (Phase 2)
Project cost	US$ 1.06 billion

The feed-water is abstracted through a "bell mouth" type intake structure and the pipes are connected to the intake pump station which consists of a sump, a band screen, a bar screen and vertical pumps. One of the reasons that it was decided to incorporate a MSF process, was due to the fact that extensive pre-treatment is normally required of raw seawater by other technologies such as reverse osmosis.

Steam from the adjacent power plant is used to heat the intake seawater which then boils rapidly and "flashes" into steam, after which the steam is then condensed and cooled.

Table 5.7: Shoaiba - Seawater intake configuration (KWI HYURK, 2009)

Offshore distance	500 m
Intake type	Direct, sub-surface intake: Bell mouth (precast concrete)
Number of intake pipes	3
Intake pipe material	GRP
Intake pipe diameter	3.7 m

The brine is discharged through a open channel to sea by gravity (Appendix C).

Table 5.8: Shoaiba - Brine outfall configuration (KWI HYURK, 2009)

Onshore pipe length	1.0 km
Offshore length	0.3 km
Discharge depth	2.5 – 4 m
Discharge structure	Outlet weirs

Measures were taken to ensure the recirculation effect of the thermal and salinity impact will be within the required ecological limits and the coral reef was protected from the brine concentration as well as construction activities.

5.4 ABU DHABI: UMM AL NAR DESALINATION PLANT

Located on the Umm Al Nar Island, about 20 km to the east of Abu Dhabi city, the existing power plant is co-located with five 57,000m³/day Multi-Stage Flash Distillation (MSF) units which desalinated seawater. Refer to Table 5.9 for general details regarding the project.

Table 5.9: Umm Al Nar - General project information *(www.water-technology.net)*

Client	ADWEA (60%), IP consortium (40%)
Plant/Process type	Multi-Stage Flash Distillation (MSF)
Production capacity	Original plant output: 284,000m³/day, 3.3 m^3/s New plant capacity expansion: 114,000m³/day, 1.3 m^3/s
Construction commenced	July 2000
Construction completed	Original plant: August 2002 New plant: Third quarter in 2006
Project cost	US$ 2.1 billion

Since the seawater near the Umm Al Nar area contains a significant amount of silt and fine sand, the intake pump station was constructed with a sedimentation settling basin in order to remove particles harmful to the pumps, pipes and evaporators. The settling basin consists of four bays with a length of approximately 25 metres and the pump station consists of 6 vertical mix flow pumps, a band screen and a bar screen (refer to Figure 5.5).

Figure 5.5: Locality and layout of Umm Al Nar desalination plant

The brine is discharged to the sea by gravity through a concrete box culvert that is located inshore. The outfall consists of a weir structure so that the brine flows over the weir into relative shallow water. *(Refer to Appendix C for additional photographs.)*

Table 5.10: Umm Al Nar - Brine outfall configuration (KWI HYURK, 2009)

Onshore length	1.0 km
Discharge location	Shoreline
Discharge depth	HHWL
Discharge structure	Open channel outfall

5.5 ISRAEL: ASHKELON DESALINATION PLANT

(www.water-technology.net)

On the western coast of Ashkelon (Israel), the construction of a 110 mil m^3/year reverse osmosis desalination plant was completed in 2005.

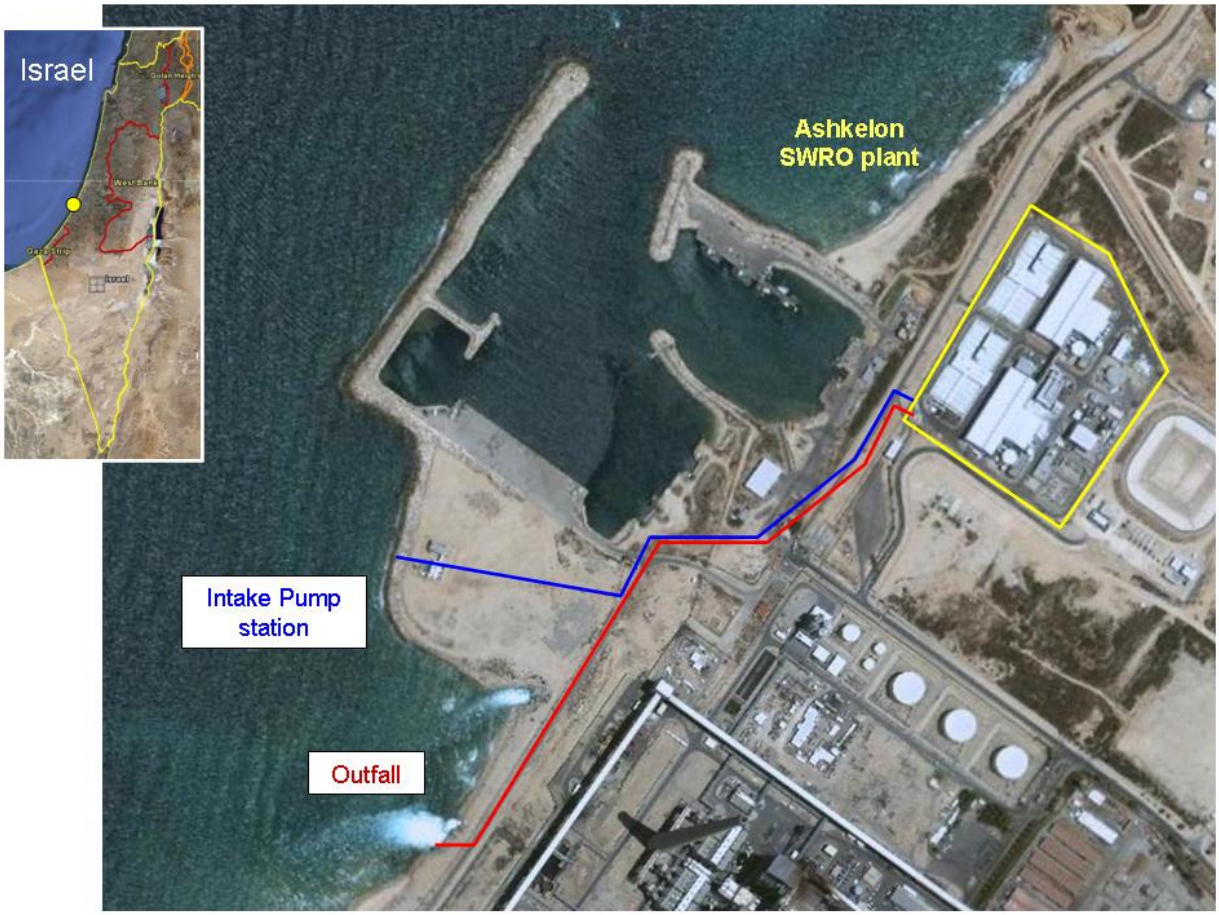


Figure 5.6: Locality and layout of Ashkelon SWRO Desalination Plant, Israel

At the time of construction, it was rated as the largest seawater reverse osmosis plant in the world. Refer to Table 5.11 for the general information of the project.

Table 5.11: Ashkelon - General project information *(www.water-technology.net)*

Client	Israeli Ministries of National Infrastructure and Finance
Plant/Process type	32 RO trains on four floors; 40,000 membrane elements
Final total production capacity	300 000 m^3/day, 3.5 m^3/s
Construction commenced	April 2003
Construction completed	August 2005
Project cost	US$ 212million

The feed water's input temperature ranges from 15 to 30°C with a salinity of approximately 40 750 ppm TDS. In order to provide twice the capacity originally foreseen, two self-contained, identical plants have been built on the site, each contributing approximately 50 million m^3/year of desalinated water. The two plants are able to operate independently, only sharing the seawater intake, the product water treatment system and the power plant. The "shared" components were also designed to allow separate service to each of the plants.

Due to site constraints and hydro-geological limitations, a direct subsurface intake type was constructed which consists of three parallel HDPE pipes, which are simple to clean (pigging) and relatively resistant to biological growth, therefore minimising maintenance costs.

Table 5.12: Ashkekon - Seawater intake configuration *(SAUVET-GOICHON, 2007)*

Intake type	Direct, sub-surface
Number of intake pipes	3 (refer to Appendix C)
Intake pipe material	HDPE
Intake pipe diameter	1.6 m
Intake pipe length	1 km

The installation of the three intake pipes are illustrated in Appendix C. The intake pumping station is equipped with 5 vertical pumps. From the pumping station, raw seawater flows to the pre-treatment facilities through two separate lines, ensuring the plant can continue to operate at half-capacity in the event of blockage or failure in one of the pipelines.

The most environmental and financial feasible method to discharge the brine stream from the desalination plant, was to dilute it with the hot water which is discharged from the adjacent Ruthenberg Power Station to the ocean. Taking into account the operation schedule of the power station and locating the brine discharge pipe from the desalination plant next to the outlet of the power station, a dilution ratio of at least 1:10 between the brine and cooling water of the power plant is achieved.

5.6 UAE: FUJAIRAH DESALINATION PLANT

Currently, the Fujairah plant, completed in 2003 is the largest desalination hybrid plant in the world which consists of MSF units coupled with the adjacent power plant, as well as a reverse osmosis component. Refer to Table 5.13 for project details of the Fujairah plant.

Table 5.13: Fujairah - General project information

Client	Fujairah Asia Power Company (FAPCO)
Plant/Process type	Multi-Stage Flash Distillation (MSF) & Seawater Reverse Osmoses (SWRO)
Production capacity	MSF: 454,000 m^3/day (62.5%), 5.3 m^3/s SWRO: 170,500 m^3/day (37.5 %), 2.0 m^3/s
Construction commenced	June 2001
Construction completed	2003
Project cost	Not known

Due to power saving considerations, a thermal desalination plant is normally jointly developed with a power station in order to reduce power costs. However, since there is only a high demand of power in Fujairah during the summer, the power plant runs on only half-capacity six months of the year. However, a reverse osmosis plant uses 2.5 to 3 times less energy and therefore, combining the two technologies ensured that the overall system is more flexible and can sustain the electricity demand when there is a mismatch between the water and electricity demand.

Figure 5.7: Locality and layout of Fujairah Seawater Desalination Plant

The direct seawater intake system is located about 400 meters offshore and comprises of three individual circular intakes connected to GRP pipes. Approximately 133 000 m^3/h of seawater is transported through the intake pipes into the desalination plant, of which 110 000 m^3/h is pumped to the MSF plant and 22 000 m^3/h to the RO plant. The feedwater is "lifted" under a 4m negative pressure by a set of 3 self priming pumps.

Table 5.14: Fujairah - Seawater intake configuration ***(SANZ, et.al., 2007)***

Offshore distance	400 m
Intake type	Direct sub-surface
Intake screens	Cu/Ni alloy intake screens
Intake depth	10 m water depth (6 m above seabed)
Number of intake pipes	3
Intake pipe material	GRP
Pressure class	3 bar (PN)
Chlorine solution line	Yes

Only one of the three intake structures supply seawater directly to the RO plant, whereas the other two structures serve the MSF plants and the power stations. Since continuous chlorine is discharged at the extraction point of the MSF plants and power stations, the RO plant requires a separate intake stream with only intermittent chlorine shock dosing.

Table 5.15: Fujairah - Brine outfall configuration (*SANZ, et.al., 2007)*

Offshore distance	Surf-zone discharge
Discharge structure	Open channel discharge (weir)

Previous assesments (*HEATON, 2005)* found that although intermittent chlorination dosing is normally sufficient to avoid the possibility of bio fouling caused by continuous chlorination, the optimum concentrate, interval and period of discharge is essential. At Fujairah, the original specified chlorination rate which was set at 4ppm was not sufficient and was changed to a maximum of 14 ppm during a period of 4 hours. Before the chlorine dosage was increased, the plant's intake pipework required cleaning every two months due to excessive marine growth. Refer to Appendix C for additional photographs.

5.7 AUSTRALIA: PERTH SEAWATER DESALINATION PLANT

In April 2007 the construction of the largest SWRO plant which is powered by renewable energy in the world was completed (first water commence November 2006). The plant is located at Kwinana (approximately 40 km South of Perth) with a daily capacity of 150,000 m³, supplying 17% of Perth's fresh water needs. Table 5.16 lists the general project information. *(www.water-technology.net)*

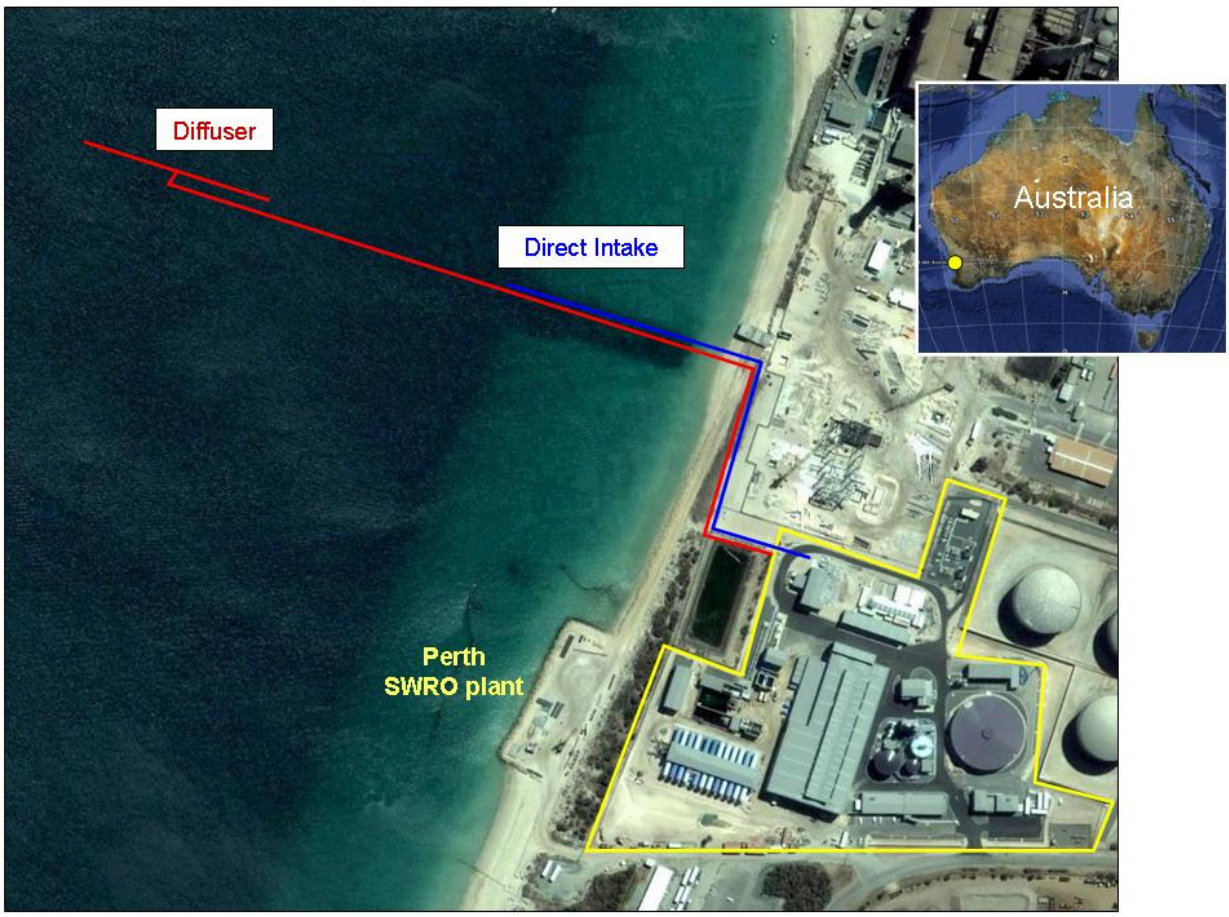


Figure 5.8: Locality and layout of Perth Seawater Desalination Plant, Australia

Table 5.16: Perth - General project information *(www.water-technology.net)*

Client	Water Corporation of Western Australia
Plant/Process type	Seawater Reverse Osmosis
Final total production capacity	150 000 m^3/day, 1.7 m^3/s
Construction commenced	April 2005
Construction completed	First water: November 2006 & Full production: April 2007
Project cost	US$310 mill

Feedwater is extracted from a direct sub-surface intake structure about 200 m offshore and the extraction flow rate at the intake headworks was designed low enough to ensure fish can easily swim against the flow. The onshore pump station is approximately 10 m below ground level (5 m below sea level) and hence the seawater flows into the pump station under its own gravity. *Refer to Tables 5.17 and 5.18 for the intake structure and pipeline configurations.*

Bar screens, with 150 mm openings is located offshore at the intake headworks in order to prevent large fish and other objects entering the intake pipe. Onshore, the seawater passes through 3 mm fine screens before entering the Intake Water Pump Station. The band screens remove any seaweed, mussels and other material (greater than 3 mm) that may enter through the intake. The screens are automatically washed down once per day, and produce approximately 0.5 m^3 of waste per day during normal operations which is macerated and discharged at a landfill facility.

Since the plant started operation, the offshore bar screens have been sufficient and normally cleaned only once a year during the annual maintenance shutdown of the desalination plant, preventing debris from being sucked into the plant. Although some build-up of sediment accumulates in the tank, it only had to be pumped clean once in the 2.5 years since operation commenced. Therefore, provision was made to seal off the pipe entrance when the tank has to be cleaned.

Table 5.17: Perth - Seawater intake configuration ***(CHRISTIE, 2009)***

Intake design flow	4 m^3/s *(measured)*
Offshore distance	200 m
Intake type	Direct, sub-surface (refer to Figure 4.25 and Appendix C)
Intake head configuration	8m high x 6m diameter (direct, sub-surface marine intake structure)
Intake depth	5 m intake level in 10 m water depth
Number of intake pipes	1
Intake pipe material	GRP
Intake pipe diameter	2.4 m
Intake pipe length	400 m
Offshore bar screen openings	150 mm

The intake pipeline of the desalination plant was not designed for pigging. Therefore, in order to minimize marine growth inside the intake pipe, provision was made to shock doze the pipe on a weekly basis with chlorine. However, the chlorine dosing was not very successful in eliminating all mussel growth inside the intake pipe and the pipe may have to be cleaned manually if the marine fouling becomes too extensive.

It was also observed that marine growth inside the intake pipe dies if the water inside the pipe is stagnant during shutdown periods. Subsequently, when the plant starts up again, the marine debris is then drawn through the pipe to the desalination plant. Therefore, it was planned to re-circulate the seawater in the intake pipe during the shutdown periods. Refer to Appendix C for additional photographs.

The plant was designed to operate continuously, drawing water via the intake structure, while the filter backwash and concentrate stream is discharged to the marine environment.

This salinity of the brine is approximately 65 ppt, with the salinity of the ambient receiving waters approximately 37 ppt. The brine is discharged under gravity via a diffuser which is designed to reduce the salinity to 0.8 ppt above ambient concentrations within a radius of 50 metres of the diffuser.

The brine is discharged at variable rates depending on the amount of product water being produced. The system contains a relatively small holding tank prior to discharge. However, the tank is not used as a buffer to discharge intermittently and can only accumulate some of the brine to be used for the backwash of the Dual Media Filters (the pre-treatment filters before reverse osmosis). Therefore, the brine is basically discharged straight out at whatever rate the desalination plant is operating at, which is normally 100% capacity.

The system was designed to ensure that the required dilution of the brine should be achieved for various operating flow rates. However, should the plant operate at a low flow rate (minimum of $^{1}/_{6}^{th}$ of full capacity), seawater is circulated through the plant and discharged with the brine stream to increase the discharge flow rate/velocity and ensure required dilutions.

Table 5.18: Perth - Brine outfall configuration *(CHRISTIE, 2009)*

Discharge design flow	2.4 m^3/s *(measured)*
Offshore distance	300 – 500 m
Discharge depth	> 15 m
Discharge structure	Diffuser
Number of ports	40
Port diameters	150 mm
Port height from the seafloor	0.5 m
Orientation of ports	60º to the horizontal
Diffuser length	160 m
Outfall pipe material	GRP
Outfall pipe diameter	1.6 m

Since the marine area is environmentally sensitive, the potential impact of the new plant on the natural environment (water) has been extensively considered with strict required monitoring of the TDS, temperature, DO and the sediment habitat in the vicinity.

5.8 AUSTRALIA: SYDNEY SEAWATER DESALINATION PLANT

Sydney's seawater reverse osmosis desalination project located at Kurnell (New South Wales) will deliver up to 15% of Sydney's water supply and scheduled for completion in December 2009. Refer to Table 5.19 for general information regarding the project.

Table 5.19: Sydney - General project information

Client	Sydney Desalination Plant Pty Ltd
Plant/Process type	Seawater Reverse Osmosis
Final total production capacity	250 000 m^3/day, 2.89 m^3/s
Construction commence	July 2007
Construction completed	October 2009
Project cost	US$ 1.75 bill

The seawater extraction and intake system will comprise of 4 intake riser type intake structures (with velocity caps), which will be connected to the main intake pipeline by 4 intake shafts *(TS-01A, 2005)* as shown in Table 5.20. The seawater intake velocities at the face of the intake facilities will not exceed 0.1 m/s in compliance with the standard EPA guidelines.

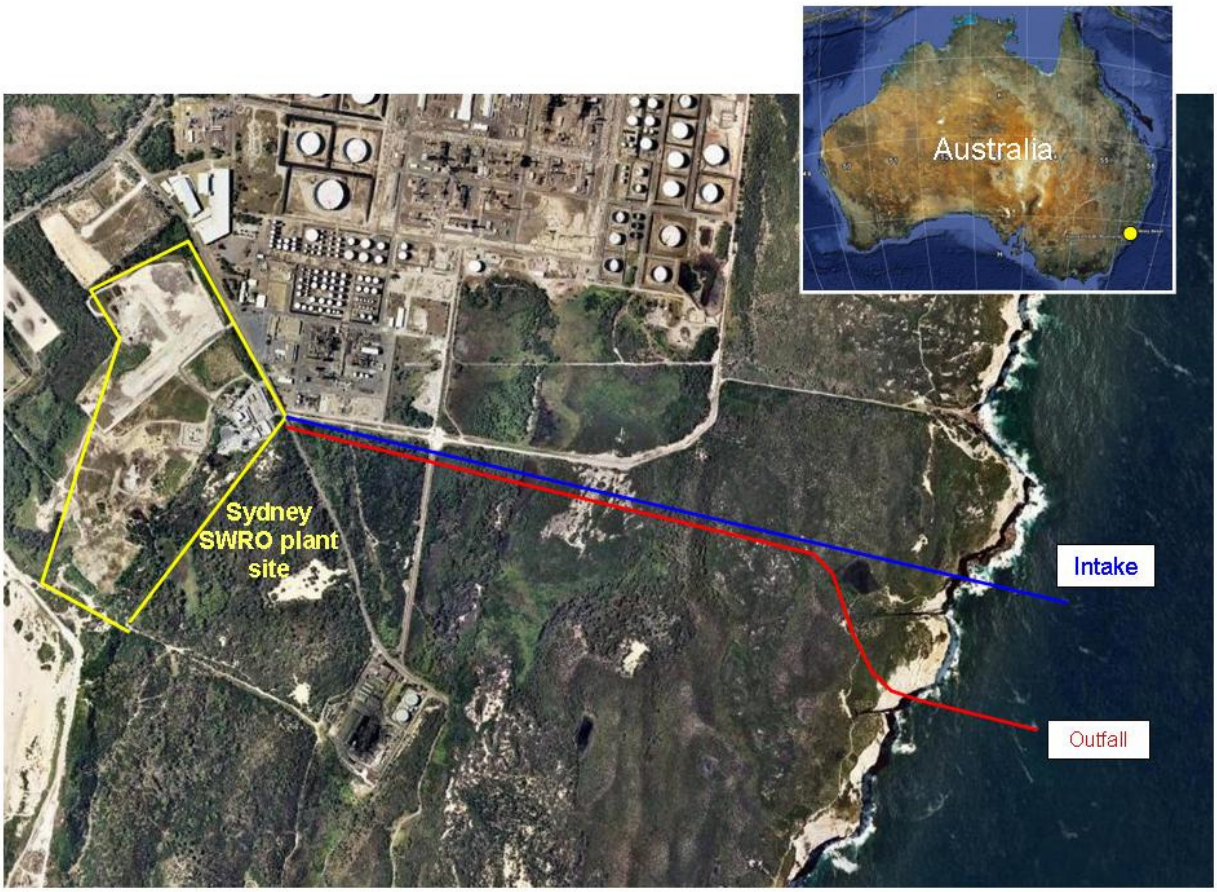


Figure 5.9: Locality and layout of Sydney Seawater Desalination Plant, Australia

Before the feedwater is pumped to the desalination plant, the seawater will undergo preliminary treatment at an on-shore screening plant, capable of screening solids from the seawater.

Table 5.20: Sydney - Seawater intake configuration *(TS-01A, 2005)*

Design flows rates	Initial design flow 250 000 m^3/day, 1.9 m^3/s Future design flow 500 000 m^3/day, 5.8 m^3/s
Intake type	Direct, sub-surface
Offshore distance	Approximately 300 m
Intake depth	20 – 25 m (on a large reef shelf)
Intake structure	4 x intake bell mouth structures
Diameter of intake shafts	1.5 m
Length of intake shafts	25 m
Number of intake ducts	1 intake tunnel with 4 risers
Intake tunnel material	Concrete tunnel lining
Intake tunnel length	2.5 km
Intake tunnel diameter	4 m approximately

Chlorine (mostly Sodium Hypochlorite) would normally be the main chemical ingredient used to clean intake pipes. Normally this would be dosed into the intake so that it is sucked into the desalination plant and the solution not discharged to the ocean.

The design of the outfall pipe, shafts and outlets is about the same as for the intake structures. The seawater concentrate will be collect at the concentrate landside chamber before being discharged offshore. The seawater concentrate will have about twice the salinity as the ambient seawater *(TS-01A, 2005)* and one or two degrees warmer. The salinity and temperature of the seawater concentrate will return to normal seawater salinity and temperature within 50 to 75 metres form the discharge point.

Table 5.21: Sydney - Brine outfall configuration *(TS-01A)*

Offshore distance	Approximately 300 m
Discharge depth	20 - 30 m
Discharge structure	4 x outlet structures
Diameter of discharge shafts	1.5 m
Length of outfall shafts	25 m
Outfall tunnel material	Concrete tunnel lining
Outfall tunnel diameter	4 m approximately
Outfall tunnel length	2.5 km

The construction of the intake and discharge structures will be supported by a large "Seafox 6" jack-up barge. The barge is 47 metres long and 35 metres wide and can be raised by four jack-up legs to about 60 metres above the seabed. (*SW455 04/08, 2008)* Refer to Appendix C for additional photographs.

5.9 USA: TAMPA BAY SEAWATER DESALINATION PLANT

Currently, the largest SWRO facility in the United States is located in Florida (Tampa bay) adjacent the power plant. Although the construction was scheduled for completion in 2003, the plant only became operational in 2006.

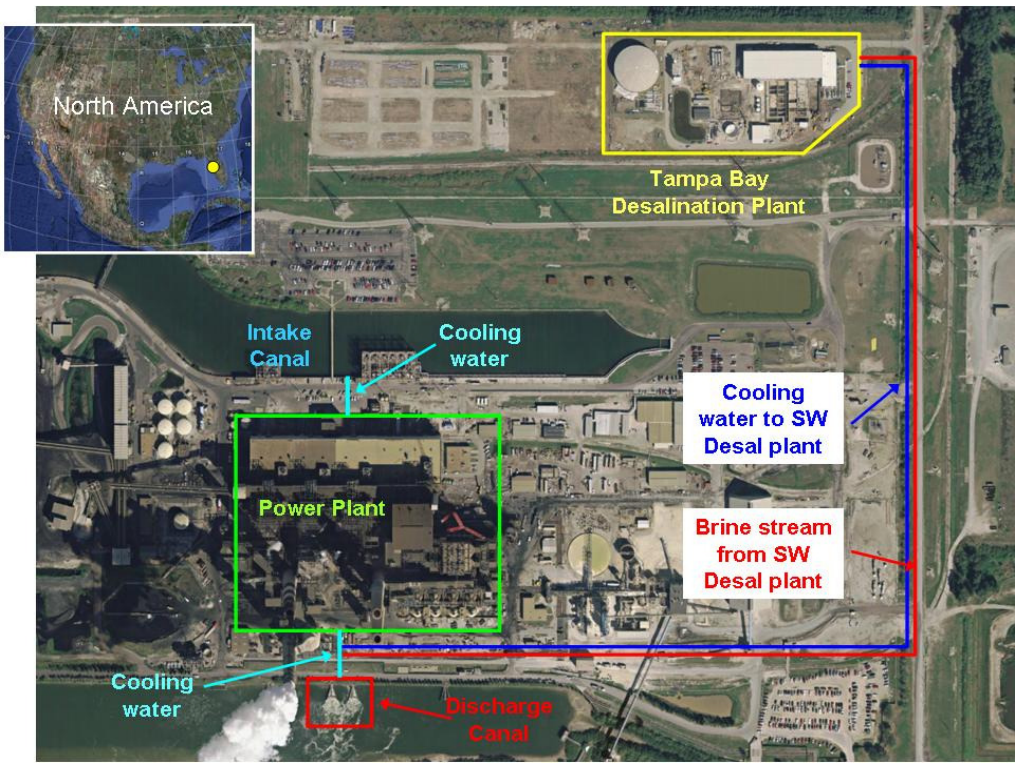


Figure 5.10: Locality and layout of Tampa Bay Seawater Desalination Plant, USA Florida

It produces an initial 95,000m^3/day, with a possible future expansion, increasing the product water by a further 37,000m^3/day. Refer to Table 5.22 for the projects general information. *(www.water-technology.net)*

Table 5.22: Tampa Bay - General project information *(www.water-technology.net)*

Owner	Tampa Bay Water, Clearwater, Florida
Plant/Process type	Seawater Reverse Osmosis
Future production capacity	132,000m³/day, 1.5 m^3/s
Construction commenced	August 2001
Construction completed	March 2003
Project cost	US$ 150 million

The seawater intake and brine discharge are connected directly to the cooling water discharge outfall of the Tampa Power Station. The power station discharge an average of 5.3 Mm^3 of cooling water per day from which the desalination facility extracts its feedwater requirements. The raw water intake is located adjacent to the neighbouring power plant's four discharge tunnels, two of which were tapped to divert around 166,000 m^3/day of the cooling outflow into the intake structure.

Fine-mesh screens (Ristroph screens) with 0.5 mm openings are located at the Power Plant's intake channel to prevent the entrainment of marine organisms. Since the eighties, when the screens were installed it operated effectively as long as frequent maintenance (manual cleaning) was performed to avoid biofouling.

From the intake, the water is pumped to the pre-treatment facility. Disinfectant is added to prevent biological growth in the delivery pipeline.

About 70 000 m^3/day of concentrate brine with approximately double the salt concentration of seawater is returned to the Power Plant's cooling water stream. The brine is diluted up to 70 times with the cooling water, lowering the salinity to about 1 to 1.5 percent higher than the ambient salinity in Tampa Bay. The above cooling water mixture is then discharged through a canal, diluting the concentrate water even more. Monitoring results of the ambient salinity during the plant's operation confirmed that no measurable changes in salinity were detected in Tampa Bay. The desalination facility's concentrate (brine stream) is discharged to the power plant's cooling water outfalls.

After the first two months of operation, the plant's filters and desalinating membranes started to clog up too quickly since the pre-treatment system could not filter particles out of the seawater sufficiently and subsequently the following modifications were made to the pre-treatment, RO and port-treatment systems: *(www.tampabaywater.org) & (DESALINATION ISSUES ASSESSMENT REPORT, 2003)*

- Additional screens were installed to remove incoming debris from the source water;
- An additional cooling water pump with variable speed capabilities were installed which enabled the water temperatures at the intake to be maintained;
- Prior to filtration, the water were rapidly mixed with treatment chemicals and conditioned to ensure the suspended solids clump together and subsequently improve the removal thereof;
- The sand filtration system was converted from dual stage to single stage;
- Diatomaceous earth filters were installed to further clean the seawater after coagulation, flocculation and sand filtration and
- The RO membrane cleaning system was redesigned to ensure more effective cleaning of the membranes.

Subsequently, pre-treatment problems resulting in excessive fouling could be successfully addressed.

5.10 SUMMARY OF EXISTING DESALINATION PLANTS

Table 5.23 summarizes the technical relevant information to this study. The selected plants were mainly RO plants with direct offshore subsurface seawater intakes. The water depths of the seawater intake headworks were all in the order of 10 meters and the brine discharge location varied from discharge at the shoreline to 18 meters water depth. Onshore fine screening by means of rotating band screens located in a pump sump is employed in some the plants.

Table 5.23: Summary of desalination plant case studies

Country	Cyprus	Japan	Saudi Arabia	Israel	Abu Dhabi	UAE	Australia	Australia	USA
Location	Larnaca	Fukuoka Kyushu	Shoaiba	Ashkelon	Umm Al Nar	Fujairah	Perth	Sydney	Tampa Bay, Florida
Completion	2001	2005	2003	2005	2002	2003	2007	2010	2006
Desalination process	SWRO*	SWRO	MSF**	SWRO	MSF	MSF & SWRO	SWRO	SWRO	SWRO
Capacity	54 000 m³/day 0.6 m³/s	50 000 m³/day 0.6 m³/s	450 000 m³/day 5.2 m³/s	300 000 m³/day 3.5 m³/s	287 000 m³/day 3.3 m³/s	454,000 m³/day (MSF) 5.3 m³/s 170,500 m³/day (RO) 2.0 m³/s	150 000 m³/day 1.7 m³/s	125 000 m³/day 1.9 m³/s (initial) 500 000 m³/day 5.8 m³/s (future)	132 000 m³/day 1.5 m³/s
Project cost	US$47 mil	US$ 440 mil	US$1.06 bill	US$212 mil	US$2.1 bill	NK***	US$ 310 mil	US$ 1.75 bill	US$150 mil
Intake									
Type	Direct: Sub-surface	Indirect: Infiltration	Direct: Concrete bell mouth	Direct: Sub-surface	Direct: Surface cooling system	Direct: Sub-surface & cooling system	Direct: Sub-surface	Direct: Sub-surface	Direct: Surface cooling system
Distance offshore	1 km	1.2 km	500 m	1 km	Shoreline	400 m	200 m	300 m	Shoreline
Depth	11 m	11.5 m	NK***	NK***	Surface intake	10 m	10 m	Approx. > 10 m	Surface intake
Pipe/conduit length	1 km	1.2 km	500 m	3 x 1 km	NA****	3 x 400 m	200 m	2.5 km	NA****
Pipe/conduit material	HDPE	HDPE	GRP	HDPE	NA****	GRP	GRP	Concrete	
Pipe/conduit diameter	1.2 m	1.8 m (main)	3.7 m	3 x 1.6 m	NA****	NK***	2.4 m	4 m	
Discharge									
Type	Diffuser	Diluted 50:50 with WWTW effluent	Open-end	Open-end	Open-end	Open-end	Diffuser	Diffuser concept	Blend back into power plant's cooling system
Distance offshore	1.5 km		300 m	Shoreline	Shoreline	Shoreline	300 – 500 m	300 m	
Depth	18 m		2.5 – 4 m	Surface	HHWL	Surface	> 15 m	Approx. > 10 m	
Pipe/conduit e material	HDPE		NA****	NA****	NA****	NA****	GRP	Concrete	NA****
Pipe/conduit diameter	1.0 m		NA****	NA****	NA****	NA****	1.6 m	4 m	NA****
Preliminary treatment									
On-land screenin	NK***	NO	YES	No	Yes	NK***	Yes	Yes	NK***
Screening	Onshore rotary screen: remove particles > 2mm	"Sand" intake acts as biological filter system	Onshore band screen & Bar screen at pump station	Offshore bar screen at intake	Onshore band screen & Bar screen at pump station	Bar screen at intake	150 mm bar screens offshore and 3 mm band screens on land	Bar screens at intake	Fine mesh screens (0.5 mm) located at Power Plant's intake channel
Pre-treatment	Flocculation Gravity filters	Ultra-filtration	NA****	Filtration	NK***	One stone filter Two sand filters	Flocculation Filtration	Filtration	Filtration
Post-treatment	Add $CaCO_3$ Adjust pH Chlorinate	Re-mineralized	Re-mineralized	Re-mineralized	NK***	NK***	Add lime, chlorine and fluoride	NK***	Re-mineralized

*Seawater Reverse Osmosis
** Multi-stage Flash
*** Not Known
**** Not Applicable

6 CASE STUDY: TREKKOPJE

Detailed design information of the marine components of a large scale desalination plant was obtained in order to use it as a basis in this thesis to compare alternative designs for specific components and subsequently contribute towards the overall design approach.

The Trekkopje desalination project, near Swakopmund on the Namibian coast is scheduled for completion beginning of 2010 and was selected for this purpose.

Areva Resources Namibia is developing a uranium mine project in Namibia and water required to operate the mine will be sourced from the reverse osmosis desalination plant. The desalination plant will initially produce 20 million m³/year of treated product water, but it was designed to operate for a future capacity of 45 million m³/year.

The extraction system comprises a direct sub-surface intake located 1.2 km offshore in 10 metre water depth, two intake pipelines and a pump station on land. The brine effluent will be discharged via an ocean outfall with the diffuser located 700 metres offshore in 6 metre water depth.

Design criteria, specialist studies which were undertaken by the designers, WSP Africa Coastal Engineers *(WSP, 2009)*, and the final design concept are summarized in the following sections.

6.1 ENVIRONMENTAL CONDITIONS

Various studies were undertaken by WSP *(WSP, 2009)* to determine the physical and environmental characteristics in order to determine the most feasible pipe route and serve as input for the physical and hydraulic design of the marine structures. In summary, the following design criteria resulted from the assessment of the site-specific conditions:

- The intake and outfall works will be relatively exposed to wave attack;
- The nearshore sea-bed slope is relatively flat;
- Large areas of exposed bedrock is present, especially close to the shoreline;
- The maximum tidal variation is approximately two metres;
- Average wind speeds of about 10 m/s are common most of the year;
- Current speeds are in the range of 0.3 m/s to 0.5 m/s; and
- The 1:100 design wave height and period was determined as 9.5 metres and 17.9 seconds.

A key concern during the design phase was the potential to extract large volumes of sediment with the feedwater into the intake pipelines. Results from the specialist study concluded that the sediment concentration are very high near the seabed and decrease rapidly with elevation towards the ocean surface. Subsequently, it was decided to locate the intake in 10 metre water depth, with the extraction point itself located 5 metres above the seafloor.

6.2 SEAWATER INTAKE STRUCTURE AND PIPELINES

The intake system for the Trekkopje project was designed with the intake end directly connected to the pumps by two intake pipelines. The schematised seawater abstraction system is presented in Figure 6.1.

6.2.1 Intake pipelines

Since seawater intake pipelines are prone to marine growth which could affect the hydraulic performance and subsequently the sustainability of the entire system, significant precautions were taken to minimize marine growth inside the pipes as much as possible by lining the inside of the offshore section of the pipe with anti-foul paint and providing chlorination dosing pipes at the intake to mitigate marine growth.

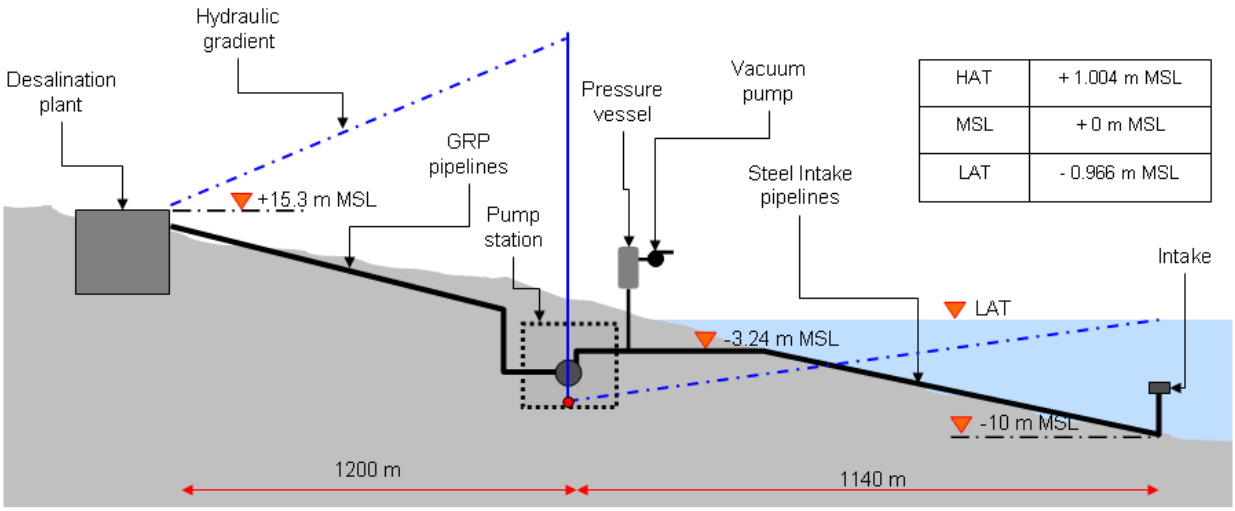


Figure 6.1: Schematic illustration of Trekkopje's intake system

The entire head required due to friction- and minor losses, as well as the entrance head loss through the screens, was calculated with roughness heights of 0.06 mm and 0.1 mm for a "new" and "old" pipe conditions. The results are presented in Table 6.1

Table 6.1: Trekkopje - Headlosses losses in each intake pipe

Roughness Height	k_s = 0.06 mm (New pipe)		k_s = 0.1 mm (Old pipe)	
Pipes in operation	1 pipe	2 pipes	1 pipe	2 pipes
Design intake flow rate per pipe	2.0 m^3/s (initial flow)	2.1 m^3/s (future flow)	2.0 m^3/s (initial flow)	2.1 m^3/s (future flow)
Total headloss	2.4 m	2.6 m	2.5 m	2.7 m

A glass epoxy lining was painted inside the steel pipe for corrosion protection. It was decided to incorporate a roughness height value (ks) of 0.1 mm for the glass epoxy lining of an old pipe, since the typical ks value for a new glass epoxy lining is in the order of 0.003 *(CHADWICK & MORFETT, 1998)*, and used to calculate the friction losses for Trekkopje's intake pipes. However, the ks value can increase dramatically due to excessive marine growth inside the pipes. Therefore extensive mitigation measures were taken to minimize marine growth. The marine growth inside intake pipelines of seawater extraction systems for abalone farms in South Africa were investigated to

obtain an indication of the expected growth rate. It was concluded that the growth is normally limited to the intake headworks itself and the first few meters of the offshore end of the pipelines. Therefore, in addition to the epoxy lining, an anti-foul lining was also painted on the intake chambers and the first offshore 100 metres of the intake pipes. Together with the above, a chlorine gas / freshwater mixture will be dosed at regular intervals at the intake structure minimize marine growth.

However, based on regular monitoring at South African abalone farms, it was concluded that no marine growth will occur inside pipelines with flow velocities of 3 m/s. For Trekkoje's design, the size of the intake pipelines was determined considering the hydraulic losses for the flow capacities and the available pump suction capacity. Also the installation of such pipe sizes and number of pipes had to be confirmed. Since it has been verified from common practice that higher velocities minimise marine growth, it was recommended to the Client to operate only one of the two pipelines during the initial stages of operation when flows are less than the capacities and thereby maximise the intake pipeline velocity.

In addition, the intake system was also equipped with a pipe pigging system which could be used to clean the pipes.

A pressure transient analysis on the seawater abstraction system was performed to establish the influence of pump failure as well as sea wave action (especially during storms) on the pressures variations at the pump suction manifold. The results of the analysis indicated that storm waves will cause excessive pressure variation at the suction manifold of the pump station. Subsequently, it was recommended that an air-vessel/priming-vacuum-chamber should be provided at the highest point in order to absorb excessive pressure oscillations in the suction manifold of the intake pumps and the Start-Up of the pumps should be done in a "soft" mode. *(BOSMAN, 2008)*

In order to protect the marine pipelines against wave action and sediment processes, the pipelines had to be buried through the surf zone at a level (-3.24 metres to MSL) which would ensure the pipelines would not be exposed due to stormy seas or seasonal variations of the sediment regime. The offshore section would simply be laid on the seafloor.

Stability calculations were done at locations along the pipeline route and results indicated that the minimum structural requirement during installation of the pipe provided sufficient mass for the pipe to withstand forces which are expected during its normal installed condition. Subsequently, a 16 mm thick steel pipe was selected with a 150 mm concrete weight coat. However, to minimize the corrosion of the steel, the pipelines were lined and coated with a corrosive paint and anodes (cathodic protection) were braced on the outside of the pipelines at specific intervals.

6.2.2 Intake structure/headworks

As mentioned previously, the seawater extraction point will be some five metres below the sea surface to avoid ingress of floating matter and five metres above the sea bottom to avoid sediment intrusion. The intake structure comprises a concrete caisson type structure as intake headworks as indicated in Figure 6.2. The design of the intake structure was not only based on the extraction requirements, but also to serve as anchor during the pipe pull operations.

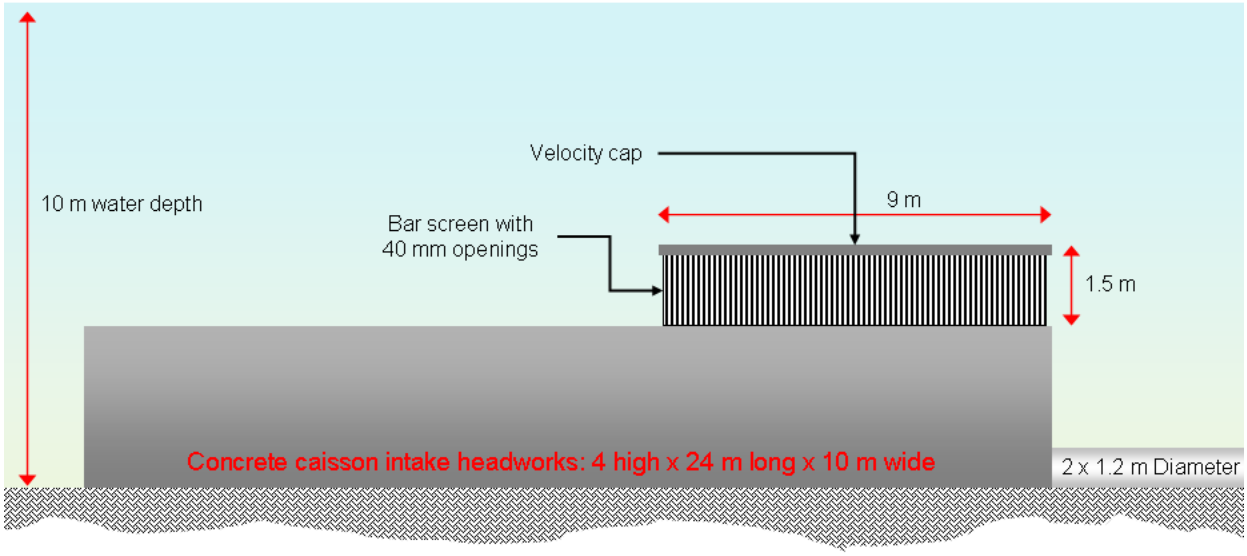


Figure 6.2: Schematic illustration of Trekkopje intake headworks

In order to minimise entrainment and entrapment of marine organisms, the entrance velocity at the screens of the intake headworks was limited to 0.15 m/s, gradually increasing the velocity as water is drawn farther into the structure towards the intake ends of the pipelines. A velocity-cap type structure (refer to section 4.3.4.3 of this thesis) was also designed to ensure horizontal flow and subsequently minimize the entrapment of marine life.

A hydraulic model study of the caisson intake structure was performed in the hydraulic laboratory of the University of Stellenbosch *(W&E US, 2008a)*. The stability of the caisson structure under wave action was tested and general hydraulics of the intake structure modelled. The objectives of the stability tests on the caisson structure had to establish whether the structure will be stable under the design wave conditions and to obtain an indication of the wave conditions under which the structure can effectively serve as an anchor while pulling the pipelines from land to sea.

In addition, physical hydraulic tests were also carried out on the caisson intake at the University's laboratory in order to determine the energy losses through the trashracks and intake headworks, the general flow patterns in the intake and the chlorination flow patterns during various flow conditions. The results showed that the pressure head losses at the intake would amount to 0.4 m for a flow of 2.1 m^3/s, which will increase to 0.42 m if the trashracks are blocked. *(W&E US, 2008b)*.

6.3 BRINE OUTFALL

The reject brine will be discharged back into the sea by means of a single discharge pipeline and diluted to the required environmental criteria by means of a diffuser at six metre water depth, located 700 metres offshore. The main design criterion of the diffuser was to achieve the required dilutions, while maintaining scour velocities in the main pipe and diffuser section.

The concentration salinity of the brine effluent will be 54 ppt and therefore about 1.5 times that of the ambient salinity which is approximately 35 ppt. Therefore, the required dilutions, based on the South African guidelines were determined as 20. The effluent's density for a salinity of 54 ppt (at 16 to 20 degrees Celsius) is approximately 1046 kg/m^3.

In order to determine the optimum diffuser configuration, a hydraulic and dilution model was applied for various configurations, until the optimum hydraulic and environmental performance was achieved. The final diffuser design consisted of a total of 20 outlet ports. During the initial operating phase of the desalination plant, half of these will be blocked off, and discharge will take place through 10 ports only.

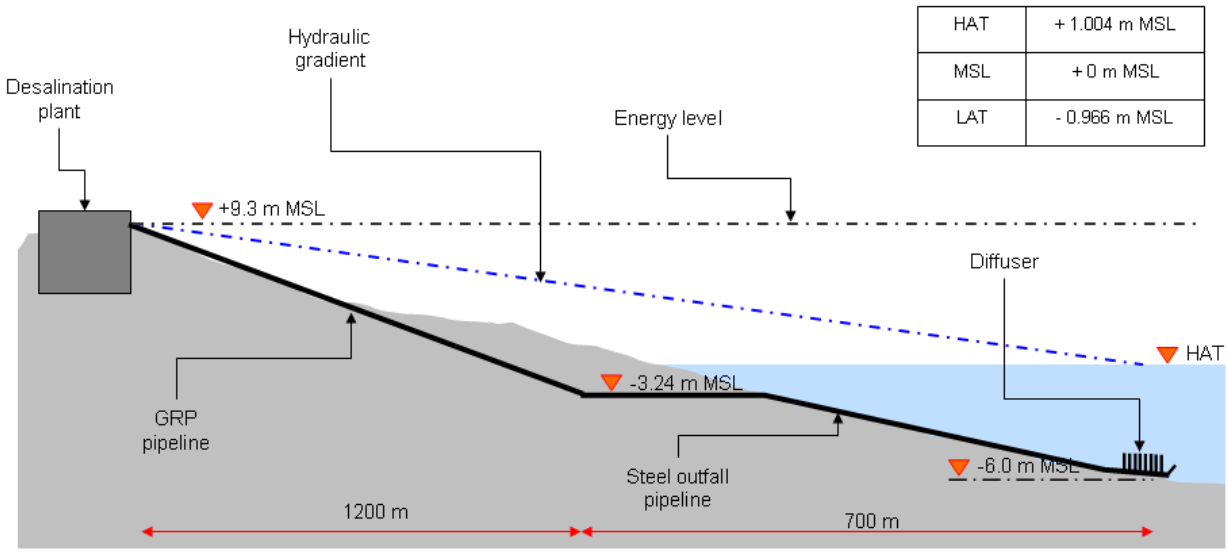


Figure 6.3: Schematic view of Trekkopje's brine discharge

The optimum diffuser configuration, based on the design discharge rates (initial and future), while ensure environmental compliance and optimum hydraulic performance of the outfall, is shown on Table 6.2:

Table 6.2: Trekkopje - Diffuser configuration for initial and future plant capacities.

	Initial plant capacity	Maximum future plant capacity
Discharge rate	4513 m3/hr	9726 m3/hr
Design discharge flow rate	1.25 m^3/s	2.7 m^3/s
Length of pipeline from CH0 to start of diffuser	600 m	600 m
Main pipe inner diameter	1.2 m	1.2 m
Length of diffuser section	60 m	60 m
Discharge depth	7.5 m	7.5 m
Tapers in diffuser	1st taper to 0.8 m 2nd taper to 0.6 m	
No. of diffuser ports	10	20
Port spacing (m)	3	3
Inner port diameter (m)	0.2	0.2
Nr of ports x main pipe diameter (m)	2 x 0.6 (offshore) 3 x 0.8 5 x 1.2 (inshore)	2 x 0.6 (offshore) 5 x 0.8 13 x 1.2 (inshore)
Port configuration	60° bend & unidirectional, in the direction of the predominant current	

The maximum headlosses in the main pipe and diffuser section was determined and the results shown in Table 6.3.

Table 6.3: Trekkopje - Headlosses in main pipe and diffuser section

Roughness height (ks)	Diffuser configuration	Design discharge flow rate (m^3/s)	Headlosses (m)		
			Pipe section (CH0 to start of diffuser)	Diffuser section	Total
0.06 mm New pipe	10 ports open	1.3	0.4	1.5	1.9
	20 ports open	2.7	1.9	1.7	3.6
0.1 mm Old pipe	10 ports open	1.3	0.5	1.5	2
	20 ports open	2.7	2.0	1.7	3.7

The required dilutions, based on the South African Marine Water Quality Guidelines, together with the brine composition were determined as 20. This means, that the brine plume had to be diluted 20 times before it reaches the required concentrations and subsequently adhere to the guidelines.

In order to determine the environmental performance of the Trekkopje outfall, the achievable dilutions were determined by the formula given by Roberts & Toms, together with CORMIX v5.0 to verify the results and to provide a better indication of the region of impact (sacrificial zone). *(Refer to section 4.5.4.5 of this document for more information regarding the Roberts & Toms formula and CORMIX model)*

Normally, the main oceanographic (ambient) process that would influence the transport of the initial dilution of an *offshore* outfall system is the offshore circulation characteristics (current speed and direction). The achievable dilutions were modelled for Stagnant (0.01 – 0.05m/s), Medium (0.05 – 0.2 m/s) as well as Extreme (0.1 – 0.4 m/s) ambient (current) velocities.

Tables 6.4 and 6.5 list the achievable dilutions determined by formula given by Roberts & Toms for an initial design flow of 1.3 m^3/s and 10 ports in operation and future design flow of 2.7 m^3/s and all 20 ports in operation .

Table 6.4: Trekkopje - Achievable dilutions for initial design flow

Ambient conditions	**Dilution @ terminal height on centre line**	
	S_{min}	S_{avg}
Stagnant	8	13
Medium	11	20
Extreme	20	34

Table 6.5: Trekkopje - Achievable dilutions for future design flow

Ambient conditions	**Dilution @ terminal height on centre line**	
	S_{min}	S_{avg}
Stagnant	8	15
Medium	13	22
Extreme	22	38

During operation of the desalination plant abnormal operating conditions may occur during which brine discharge is much less than the design flows. Normally, if possible, it would be best to provide for facilities to discharge intermittently at design flow rate, than constantly at a reduced rate.

However, the client preferred to rather discharge at a constant flow rate lower than that of the design flow rate, than constructing a brine holding tank to discharge intermittently at design flow rates. The main risks which should be considered if effluent is discharged through an outfall system at low flows (much less than the design flow rate) are the reduction in main pipe velocities and subsequently the self-cleaning ability of the pipeline, the reduction in port exit velocities and subsequently the achievable dilutions as well as the possibility that the Froude number at the ports could fall below unity and subsequently lead to seawater intrusion.

This thesis will investigate the design and operation of a brine reservoir (storage tank), together with isolating valves which will allow the intermittent discharge of brine at design flows in the Chapter 7.2 of this thesis.

6.4 CONSTRUCTION METHODOLOGY

Murray & Roberts Marine was appointed as contractor for the construction of the seawater abstraction and brine disposal facilities. The installation of the marine pipelines was scheduled for December 2009 using the Bottom Tow method, which in summary, entails the following basic methodology:

- The 3 pipelines (two intake and one discharge) are assembled onshore in pipe strings of 160 m lengths
- A launch-way, with roller supports is constructed to reduce friction
- A trench through the surf zone was excavated to facilitate the burial of the three pipelines in that zone
- The inner surf zone trench excavation was protected by constructing a sheetpiled cofferdam
- A concrete caisson structure was constructed in Walvis Bay, towed to the site, and placed at the designed intake location about 1.2 km offshore which will serve as the pulling anchor as well as the intake structure
- A 300 ton linear winch is to be secured on land, with the winch wire (pull wire) connected to the head of first pipe strings of the two intake and one outfall pipelines
- The first three pipe strings is then pulled through the surf zone until the tail reaches the shoreline. The next string is then rolled sideways onto the launch-way and welded to the previously installed strings. The cycle continues until all the strings have been installed
- Once the pipeline is in position, the pull wire is disconnected and the pipeline flooded.

Photographs of the various construction phases, together with a summary of each phase are shown in Appendix D, E, F and G.

7 ANALYSIS OF ALTERNATIVE DESIGN CONCEPT FOR TREKKOPJE

Although general design guidelines are summarized in the literature study for marine components of a seawater desalination plant, this thesis investigated two specific components of the intake and discharge systems in more detail to contribute towards a better understanding ofthe overall design approach. One of the components which was investigated was the construction of a sump for the intake system in order to screen the feedwater on land. The cost and overall feasibility was then compared with the actual design of Trekkopje. The second component was assessing an intermittent discharge facility for the brine outfall and comparing it with Trekkopje's outfall system which does not has such a component.

The above components were based on Trekkopje's intake and discharge design criteria, and the findings shown in the following sections.

7.1 ASSESSMENT OF ON-LAND SCREENING VERSUS OFFSHORE SCREENING

The main reason that the feedwater of a desalination plant should be screened is to remove marine organisms and other solid materials which could endanger the equipment downstream such as the pumps. Screening generally involves coarse and fine screening stages.

Coarse screening can either be performed offshore at the intake headworks such as Trekkopje, on on-land. According to the technical manual of the US Army Corps for domestic wastewater treatment *(TM 5-814-3, 1988)*, the opening of trash racks could vary between 40 and 100 mm, whereas the openings of mechanically cleaned bar screens can range from 13 to 40 mm. Fine screening is mostly performed after coarse screening and the openings normally ranges between 0.5 and 5 mm. Seawater intake screens need to be cleaned on a regular basis in order to remove debris and other solids. However, whether on-land or offshore, marine organisms, especially mussels, will also tend to grow on the screens which will lead to maintenance.

Generally, the sustainability of an intake system will depend greatly on the operational and maintenance procedures and associated costs and normally offshore maintenance would be more expensive than that of maintenance on-land. Furthermore, while offshore maintenance of the intake screens is in progress, the desalination plant would have to be shut down throughout the cleaning operations. As experienced in Perth (refer to Section 5.7) during shutdown periods, stagnant water could lead to the dying of marine growth inside the intake pipelines, which could subsequently damage the pumps and desalination filters as soon as the plant is operating again. However, constructing a sump on land where the feedwater can be screened would lead to additional capital cost for the overall project.

Therefore, in order to contribute towards the overall design approach formulated from this thesis, the alternative for having a sump on land to screen the feedwater was investigated, based on Trekkopje's design flow conditions, intake pipes and headworks. This alternative was then roughly compared with Trekkopje's actual intake system in terms of costs and overall feasibility.

7.1.1 Feasibility assessment of constructing an on-land screening facility

As part of this thesis, a screening sump on land (attracting flow under gravity) was designed which would comprise mechanically cleaned coarse bar screens (40 mm), together with fine screens. This preliminary design was priced and compared with the overall cost of the desalination plant, pump station and marine structures of the Trekkopje project, which comprise 40 mm trash racks at the intake headworks where the feedwater is screened offshore. No finer screening is employed.

The main design criterion is to ensure the minimum required level of the sump can be maintained, taking into account extreme seawater levels, friction and local headlosses and subsequently ensuring that sufficient pipe velocity is maintained to prevent the deposition of sediments in the intake pipeline. The minimum and maximum water levels of the sump were determined using the Bernoulli energy equation with regard to the tidal range and the minimum and maximum headlosses in the intake pipes.

Determine sump levels and dimensions

The minimum and maximum design water levels of the sump were determined with Bernoulli's energy equation.

Since the reason for this alternative design is to compare screening on-land versus screening offshore and the related costs, Trekkopje's main design criteria and modelled friction losses at the intake headwork were used to design a screening facility on-land which can realistically be compared to Trekkopje's actual design in terms of costs and general feasibility. For more information regarding the design criteria and hydraulic losses which were determined for Trekkopje project, and incorporated in this chapter's calculations, refer to Section 6.2.1 (Table 6.1) and Section 6.2.2.

The following design flow rates and friction losses, based on Trekkopje's design criteria were used:

Plant operating at initial plant capacity with one pipe in operation:

Q_{min}	= 2.0 m^3/s	Initial intake flow rate per pipe (Table 6.1)
$h_{f(pipe)} + h_{L(pipe)}$	= 2.4 m	Total headloss per pipe (new pipe)
$h_{f(intake)}$	= 0.4 m	Headloss at intake

Plant operating at future plant capacity with two pipes in operation:

Q_{max}	= 2.1 m^3/s	Future intake flow rate per pipe (Table 6.1)
$h_{f(pipe)} + h_{L(pipe)}$	= 2.7 m	Total headloss per pipe (old pipe)
$h_{f(intake)}$	= 0.42 m	Headloss at intake if trash racks are fully blocked

The following extreme tidal levels according to the South African Tide Tables (*SA NAVY, 2009*), should be taken into account when determining the minimum and maximum water levels of the sump and subsequently the dimensions of the structure:

For determining the minimum water level of the sump:

LAT	= -0.966 m	Lowest astronomical tide to MSL

For determining the maximum water level of the sump:

HAT = +1.004 m Highest astronomical tide to MSL

Using the same pipe diameter as that from Trekkopje of approximately 1.2 m, and the initial and future design flow rates per pipe of 2.0 m^3/s and 2.1 m^3/s, the velocities in each pipe for the initial and future plant capacities are 1.8 m/s and 1.9 m/s.

Calculating the minimum water level of the sump

The minimum water level of the sump should ensure that the required feedwater flow rate can be extracted during the lowest tidal levels, taking into account the maximum friction and local losses at the intake headworks and pipes. At both the ocean and sump level, the pressure is atmospheric and at the ocean, the velocity is negligible. Therefore, using Bernoulli's energy equation:

$$\mathbf{z = v_{pipe}^2/2g + z_{sump} + h_{f(pipe)} + h_{L(pipe)} + h_{f(intake)}}$$

where: v_{pipe} = average pipe velocity (m/s)
z = difference between sea surface and sump level (m)
z_{sump} = set sump level to zero (m)
h_f = frictional head loss (m)
h_L = local head loss (m)

$$\mathbf{= 1.9^2 / (2 \times 9.81) + 0 + 2.7 + 0.42}$$
$$\mathbf{= 3.3\ m}$$

Taking into account the sea water level during spring low, the lowest required water level of the sump should be 3.3 metres below spring low in order to supply the required extraction rate.

Therefore: $\mathbf{z_{sump(min)} = -0.966 - 3.3}$
$\mathbf{= -4.27\ m\ to\ MSL}$

Calculating the maximum water level of the sump

The maximum required water level of the sump while the plant is in operation will depend on the highest tidal level, taking into account the minimum friction and local losses at the intake headworks and pipes. Using Bernoulli's energy equation, it was determined as follows:

$$\mathbf{z = v_{pipe}^2/2g + h_{f(pipe)} + h_{L(pipe)} + h_{f(intake)}}$$

where: v_{pipe} = average pipe velocity (m/s)
z = difference between sea surface and sump level (m)
z_{sump} = set sump level to zero (m)
h_f = frictional head loss (m)
h_L = local head loss (m)

$$\mathbf{= 1.8^2 / (2 \times 9.81) + 0 + 2.4 + 0.4}$$
$$\mathbf{= 2.97\ m}$$

Taking into account the sea water level during the highest astronomical tide (HAT), the highest required water level of the sump should be 2.97 metres below HAT in order to supply the required extraction rate.

Therefore: $z_{sump(max)}$ **= +1.004 – 2.97**
= -1.97 m to MSL

Dimensions of the screening sump
Therefore, from the above results, the lowest and highest extreme water levels in the sump during operation would be -4.27 m MSL and -1.97 m MSL respectively. The dimensions of the sump should also make provision for the water level while the plant is not in operation, taking into account the level of the highest astronomical tide and making provision for additional water level variations (storm surge and wave setup) of 1.5 metres.

The dimensions of the channels for the screening sump of the feedwater were based on the same requirements than that for screening chambers of domestic wastewater works. According to the technical manual of the US Army Corps *(TM 5-814-3, 1988)*, the following is specified in terms of the screening chambers:

- The average approach velocity should vary between 0.45 and 0.6 m/s;
- The maximum approach velocity should not exceed 0.9 m/s;
- The minimum approach velocity should not fall below 0.3 ms/;
- The minimum and maximum width of the approach channels is 0.6 and 1.2 m; and
- A channel length of more or equal to 7.6 m is required in front of the screens.

Since the pipes themselves have a diameter of 1.2 m, it was decided to make the width of the approach channels 1.5 m instead of the specified 1.2 m. Therefore, for a maximum velocity of 0.9 m/s for the lowest water level in the sump, a channel with of 1.5 m and a flow rate of 2.1 m^3/s, the water height should be 1.55 m. Subsequently, the invert level of the sump should be 1.55 m below the lowest water level in the sump (-4.27 m MSL), which means that the base level of the sump should be at a level of -5.82 m MSL.

During operation the highest water level in the sump is -1.97 m MSL and therefore the water height, taking into account the base of the sump level is -5.82 m MSL is 3.85 m. Therefore, calculating the cross-sectional area, together with a flow rate of 2.0 m^3/s, the velocity would be 0.35 m/s, which is more than the specified minimum velocity of 0.3 m/s.

Taking into account the above calculations and requirements, the screening sump's dimensions are schematically illustrated in Figure 7.1 and the overall intake configuration in Figure 7.2.

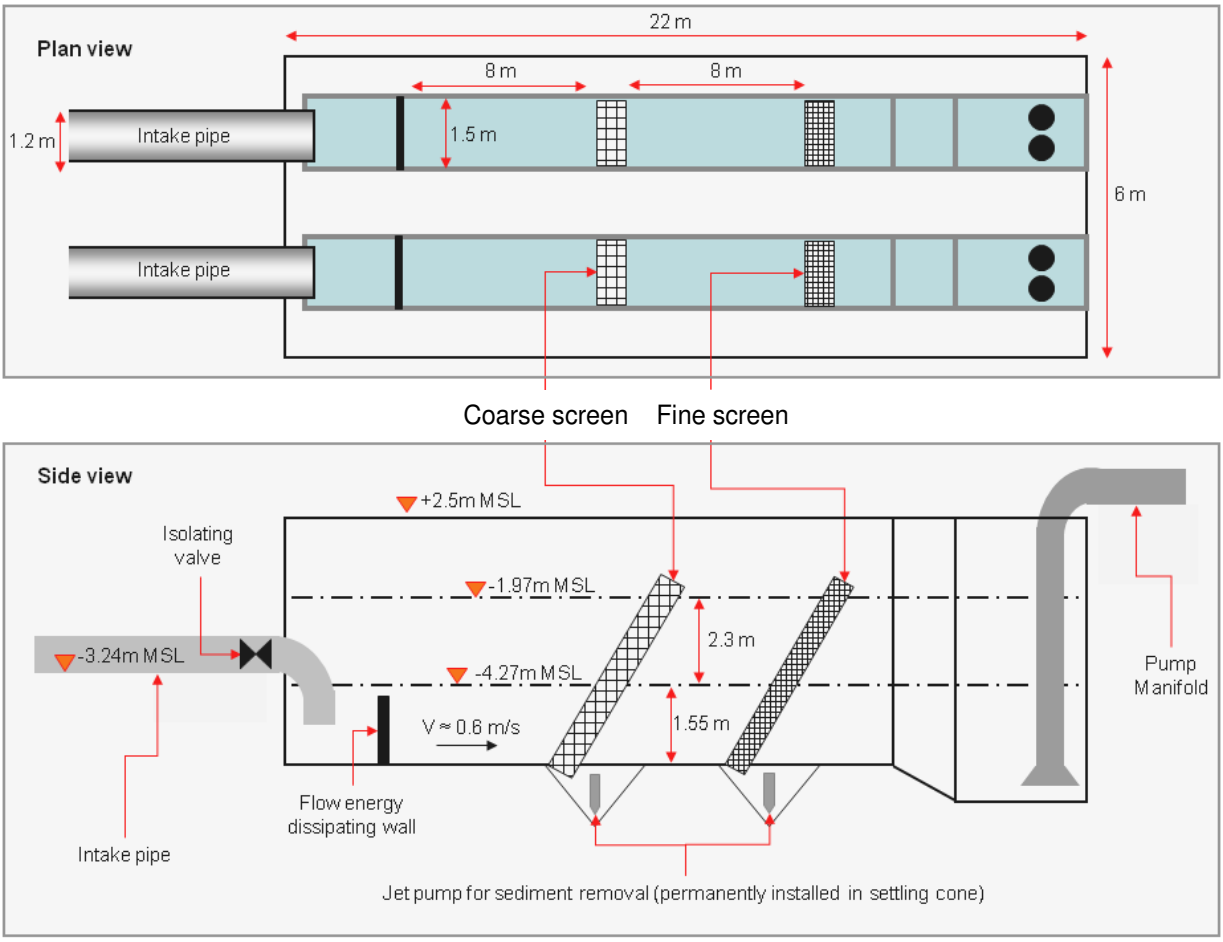


Figure 7.1: Schematic presentation of sediment and marine fouling removal sump

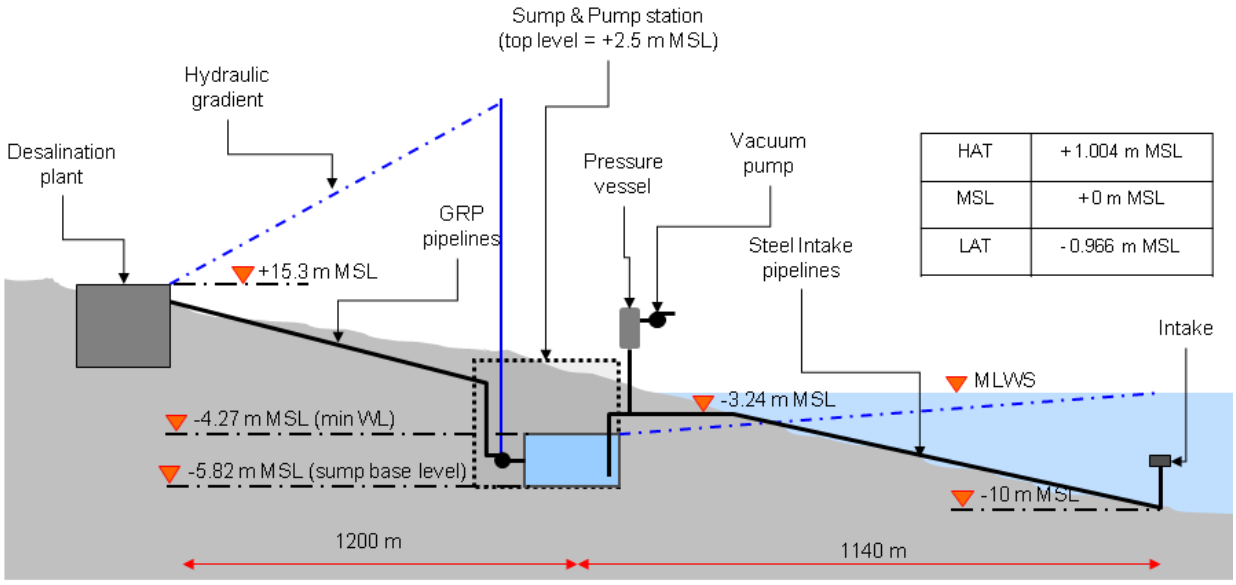


Figure 7.2: Schematic view of proposed sump configuration

A rough preliminary cost was estimated for the above sump and screens and compared with the overall project costs of the Trekkopje project, which includes all the marine structures, desalination plant and pump station. Therefore, expanding the existing pump station, making provision for the sump and additional screens, will cost approximately 1.4% of the total cost of the desalination plant, compared with the estimated cost of 2% of the total cost of desalination plant for offshore

maintenance operations over 10 years. It was roughly estimated that the offshore screens will have to be cleaned three times a year due to site specific conditions and the relatively small screen openings. The cost estimate was based on local rates for a dive team, dive platform and the necessary equipment.

7.1.2 Summary: On land screening option for Trekkopje

Although the initial capital cost of a project could be less by rather providing trash racks with small openings (e.g. 40 mm) offshore at the intake headworks for screening of the feedwater, than constructing an on-land screening facility, the overall sustainability of the system should be considered.

Depending on site specific conditions, the maintenance of offshore marine structures is more difficult, dangerous and subsequently expensive than maintenance on land. Offshore marine structures normally serve as a very favourable habitat for marine organisms, such as mussels and see grass. Taking into account that the plant would also have to be stopped while the bars at the intake structure are being cleaned, the overall cost related to screening feedwater offshore could amount to much more than the initial capital cost to construct an on-land screening facility.

On the contrary, coarse as well as fine screening can be performed at a sump on land, with much lower maintenance costs due to the easy accessibility of the screens. Furthermore, the sump will also serve as a buffer and protect the pumps against fluctuations and subsequently prolong the operational lifetime of the pumps.

Although bars with openings of approximately 150 mm would be required at the intake headworks to prevent large marine organisms, fishing nets or other large object from entering the intake pipes, it is not necessary to screen the feedwater offshore at the intake headworks with openings of 40 mm.

As determined in the above section, the start-up capital costs of the sump and screens could amount to an additional 1.4 % when compared with the entire desalination system at Trekkopje. Considering this percentage with the probable cost of future maintenance, possible shut-downs of the plant due to maintenance or even excessive blockage of the offshore screens, it is considered an important factor to be assessed when developing a desalination system.

7.2 ASSESSMENT OF AN INTERMITTENT DISCHARGE SYSTEM VERSUS NOT

From the literature study it is clear that the main two objectives (which go hand in hand) for any ocean outfall are to ensure an optimum hydraulic design, while adhering to the required environmental guidelines and legislation.

One of the hydraulic design aspects of the Trekkopje outfall which has to be considered beforehand is whether the sediment which is removed during the treatment processes should be discharged with the brine, or disposed by other means. Depending on the type and volume of sediment, and as long as the scouring velocities is maintained and the plume achieves the required dilution levels, sediment can be discharged with the brine stream. However, should the above occur, it is essential to ensure that the

scouring velocities are maintained and therefore intermittent discharge facilities are required should the plant generate lower brine volumes than designed for.

Based on the Trekkopje project, this thesis investigated the feasibility of constructing a brine reservoir to facilitate intermittent discharge in order to contribute to the overall design approach.

7.2.1 Feasibility assessment of an intermittent discharge system based on Trekkopje

It is always better, whenever possible, to design an ocean outfall to discharge continuously for future as well as initial flow rates, while achieving the required dilutions.

However, it is anticipated that the Trekkopje desalination plant will operate between 10 and 50% of the initial design capacity within the first few months of operation. As shown previously (Table 6.2), the brine flow rate which will be generated when the plant operates at its initial design capacity is 1.25 m^3/s. Subsequently, the outfall pipe and diffuser for Trekkopje were designed to ensure scour velocities in the main pipe and compliance to the required dilutions for the initial and future flow rates. Therefore, if the brine discharge rate reduces to only 10 or 50% of the initial design flow rates, the system will no longer comply with the above design criteria. In order to address the impact on the achievable dilutions due to low flow rates, some of the ports of the Trekkopje diffuser will be closed, which will subsequently increase the flow rates in the open ports and therefore the achievable dilutions.

However, the velocities in the main pipe will not be high enough to prevent deposition and ensure transport. The process of “opening” and “closing” ports offshore is also expensive and the plant will have to stop operation while the offshore activities are in progress.

Taking the above into account it could be more feasible to deal with the low flow scenarios by discharging intermittently at design flow rates. During low flow rates, the main outfall pipe can be closed with an isolating valve while the effluent is collected in a reservoir. As soon as the required effluent volume is collected, the isolating valve is opened and the effluent discharged at the design flow rates.

Using Trekkopje’s outfall parameters such as the main pipe diameter, flow rates and tidal variations, a conceptual brine reservoir was designed in order to compare the additional cost of the reservoir with the overall project cost of the entire Trekkopje project and subsequently the feasibility.

Determine minimum water level of reservoir

The main objective of the brine reservoir is to ensure that sufficient head is always available to discharge at design flow rates and subsequently achieve the required dilutions. The Trekkopje design was used as a basis for designing an intermittent system in order to compare the additional costs in terms of percentage.

Since the minimum water level of the reservoir must be determined taking into account the total headlosses of the main pipe and diffuser section during the minimum design flow conditions during extreme tidal and storm conditions, the following design criteria of the Trekkopje outfall design was

used. *For more information regarding the design criteria and hydraulic losses which were determined for Trekkopje project, and incorporated in this chapter's calculations, refer to Section 6.3.*

Plant operating at initial plant capacity:

Q = 1.25 m^3/s Initial brine discharge rate

$h_{f(pipe)} + h_{L(pipe)}$ = 1.91 m Total headloss of pipe and diffuser for initial discharge rate

In order to determine the minimum water level of the reservoir, the highest astronomical (HAT = +1.004m MSL) (*SA NAVY, 2009*), should be taken into account together with the overall headlosses in the main pipe and diffuser section.

Using the same pipe diameter as that from Trekkopje of approximately 1.2 m, and the initial brine flow rates of 1.25 m^3/s the velocity of the main outfall pipe will be 1.13 m/s.

The minimum water level of the reservoir was determined by using Bernoulli's energy equation (Appendix A), with the following design parameters, based on Trekkopje:

At both the reservoir and ocean level, the pressure is atmospheric and at the reservoir, the velocity is negligible. Therefore, using Bernoulli's energy equation:

$$\mathbf{Z = v_{pipe}^2/2g + z_{reservoir} + h_f + h_L}$$

where: v_{pipe} = average pipe velocity (m/s)

z = difference between sea surface and sump level (m)

$z_{reservoir}$ = set reservoir level to zero (m)

h_f = frictional head loss (m)

h_L = l ocal head loss (m)

$$\mathbf{= 1.13^2 / (2 \times 9.81) + 0 + 1.91}$$
$$\mathbf{= 1.98\ m}$$

Taking into account the sea water level of the highest astronomical tide (HAT = +1.004 m to MSL), the minimum required water level of the reservoir should be 1.98 metres above the highest astronomical tide (+1.004 m to MSL).

Therefore: $z_{reservoir}$ min = +1.004 + 1.98

= +2.98m to MSL

Therefore, the minimum water level required to obtain the necessary dilutions will be 2.91m MSL

Determine maximum water level of reservoir

The maximum water level of the reservoir depends on a number of factors. To begin with, the discharge period and stand-still period of the outfall is important to minimize seawater intrusion and prevent the settlement of sediment in the diffuser section, while taking into account the magnitude of

flow rates and the operational and practical implications of closing and opening valves. In addition, environmental issues should be taken into account (e.g. aesthetic impact).

In order to determine the optimum dimensions of the brine reservoir the flow rate, pipe velocity and friction losses had to be determined for a number of water levels. Using a spreadsheet, the following iteration process can be used to determine the maximum water level:

- Step 1: Determine the flow rate for a "first estimate" water level (Bernoulli equation)
- Step 2: Determine the pipe velocity for the design flow rate (v = Q/A)
- Step 3: Determine friction losses for the above pipe velocity (Darcy-Weisbach)

Step 1 to Step 3 was repeated for a number of water levels and the results listed in Table H1 (Appendix H). In order to select the optimum maximum water level, the flow rate, velocity and headloss for a number of different water levels can be plotted (Figure 7.3). Taking into account the general aesthetics of the site with regards to the overall height and width of the reservoir structure, it was decided that the maximum water level should not exceed 4 metres above MSL, as shown in Figure 7.3.

After the initial water levels for the brine reservoir is calculated, the discharge and standstill periods should be calculated for the intermittent discharge system. The following steps follow on the above calculations in order to select the optimum discharge and stand-still periods:

- Step 4: Determine volume of reservoir for the range of water level (minimum and maximum water levels);
- Step 5: Determine the discharge rate for the various water levels and for each time step (Bernoulli);
- Step 6: Determine the discharge period in minutes, taking into account the various discharge rates at each time step (Time = volume/Q); and
- Step 7: Determine the inflow period taking into account the inflow flow rate and required volume to be discharged (Time = volume/Q).

Steps 1 to 7 were repeated in order to determine the optimum water level of the brine reservoir and the results shown in Table H2 (Appendix H).

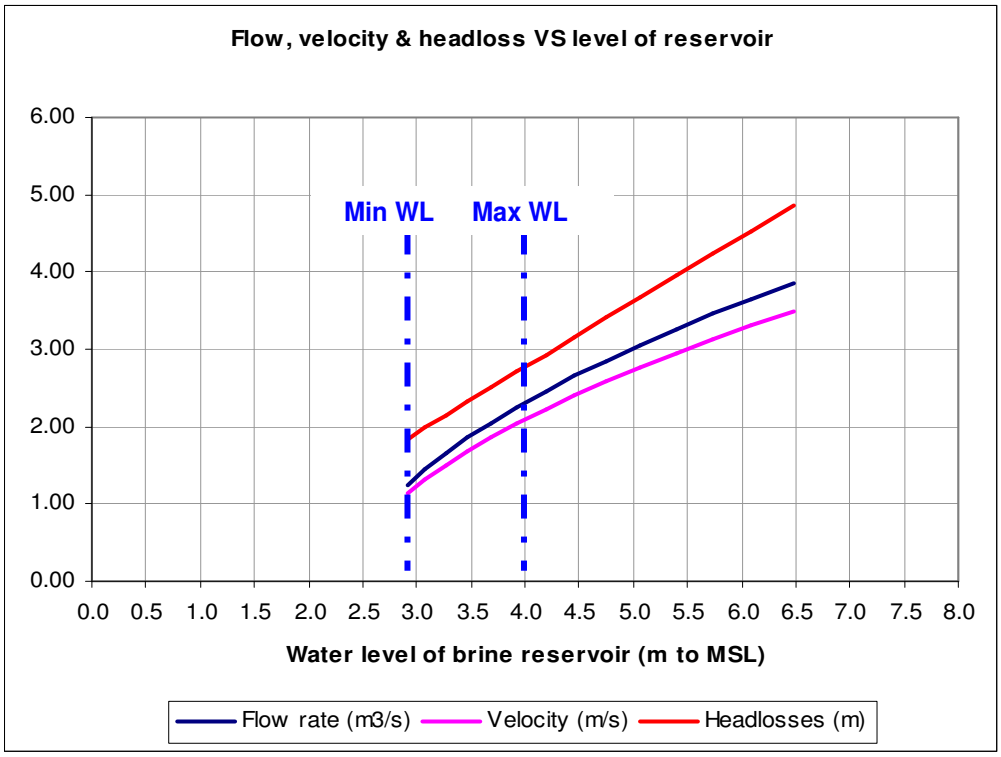


Figure 7.3: Determine maximum water level of reservoir for optimum hydraulic performance

The results of the proposed operational requirements of the intermittent procedure are summarized in Table 7.1. Table 7.2 presents the proposed reservoir tank dimensions together with the applicable operational discharge arrangement for each of the reduced flow rates. A schematic design of the proposed outfall system containing intermittent discharge facilities are shown in Figure 7.4.

Table 7.1: Inflow and discharge rates

Reduced flow rate	1000 m^3/hr flow rate	50% of initial plant capacity
Inflow rate	0.278 m^3/s	0.627 m^3/s
Inflow time to fill reservoir to required water level	96 min	43 min
Min discharge rate	1.254 m^3/s	1.254 m^3/s
Period it takes to empty reservoir	15 in	15 min

Table 7.2: Brine holding tank dimensions and operating requirements

Brine holding tank	
Shape	Cylindrical
Diameter	42 m
Height	1.2 m
HAT	+1.004 m MSL
Min water level to ensure required dilutions	+2.98 m MSL (headloss above HAT)
Max water level	+4.0 m MSL
Volume brine	1601 m^3

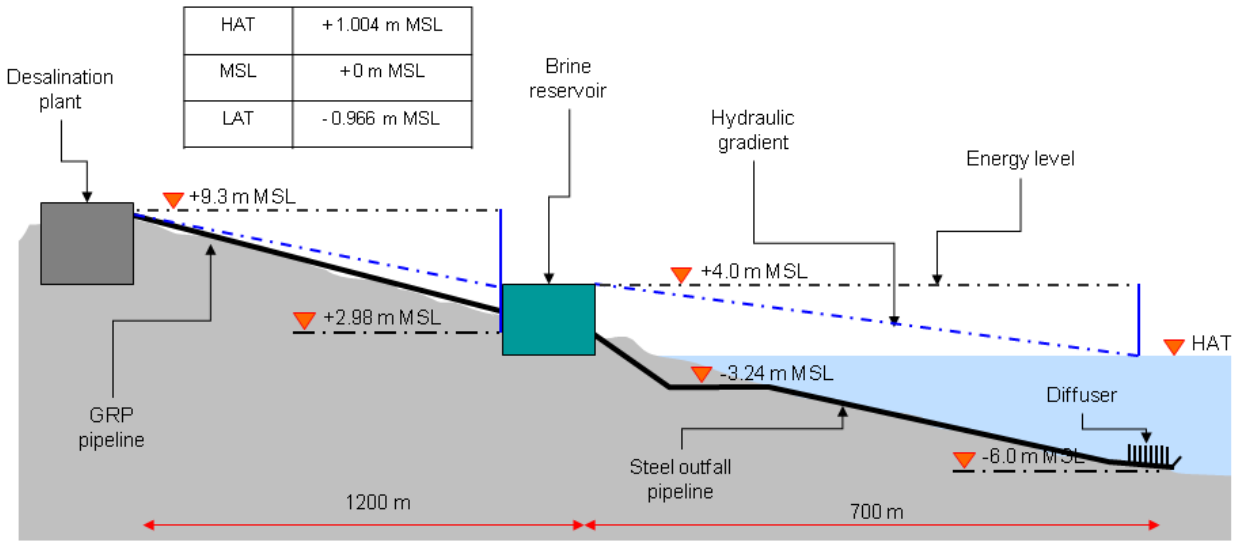


Figure 7.4: Proposed brine outfall configuration

It was roughly estimated that the construction cost of the brine reservoir would be about 1.3% of the total construction cost of the desalination plant, marine structures and pump station.

7.2.2 Summary: Intermittent discharge system alternative for Trekkopje

It is always better, whenever possible, to design an ocean outfall to discharge continuously. However, in order to deal with possible fluctuations due to unforeseen production or operational issues, an outfall system should be designed to discharge intermittently at design flow rates.

An outfall system is designed to comply with environmental criteria and hydraulic requirements for a specific design flow rate and effluent composition. Subsequently, the system will not perform according to the design (environmental and physical) requirements if effluent is discharged at a reduced flow rate.

As shown above (Section 7.2.1), constructing a brine reservoir and valves in order to discharge intermittently can cost approximately 1.3% of the entire desalination project, and therefore other methods to mitigate the effect of reduced flow rates are normally investigated.

These methods includes closing some of the diffuser ports in order to raise the flow rates at the remaining “open” ports, subsequently raising the port velocities and therefore the achievable dilutions such as with Trekkopje.

8 OPTIMIZATION OF DIFFUSER CONFIGURATION

As mentioned in Chapter 4.5.4.3, the main objective of a brine outfall diffuser is to dilute the brine effluent in accordance with the relevant marine water quality guidelines while maintaining self-cleaning velocities and ensuring the friction and local losses does not exceed the available head. The optimum design would have to incorporate the above requirements and still be feasible in terms of manufacturing, installation, operational requirements and subsequently the associated costs.

From the research undertaken for this study, as well as the perceived attitudes of developers in the sector, it seems that the design focus falls generally more on the intake-, than the discharge system. There also seem to be different views and assumptions on the necessity for constructing an optimized diffuser and specific operational procedures to ensure it performs according to its design and subsequently environmental criteria, versus merely an open ended pipe or neglecting operational requirements.

The aim of this chapter is firstly to illustrate the interactive process of designing a brine diffuser by analysing three diffuser configuration scenarios (based on Trekkopje's design criteria as way of explanation). The results indicate how the achievable dilutions depend on the port velocities, which in turn depend on the port diameter and flows, and on the overall pipe hydraulics.

In order to provide developers with an initial estimation of the diffuser configuration requirements, a method was developed, based on scientific theories and generally accepted environmental regulations, which will provide a rough idea of the required number of ports for a specific discharge flow rate and port diameters.

In order to design the optimum diffuser (finding a balance between the dilutions which can be achieved and a hydraulically sound diffuser layout), it is necessary to understand the concept of the various components. Using the WAMTech model (refer to Section 4.5.4.4), three different diffuser configurations were modelled to illustrate how the interdependence between the various hydraulic parameters and achievable dilutions affect each other.

8.1 HYDRAULIC AND ENVIRONMENTAL MODEL FOR DIFFUSER DESIGN

The main concept of a hydraulic and environmental model is that the various flows in all the ports are determined by the Bernoulli energy equation between each successive port and subsequently the dilutions for each port calculated.

The development and evaluation of the WAMTech dilution/hydraulic model was primarily based on the EPA information provided below as well as extensive field/prototype experiments conducted in South Africa.

During 1985 the United States Environmental Protection Agency (US-EPA, 1985) "took stock" of numerous plume "models" and published computer programmes of a number of numerical models to describe the dilutions which can be achieved when a waste water plume is discharged at depth and being driven by the momentum and the buoyancy, entrains clean ambient sea water to be "mixed" with the waste water jet. Original programmes published by Baumgartner *iet al.* (1971) were

updated in US-EPA (1985). One of the programmes, UOUTPLM, computes rise heights and initial dilutions for stagnant and moving water (ocean currents). The computations for UOUTPLM are based on tracking a plume element of a single jet as it gains mass due to entrainment of the ambient sea water. Change in momentum, energy, density, buoyancy and dilution are computed as the element rises through the water column until the vertical velocity reaches zero or the water surface is reached. Another programme which was provided by United States Environmental Protection Agency (US-EPA, 1985) is UDKHDEN which is a fully three dimensional model which took into account the stratified nature of the sea water and varying current velocities throughout the water column. The governing equations for the numerical development of UDKHDEN (US-EPA, 1985) are:

- Conservation of mass
- Conservation of energy
- Conservation of pollutant (conservative substances)
- Conservation of momentum, developed to convert the plume coordinates to 3-D Cartesian coordinates for the spatial output of the results.

For the derivations of the above equations the following assumptions were made:

- Flow is steady in the mean
- Fluid is incompressible and density variations are only included in the buoyant terms
- All other fluid properties are constant
- No frictional heating
- Pressure variations are hydrostatic
- Ambient turbulence are included in the entrainment function only
- Flow within the jets are axisymmetric and free.

In the WAMTech model, the UOUTPLM and UDKHDEN programmes were combined and coupled to a multi-port diffuser hydraulic model (based on the balance of energy between two adjacent ports). An interactive model was constructed which illustrates the geometry of the rising plumes, achievable dilutions and all hydraulic parameters (port flow, velocity, Froude No as well as the main pipe velocity along the diffuser).

Trekkopje's design flow rates and required dilutions were used as a basis for the runs. The three modelled diffuser configurations included a multi-tapered main pipe, an un-tapered main pipe as well as the Trekkopje configuration. A schematic illustration of the diffuser configurations is shown in Figure 8.1.

The environmental and hydraulic performance of all three different diffuser configurations was modelled using the WAMTech model with the following design parameters, based on Trekkopje:

- Design flow rate: 2.7 m^3/s (Max future plant capacity, *WSP 2008*)
- Main pipe inner diameter: 1.2 m *(WSP, 2008)*
- Discharge depth: 7 m *(WSP, 2008)*
- No of diffuser ports: 20 *(WSP, 2008)*
- Ambient conditions: Stagnant

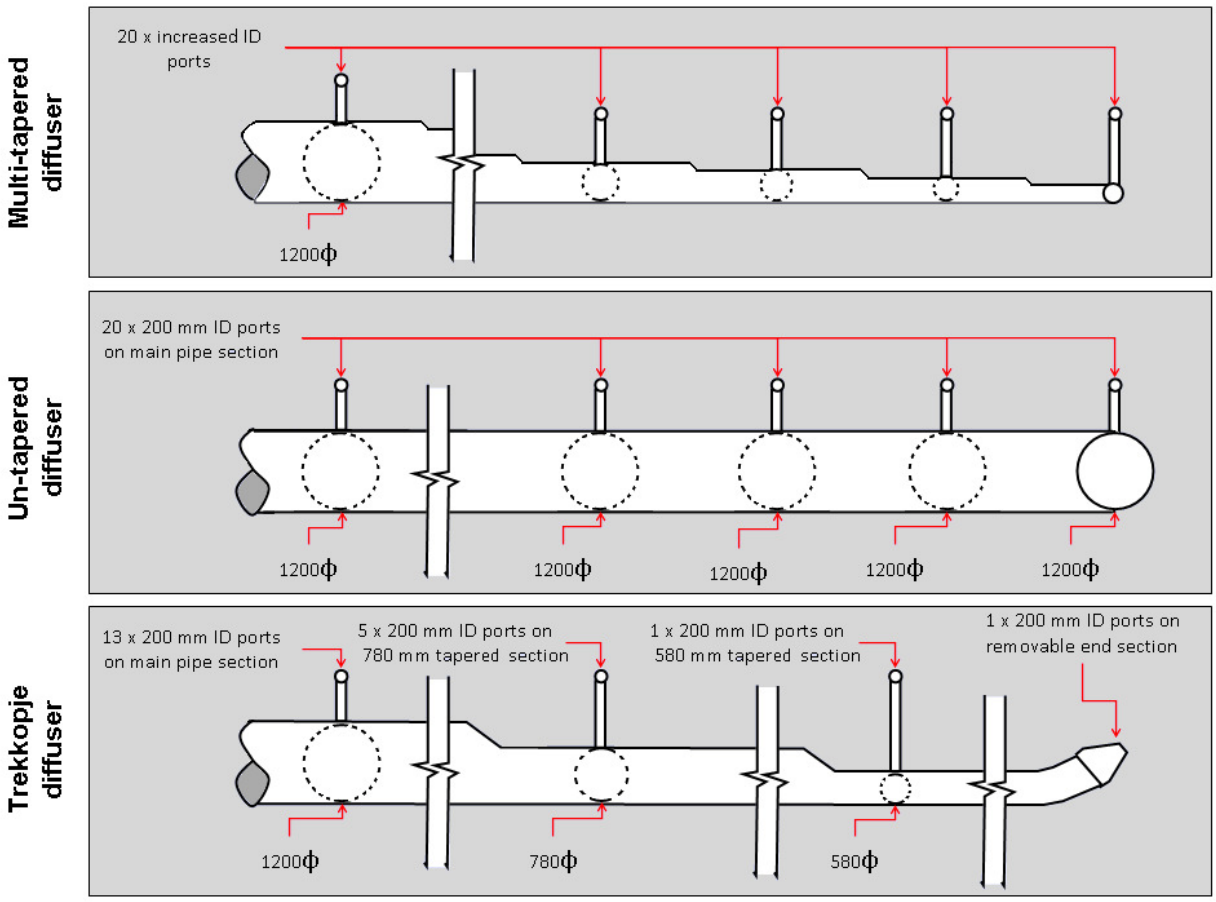


Figure 8.1: Modelled diffuser configurations

8.2 DIFFUSER DESIGN

8.2.1 General

The primary objective for the design of a diffuser for a sea outfall is discharge the effluent in such a way that the environmental criteria (for the specific area and the project) are adhered to during all weather (sea) conditions throughout the design life of the outfall. In order to achieve this objective, Only a flawless hydraulic outfall (main pipe and diffuser) design will ensure that this objective can be met.

Basic criteria for the hydraulic/environmental design of a diffuser for a sea outfall are:

- Design flows must be discharged satisfactorily through the ports. A rule of thumb for the continuity of flow is that the total cross-sectional areas of the ports should not be less than 0.7 times the cross-sectional area of the main pipe at any point in the diffuser.

- Depending on the physical characteristic of the effluent as well as the nature of the immediate area where the diffuser is located, port diameters should not be too “small” in order to minimize the possibility of port blockage. Flow interference to one port will have an adverse effect on the overall hydraulic performance of the outfall and the subsequent negative impact on the environment, which is the primary objective for the design.

- Sufficient flow in each port should be maintained to prevent the intrusion of seawater. This can be adhered to if the port exit velocities are high enough to ensure that the densimetric Froude Number for each port is always greater than unity (i.e. 1) for all flow scenarios.

- The even distribution of flows, through all the diffuser ports, is important, because the flow for a port is directly related to the achievable dilution, and the "worst performing" port will be considered as representative of the achievable dilution of the entire system (overall performance of the diffuser).

- It is of the utmost importance to maintain main pipe flows throughout the diffuser section to limit the deposition of material (sediment) and to ensure that when required, scouring flows can be introduced within the diffuser section. This can only be achieved by introducing tapers (gradually reducing the main pipe diameter) in the diffuser section.

- Optimum dilution will be obtained with ports which will provide the "longest" path for the effluent plume before the process of initial dilution ceases, that is when the ability to entrain "clean" sea water ceases (plume reaching the surface or the entrainment processes within the plume are not sufficient vigorous to entrain adjacent water). As the geometry of the path of a plume is a vector of the horizontal and vertical fluxes of the plume with an additional horizontal component from the ambient current (vertical ambient density currents may also have an effect, but due to the varying nature and difficulty to measure and quantify, these are neglected for analytical and 2-D calculations and modelling). The length of the path as well as entrainment "ability" of the plume depends on:

  - The momentum and buoyancy flux (driving forces) of the plume.
  - The angle of discharge.
  - The dynamic nature of the receiving water body (currents).

Examples: For a buoyant plume, discharging through an upright diffuser in stagnant conditions (no currents), the length of the path of the plume will be limited to the waterdepth where a horizontal port will lengthen the path due to the horizontal jet momentum and the vertical buoyancy flux. For a dense effluent, discharging through an upright port in stagnant water, the discharged plume will "fall back" on the port and the "contamination" of sea water to be entrained will make the theory null and void.

- The theory of dilution is based on that water with ambient quality is entrained to mix and to dilute the effluent in the plume. Therefore, the distance between any two ports must be such that the plumes (during the initial dilution process) do not merge, and subsequently result in the entrainment of already "contaminated" water.

8.2.2 Trekkopje's sea outfall

When considering the Trekkopje's sea outfall (refer to Chapter 6.3), the influencing aspects (controlling parameters) are:

- It is a dense effluent (1046 g/l), thus no positive buoyancy flux.
- The required dilution to meet the environmental objectives is 20.
- The diameter for the main pipeline is 1200 mm (selected to ensure scouring velocities are maintained for the flow scenarios).

What are the first "ball park" assumptions for the Trekkopje's outfall with regard to the basic assumptions in Chapter 8.2.1 for a sea outfall?

- Additional momentum is required for a "long" enough path to entrain seawater for achieving the required dilutions, thus the momentum flux for each port has to be increased.

- Due to a required dilution of only 20 and the limiting rising height of the dense effluent, the depth of the outfall and the subsequent offshore length can be limited. However, the closer inshore, the more vulnerable is a diffuser (physically and hydraulically) due to the more extreme inshore conditions (processes, physical interference and stability) and the objective should be to make the diffuser as short and rigorous (not to small port diameters and limiting riser heights) as possible.

- The discharge angle of the ports should be inclined to the vertical (e.g. 45 deg) of the main pipe to achieve the maximum path length before the momentum flux is lost and to ensure that the dense effluent is "thrown" away from the diffuser. The ports discharge to alternative sides of the main pipe to:

- Double the port spacing with regard to the possible merging of plumes.

- Neutralize the lateral force on the main pipe, exerted by the jets. This is important for this outfall where jet velocities are relatively high for discharging the dense effluent.

Initial modelling must be done, using numerous configurations, with the above assumptions and criteria as a guideline and it was concluded that 20 ports with an internal diameter of 200 mm will satisfy the assumptions and criteria.

To examine the sensitivity of the solutions, a review of the effect on diffuser hydraulics and achievable dilutions of this configuration (20 x 200 mm ID ports) and the three main pipe options (as illustrated in Figure 8.1) with reference to the basic assumptions in Chapter 8.2.1 is discussed in Chapter 8.3 and 8.4.

8.3 DIFFUSER HYDRAULICS

8.3.1 Main pipe hydraulics

Refer to the general assumptions for a diffuser design if Chapter 8.2.1:

One of the most important considerations for the design of an outfall diffuser is to ensure that the "designed" hydraulic conditions can be maintained throughout the life-time of the system. Any physical changes to the diffuser (port blockage or diameter changes of the main pipeline in the

diffuser) will have adverse cumulative effects in time on the hydraulic behaviour as well as possible structural damage (for example port "blow-outs") and the subsequent adverse environmental performance of the system. For this effluent, the most likely physical interference with the hydraulic behaviour will be due to possible sedimentation when velocities are not maintained in the main pipe of the diffuser section. The ultimate solution (maintain a constant main pipe velocity), the "cheapest" solution (constant main pipe diameter) and the practical solution are discussed below. As the port velocity is the controlling parameter for the achievable dilutions, the options are verified in Section 8.3.2 to ensure that required dilutions are not sacrificed.

The total cross-sectional area of the ports is 0.3 times the cross-sectional area of the main pipe at the onshore end of diffuser for the 3 main pipe configurations, which is less than the recommended 0.7, because additional jet momentum is required to discharge the dense effluent. The effect will be increased pressure along the entire diffuser. A change of this area ratio at any point in the diffuser will have a direct effect on the velocity in the main pipe.

For the gradually tapered main pipe this factor is maintained along the length of the diffuser, resulting in a constant main pipe velocity along the length of the pipeline. For the 3-taper main pipe the ratio was increased at each taper and for the un-tapered main pipe, this factor gradually reduces to 0 towards the offshore end of the diffuser.

Figure 8.2 illustrates the main pipe velocities in comparison with the main pipe diameter at each port for the multi-tapered diffuser configuration.

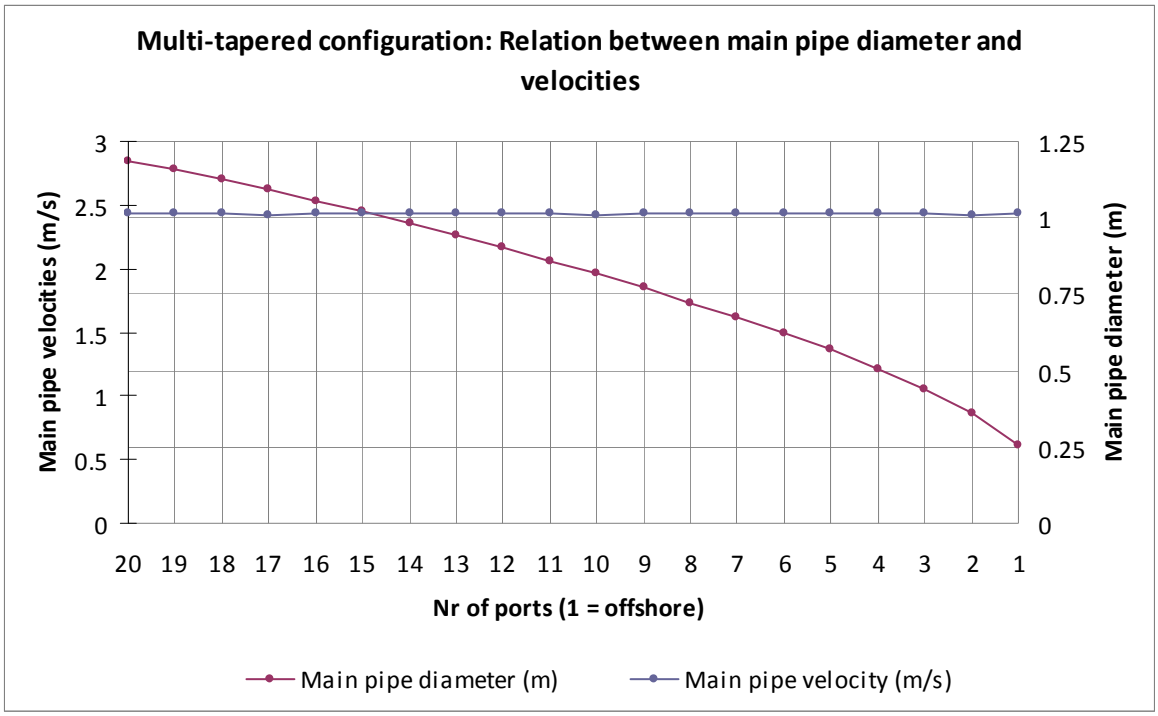


Figure 8.2: Relation between main diffuser pipe diameter and velocities for a multi-tapered diffuser

The hydraulic modelled results of an un-tapered main diffuser pipe are illustrated in Figure 8.3. From this figure, it is clear that the velocities will decrease as the effluent is discharged through the ports. Since scouring velocities (v > 0.7 m/s) cannot be maintained in approximately a third of the diffuser length, this configuration will not be feasible if the effluent contains suspended solids or sediments, which is a possibility in any discharge system.

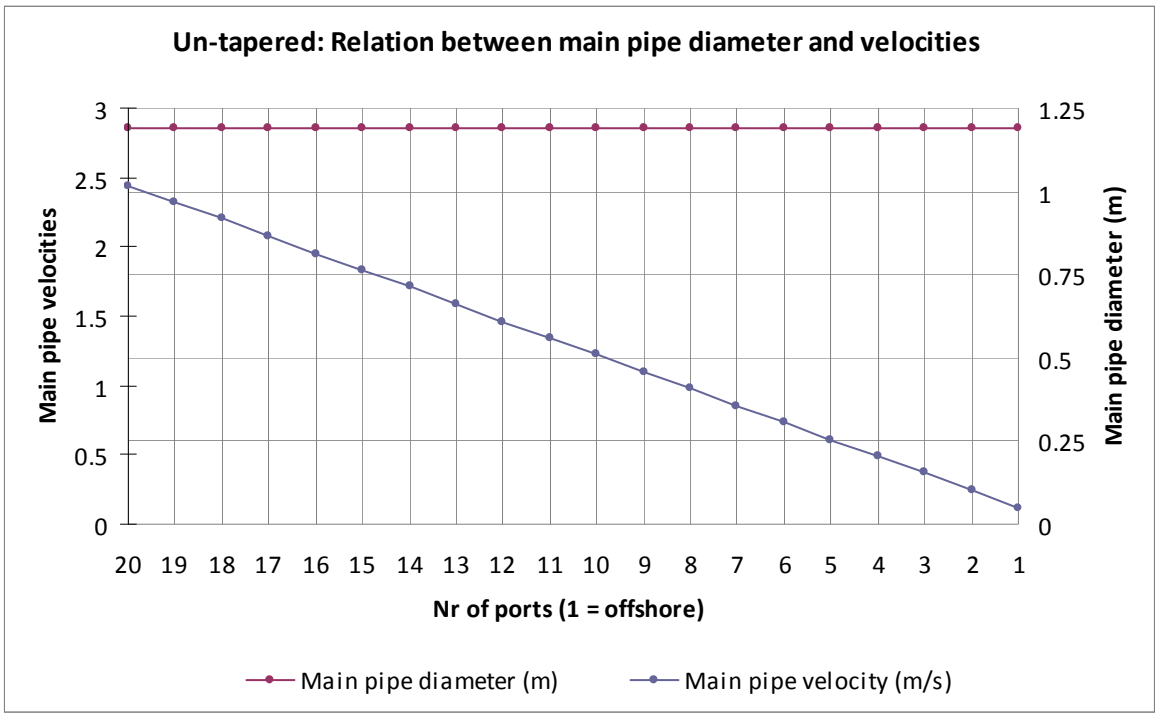


Figure 8.3: Relation between main diffuser pipe diameter and velocities for an un-tapered configuration

In theory a gradual tapered diffuser (tapered at each port) to compensate for the reduced flow, will be the ultimate solution. However, the practicality of this and the associated cost of diffuser manufacturing should be considered and depending on the design (total number of ports) the number of tapers should be limited without sacrificing the design criterion of scouring/flushing pipe velocities.

Figure 8.4 illustrates the Trekkopje's main pipe (2 tapers at port no. 8 and port no.3) velocities and main pipe diameter at each port. The velocity in the main pipe is more than 0.7 m/s, except in the last offshore section (10% of the overall length of the diffuser) which was considered acceptable by the designers.

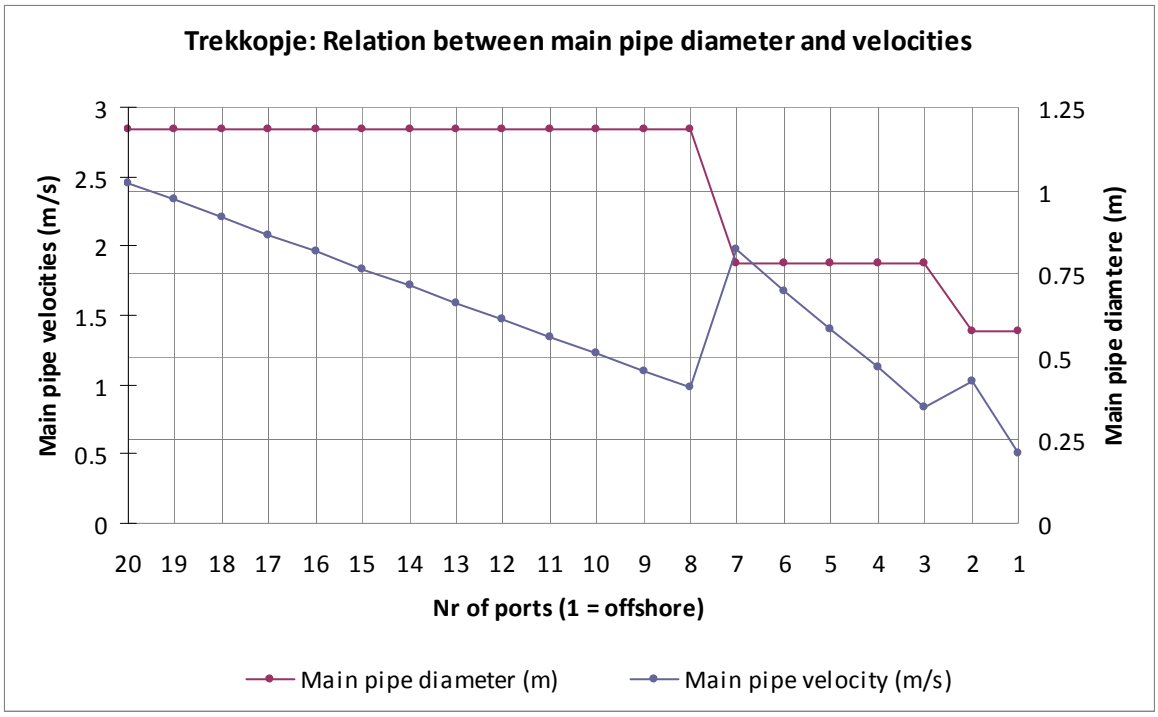


Figure 8.4: Relation between main diffuser pipe diameter and velocities for Trekkopje configuration

8.3.2 Port hydraulics

Refer to the general assumptions for a diffuser design if Chapter 8.2.1:

The diameter of all the ports is 200 mm. This is large enough to prevent any blockage from possible material in the effluent. The 200 mm ID of the ports also lead to a "robust" port which will be less vulnerable to physical damage in the relatively shallow water.

The densimetric Froude Number for each port should be more than unity to ensure full pipe (port) flow and the subsequent prevention of seawater intrusion. The Froude Number (refer to Appendix A for the equation) of the jet at exit is a function of the port exit velocity, which is shown in Figure 8.5 for the multi-tapered as well as Trekkopje's diffuser configuration. For both options the minimum Froude Number exceeds 20.

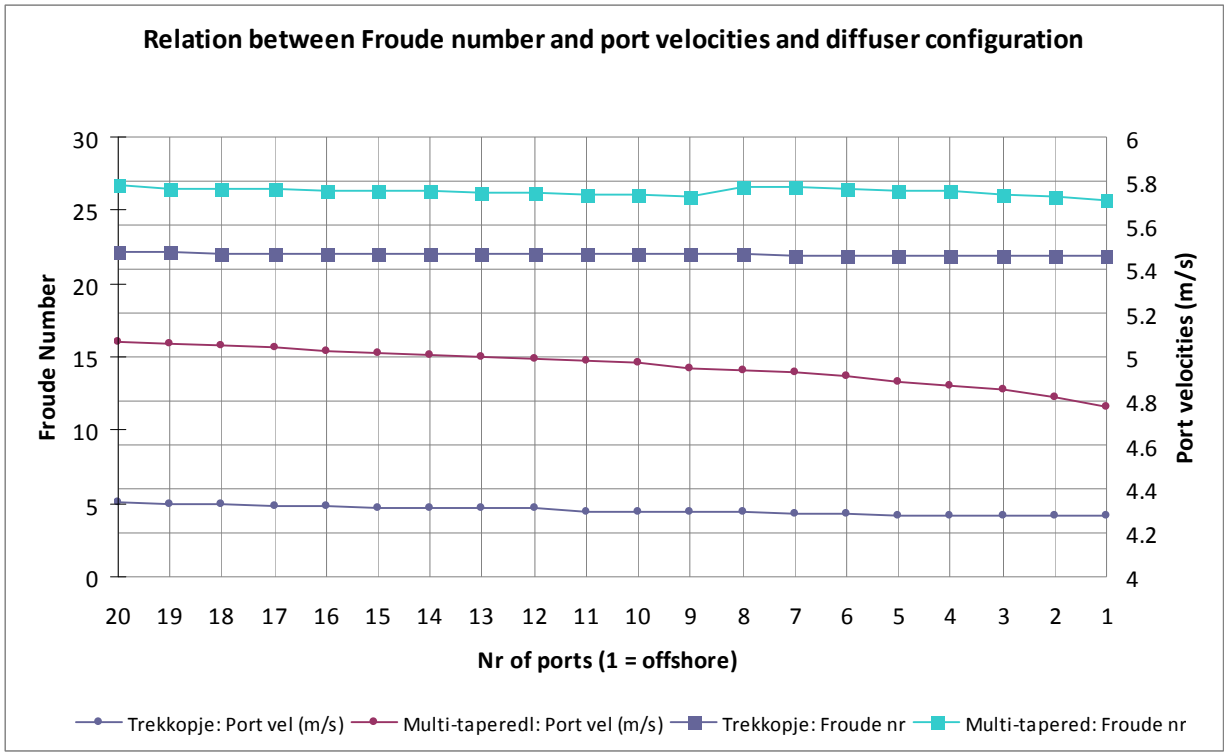


Figure 8.5: Relation between Froude number and diffuser configuration

The basic criteria, "maintain an even distribution of flows, through all the diffuser ports", is easily adhered to for all main pipe configurations due to the high pressure which is maintained along the entire diffuser, as a result of the low cross-sectional area ratio and the relatively short diffuser with only 20 ports. The maximum, minimum and average port exit velocities are 5.07, 4.77, 4.96 m/s respectively for the multi-tapered diffuser and 4.34, 4.28, 4.30 m/s respectively for the Trekkopje's diffuser configuration.

The discharge angle of the ports is 60 degrees to achieve the maximum path length before the momentum flux is lost and to ensure that the dense effluent is "thrown" away from the diffuser.

The port spacing along the entire diffuser is 3 m. Thus the distance between adjacent ports is sufficient to prevent merging of the plumes. The maximum plume diameter at the summit of the plume during stagnant conditions is 1.8 m.

8.4 ACHIEVABLE DILUTIONS

The physical properties of the brine plume (negatively buoyant) limit the initial dilutions which can be achieved. A buoyant plume dilutes by entraining clean seawater as it rises to the water surface, whereas a dense plume will sink to the seafloor as indicated in Figure 8.6. Although the brine characteristics and marine water quality guidelines will vary for different desalination technologies and countries, the following general conclusions could be drawn from the literature study:

- Generally, the salinity concentration of a brine stream from a seawater reverse osmosis plant will be double that of the ambient seawater; and

- From international marine water quality guidelines, the allowable salinity of a diluted effluent plume should be within the range of 33 to 36 ppt.

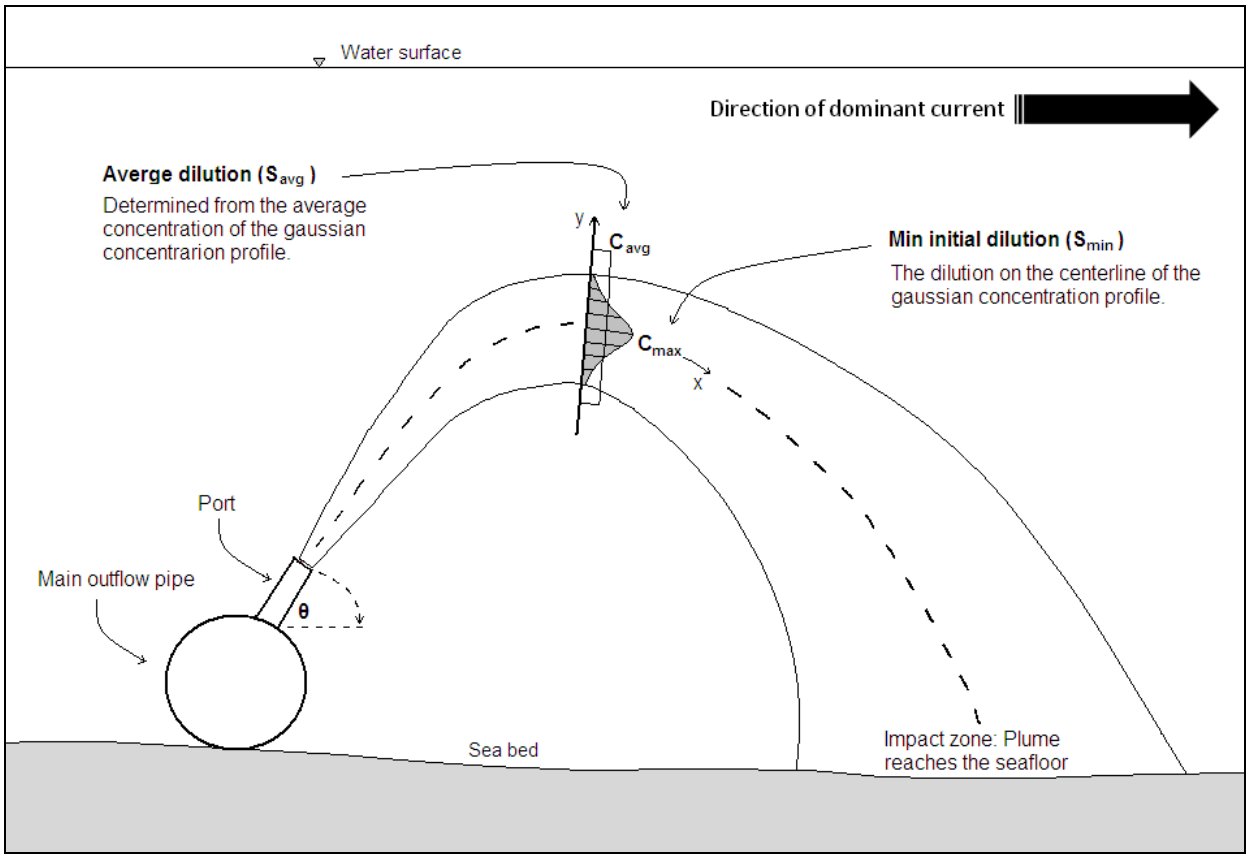


Figure 8.6: Plume characteristics of a dense effluent

Taking into account the above, the required dilutions (S_{min}) were determined as 20 (refer to Section 4.5.3.1 for the equation which can be used to determine required dilutions). Futhermore, this is also the same required dilutions for the Trekkopje project.

From Trekkopje's investigation, it was found that although the required dilution (S_{min}) of 20 could not be met during the initial dilution process (refer to Figure 7.9) (which is normally considered when an outfall system is designed), an initial dilution of approximately 16 times was considered sufficient under medium ambient conditions, providing that the required dilution of 20 is reached before the plume reaches the seafloor.

- *Coastal currents:* A coastline could be bounded by major circulation systems (South Africa has two: the "warm" south-bound Aghulas current along the east and south coast and "cold" north moving Benguela system along west coast). Offshore circulation characteristics (speed and direction of currents) are the main oceanographic processes that would influence the initial dilution of a buoyancy stream and its subsequent transport and dispersion to distant locations. Currents in the surf zone are wave-dominated and initial mixing is rapid due to the vigorous processes of which long-shore and corss-shore transport is the most dominant.

  Wind driven currents are an important factor to take into account for the design of a sea outfall, especially when a buoyant plume rises to the surface and the subsequent transport of the waste field to distant locations. With regard to initial dilutions near the sea floor, the

influence of the wind will depend on the persistence of the wind from a specific direction as the wind stress on the water surface will gradually generate sub-surface currents due to shear between the moving surface water and the lower layers. The effect of the wind driven currents on the initial dilutions for a dense plume (submerged) will be less significant. The site for the Trekkopje's sea outfall is north of the Tropic of Capricorn, and thus further away from the southerly trade winds (cyclonic systems) and closer to the equatorial doldrums. The winds are moderate and most of the time from a SW'erly direction. The effect of diurnal onshore/offshore winds due to day and night temperatures differences between land and sea will not have a significant effect on the deeper currents which will influence the initial dilutions of a submerged dense jet, because of the relatively short duration of the peak wind speed from one direction.

The strongest near-shore currents occur during S to SW'erly wind conditions (blowing at an inclined angle to the north) and spring tide conditions when the wind driven north bound currents are superimposed on the diurnal tidal currents and possible inshore north bound water movement of the anti-clockwise moving Benguela system (northerly direction along the west coast of Africa). The net water circulation along the Namibian coastline is to the north and the occurrence of southbound currents at the outfall site is rare.

- *Stratification:* When denser seawater underlies lighter sea water causing a vertical density gradient in the water column, depending on the vertical temperature gradient between warmer upper water layers and colder deeper water and the salinity gradient. (Stratification not significant for water depth less than 10 metres.) Density stratification is the major factor that influences the rising of a buoyant wastewater plume and thus determines whether a buoyant discharge from an ocean outfall remains beneath the surface as a submerged field or continues to rise to become a surface field.

Figure 8.7 indicates that for the multi-tapered diffuser configuration, the achievable dilutions (S_{min}) during stagnant ambient conditions will be limited to 17, with port velocities of about 5m/s. (Note: The maximum and minimum port velocities are 5.07 and 4.77 m/s. This small variation is due to the high pressure which is maintained in the relatively short diffuser with an area ratio < 0.32). These dilutions are brought about only by the momentum flux of the jet, as the buoyancy is negative.

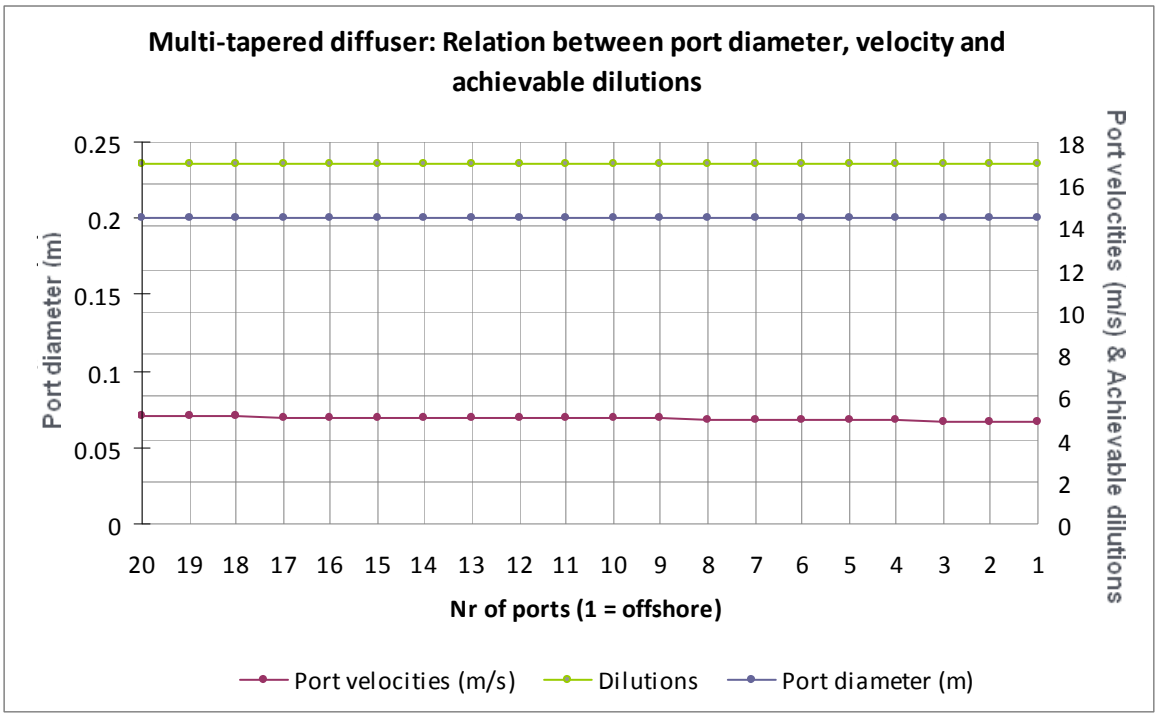


Figure 8.7: Hydraulic and environmental performance of a multi-tapered diffuser

However, Figures 8.8 and 8.9 indicate that the achievable dilutions (S_{min}) will be about 15 for the Trekkopje diffuser as well as the un-tapered diffuser. The port velocities of Trekkopje and the un-tapered diffuser's are lower than the velocities of the theoretical diffuser configuration, therefore the higher achievable dilutions for the multi-tapered diffuser configuration.

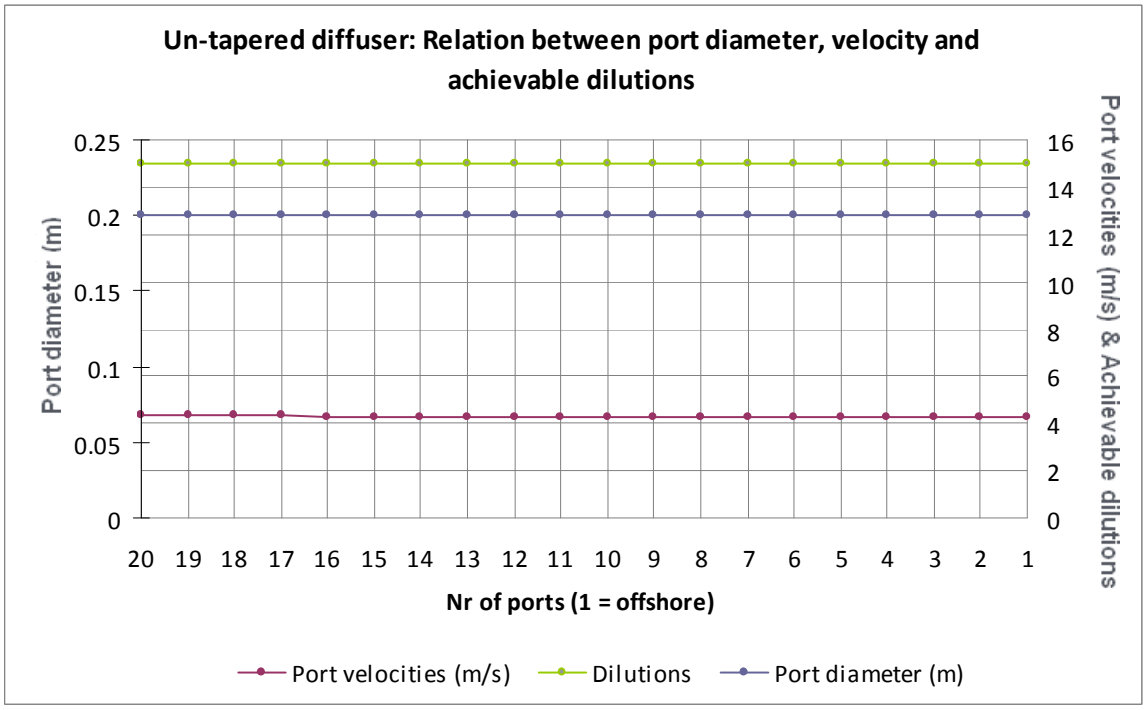


Figure 8.8: Hydraulic and environmental performance of an un-tapered diffuser

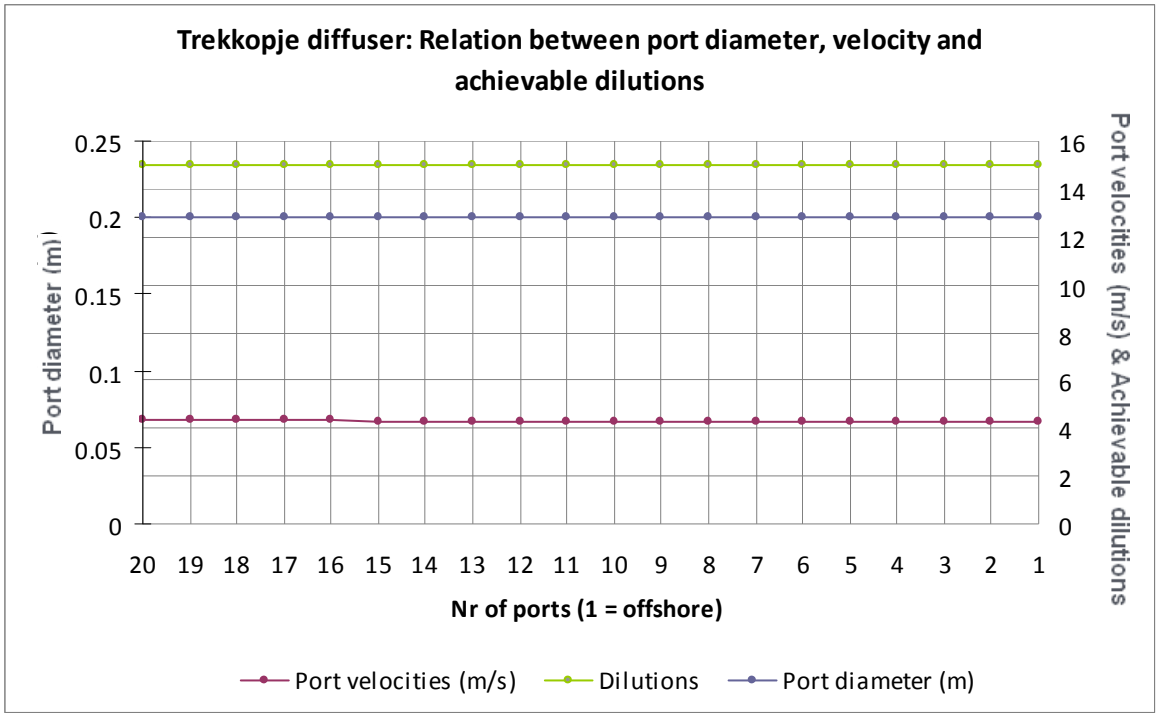


Figure 8.9: Hydraulic and environmental performance of Trekkopje's diffuser

8.5 CONCLUSIONS FOR OPTIMIZING A BRINE DIFFUSER

From the available literature, general procedures to design an ocean outfall have been described in Chapter 4.5. Taking into account the above assessment, the following design approach for a brine diffuser is recommended as a point of departure to contribute to the general guidelines provided in Chapter 4.5:

1. Determine the influencing aspects (controlling parameters):

   1.1 Effluent density (which is more dense than seawater for a brine effluent);
   1.2 The required dilution to meet the environmental objectives (which would normally be in the order of 20, depending on the site specific requirements, as discussed in Section 4.5.3.1 and Section 6.3); and
   1.3 The diameter of the main pipeline, which would first be determined for the outfall pipe itself, which subsequently will determine the diffuser configuration (i.e. the port diameters and the number of ports).

2. Additional momentum is required for a "long" enough path of the jet plume to entrain seawater for achieving the required dilutions, thus the momentum flux for each port has to be increased. It is not advisable to raise the port velocities too high, since the forces on diffuser components become greater as the port velocities increase.

3. Although a brine diffuser does not require great depth due to the limiting rising height of the effluent plume, the more inshore any marine structure, the more vulnerable to nearshore physical processes (wave forces and unstable seabed conditions).

4. The discharge angle of the ports should be inclined to the horizontal in order to achieve the maximum path length of the plume.

5. Initial modelling, using numerous configurations, is required to optimize the diffuser design.

This thesis also investigated a method to provide a rough indication of the number of ports which would be required for a brine outfall. The number of ports will be determined by the required dilutions and the discharge flow rate.

The initial dilutions were estimated according to Roberts and Toms *(WRC, 1992)* who formulated an equation (refer to section 4.5.4.4) for determining the initial dilution (dilution at terminal rise height at the centre line of the plume, refer to Figure 8.5) for a dense effluent with a port discharge angle of 60 degrees to the horizontal.

Assuming an initial dilution of 20, thus working "backwards", the flow rate per port was determined by dividing the design flow rate equally between a number of ports and subsequently calculating the port velocities and dilutions for a range of port diameters as illustrated in Figure 8.10. Refer to Appendix H for the calculation spreadsheet.

The use of the graph (Figure 8.10) for ball park estimates is illustrated more clearly by means of the following example:

Example: Determining minimum and maximum number of ports and port diameters for a specific discharge flow rate

Design criteria:
- Discharge flow rate (Q) = 2.7 m^3/s (Based on the Trekkopje project)
- Main pipe diameter (D_{main}) = 1.2 m (Based on the Trekkopje project)

Part A:
As the number of ports of a diffuser reduces, so does the length of the diffuser, the manufacturing, installation as well as maintenance costs and subsequently the overall financial feasibility. The first part of this example illustrates how the minimum required number of ports for a certain discharge flow rate and port diameter can be determined for a first-estimate in a diffuser design process.

In order to determine the minimum required number of ports using Figure 8.10, the discharge flow rate and a port diameter is required.

The engineer has to select a realistic port diameter for a first estimate, taking the following considerations into account:
- For the continuity of flow, the total cross-sectional areas of the ports should not be less than 0.7 times the cross-sectional area of the main pipe at any point in the diffuser;
- A port diameter of less than 0.075 m is susceptible to blockage;
- Standard available pipe sizes for various pipe materials and cost of manufacturing special

pipe sizes for the ports; and

- A dense effluent requires a high port exit momentum to increase the terminal rise height and the length of path to entrain seawater to ensure that the maximum dilutions are achieved.

For this example, taking the above into account, a port diameter of 0.2 m (for all the ports).

Figure 8.10, which presents the relation between discharge flow rates, ports diameter and required number of ports, indicates that 9 ports would be required to achieve a 20 times dilution of a dense effluent discharged at 2.7 m^3/s through a diffuser with 0.2 m diameter ports.

Part B:

Part B of this example illustrates how the minimum required port diameter could be obtained for a certain number of ports and the discharge flow rate.

In order to select a conservative maximum number of ports the following should be taken into account:

- A required dilution of 20 is not very high to achieve in a dynamic environment; and
- Depending on the discharge flow rate and ambient conditions, it would normally be impractical and not feasible from a financially point of view to design a diffuser for a brine outfall with more than about 30 ports which only required 20 times dilutions.

Taking the above into account a number of 26 ports were selected.

Therefore, from Figure 8.10, the minimum required port diameter for a flow rate of 2.7 m^3/s and a diffuser with 26 ports is 0.13 m.

Note: The above steps could be repeated a number of times, but the following should be taken into account:

- *The effluent is dense and has a salinity of approximately 1.5 times that of the ambient seawater;*
- *The graph is based on a required dilution of 20;*
- *Assume the same port diameter for all ports;*
- *The grey area indicates the generally accepted port diameters and number of ports for a brine effluent with 20 times required dilutions; and*
- *The graph will only serve as a rough indication of a diffuser configuration and detailed hydraulic and environmental modelling is required in order to determine the most optimum design.*

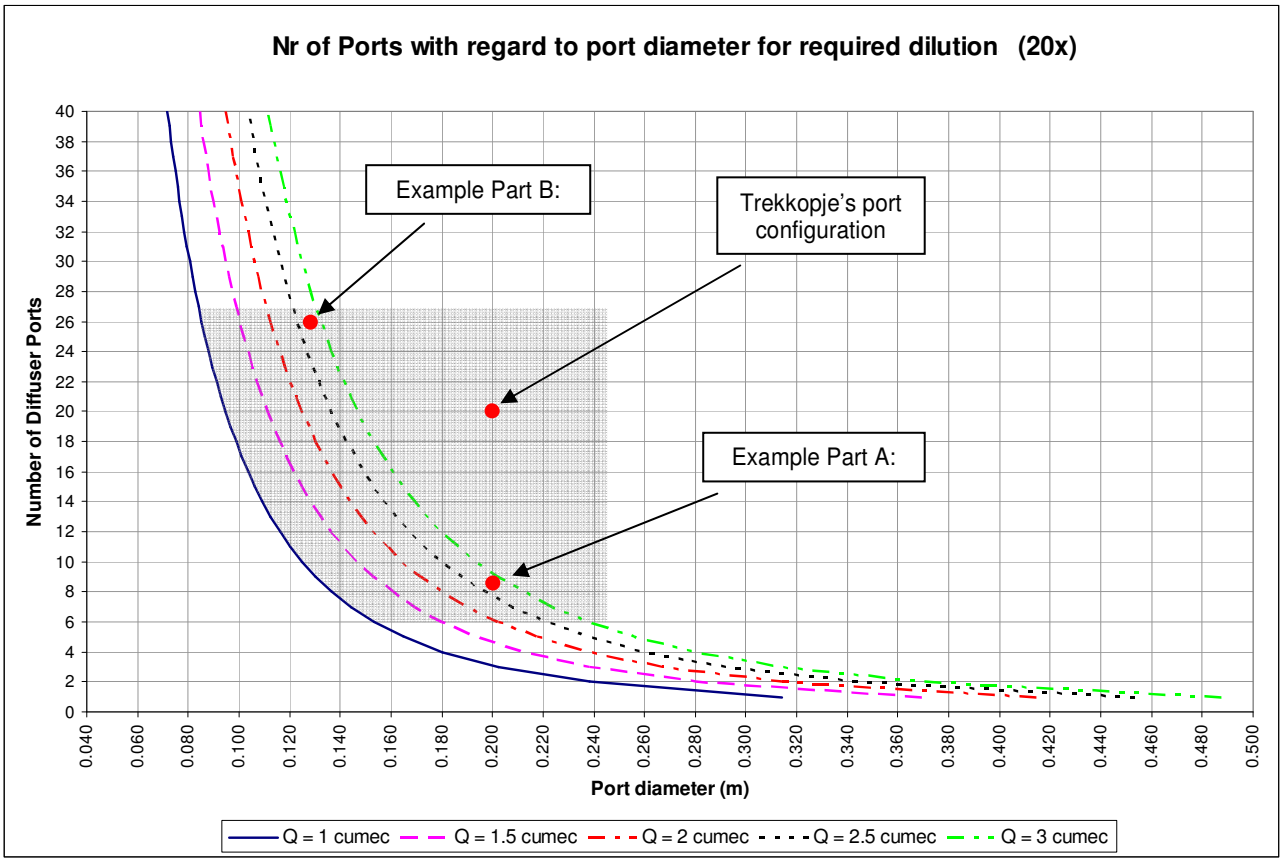


Figure 8.10: Relation between discharge volumes, ports diameters and required number of ports

9 DEVELOPMENT OF DESIGN APPROACH

This investigation aims to provide guidance for engineers which will assist them when designing marine structures for large scale seawater desalination plants in the form of a general design approach. The study provides a summary of existing information, guidelines, requirements and design codes which is relevant in terms the environmental as well as physical considerations for the various components. The overall design approach is provided in the form of a flow diagram where each step is referenced to the specific sections in this thesis where the aspects are discussed in detail. In addition, an assessment was undertaken to determine the feasibility of certain components of the intake and discharge system in order to make practical recommendations as part of the overall design approach. .

Although the aim is to provide assistance with the design of a seawater intake structure, brine outfall and marine pipelines, it is necessary for the engineer to understand the function and operations of the various desalination technologies which affect the required feedwater and discharge volumes and quality which in turn affect the environmental, hydraulic, operational as well as physical design, of the marine structures.

Even though certain components were investigated in detail, this investigation does not address all the relevant aspects of the entire process in such detail as required for a final design. Most of the components are addressed to some extent and reference is made to relevant codes, guidelines or additional information where appropriate.

Taking into account the literature study together with the analysis, the main objectives when considering the design of the seawater intake, brine discharge and marine pipelines are briefly summarized in the following sections, together with the flow diagram of the overall design approach.

9.1 DIRECT INTAKE DESIGN

A reliable feedwater supply system is a key factor for the successful operation of a seawater desalination facility. The intake structure itself should be designed to ensure the required flow and feedwater quality can be extracted while minimizing the environmental impact. Furthermore, the hydraulic and physical design should be optimized to ensure that future operational and maintenance requirements are cost effective and practical.

In order to optimize the intake structure, a number of considerations should be taken into account, such as the type of desalination technology, which affects the feedwater requirements. The specific environmental and physical characteristics of the site will determine the location, type of intake structure, required environmental mitigation measures as well as operational and maintenance procedures and finally the constructability of the intake.

Intake type selection

For large seawater desalination plants (e.g. capacity greater than say 100 000 m^3/day), it can be concluded that direct seawater intakes (extracting seawater directly from the ocean) is more feasible than indirect (sub-seafloor) intakes since direct intakes can normally extract greater feedwater volumes than indirect intakes.

Furthermore, direct intakes are sub-divided into two main groups: surface – and sub-surface intakes. Surface intakes are normally located next to the shore, protected by coastal structures such as breakwaters or in an estuary, whereas sub-surface intakes are located offshore and the feedwater transported to the desalination plant via an offshore marine pipeline. Both types have advantages and disadvantages which will have to be evaluated to ensure financial and operational feasibility. However, in principle, the main considerations are the required extraction rate and quality, the environmental affect and the financial implications of initial construction and maintenance requirements.

Depending on the site-specific environmental and physical characteristics, a surface intake located adjacent the shore may require substantial coastal structures such as breakwaters to protect the intake against wave action and the sediment regime. In principle, the structures are normally more expensive than an offshore intake with a connecting pipeline. However, the cost of maintaining an offshore structure is normally much more than an onshore structure and the construction costs of an offshore intake and the connecting pipeline increase substantially as the required extraction volumes increase. On the other hand, the more exposed the coastline and the rougher the seas, the greater the size of the protection (breakwaters) structures and the more difficult to construct, which subsequently affects the cost.

Therefore, it can be recommended that for a generally exposed coastline, with rough seas a sub-surface offshore intake would be more feasible. However, it becomes less feasible the greater the

required extraction volume. Subsequently, for co-located plants (desalination as well as power plants), a surface intake located at the shore would be more feasible taking into consideration the vast volumes required.

Location

A number of considerations will determine the optimum location of the intake structure, which should be a balance between the proximity to the desalination plant versus the environmental and physical requirements and characteristics.

Generally, the deeper the intake structure (furthest offshore) with the extraction point raised a few metres from the seafloor, the cleaner the feedwater (e.g. less sediment in suspension) and the less impact of wave forces on the structure, while ensuring the extraction point is submerged at all times. The primary treatment processes at the plant increase in complexity and cost as the feedwater quality decreases. However, the further offshore the extraction location, the more expensive the initial construction costs.

The location of existing ocean outfalls should be considered as this could affect the feedwater quality and especially the location of the brine outfall which could lead to re-circulation.

From an environmental point of view for a sub-surface direct offshore intake, the deeper the extraction point is located the less oxygen in the water and subsequently marine life. When selecting a location for a direct surface intake at the shore, the beneficial uses and environmental sensitive areas and possible negative affect on the coastline (alongshore sediment regime) have to be considered.

Finally, the proximity of the proposed location with regards to port and fishing activities together with popular navigation ship routes should be taken into account. Ship anchors can cause major damage to sub-surface structures, fishing nets can block the intake screens and pollution caused by vessels or port activities could impact the feedwater quality.

Physical design considerations for direct seawater intakes

The feasibility of a seawater intake design depends greatly on the constructability of the intake, taking into account the various constrains/limitations of construction and maintenance operations in the marine environment. Together with the above, the optimization of the hydraulic design and structural stability, the efficient mitigation of possible environmental impacts and cost effective operational and maintenance procedures will ensure that the sustainability of any type of seawater intake structure.

Figure 9.1 illustrates the design procedure and items to be addressed for the design of a direct seawater intake system. The various Sections refer to the specific design guidelines or general regulations, focussing on direct seawater intakes.

9.2 MARINE PIPELINES

If feedwater is extracted from an offshore intake structure, the feedwater is transferred via a tunnel or pipeline from the seawater intake to land and subsequently the brine waste stream is transported from

the outfall headworks (pump station or brine holding tank) through the main outfall pipe out to the diffuser.

Site-specific environmental processes such as currents, waves, sediment regime (which affects the seabed stability), seabed bathymetry and geophysical conditions will greatly affect the route selection, the structural design, and length and installation method of a marine pipeline.

It is generally recommended that the pipe should be buried in the surf zone at a sufficient depth to ensure it does not become exposed during its lifetime and is protected against wave attack. Beyond the surf-zone, the pipe could either lie on the seafloor or be buried, depending on the exposure and risk to damage from vessel anchors, etc.

The diameter of the pipe depends on the hydraulic analysis, ensuring the pipe flow full and taking into account the available head, whereas the wall thickness will depend on the internal and external pressures.

One of the main concerns of a marine intake pipe is the rate and extent of marine growth which will affect the inside pipe roughness and subsequently the hydraulic operations and maintenance procedures and cost. Sufficient head should be ensured beforehand by either pumping or gravity, taking into consideration the increase in pipe roughness. Furthermore, the type of pipe material and anti-fowling marine lining could also minimize marine growth. However, it is recommended to make provision for a pipe pigging system to mechanically clean the pipeline and subsequently it is also best practise to design a backup pipeline to prevent the desalination plant from shutdown while the pipe is mechanically cleaned. Based on Perth's case study, it was found that although chlorine dosing at the intake pipelines may reduce marine growth, it will not necessarily be successful in eliminating mussel growth.

Another main concern of an outfall pipeline is to prevent the deposition of sediment. This can be prevented by ensuring scouring velocities, or otherwise provision should be made for occasional flushing of the pipeline (discharging at high velocities).

9.3 BRINE OUTFALL

A number of discharge options for the brine stream of a desalination plant are available. However, this thesis only investigated the discharge of brine effluent though an offshore ocean outfall since it is normally the most feasible option for large desalination plants with large brine volumes.

The function of an ocean outfall it to discharge effluent while ensuring the impact to the environment (natural as well as beneficial uses) is minimized and the system adheres to the appropriate environmental guidelines, regulations and legislation.

The brine characteristics (volumes and quantity), proposed route, discharge location and achievable dilutions are the main elements which could negatively impact the environment and should be addressed accordingly.

Since the entire hydraulic process of the diffuser is an iterative process with each component influencing the following, this can best be done by a numerical model for the optimum design of a multi-port diffuser. With a numerical model it is possible to do a number of runs for a number of diffuser configurations to determine the optimum design.

In order to provide developers with an initial estimation of the diffuser configuration requirements, a method was provided, based on scientific theories and generally accepted environmental regulations, which will provide a rough idea of the required number of ports for a specific discharge flow rate and port diameters.

The requirements and estimated financial implications of providing intermittent discharge for an outfall system were also assessed in order to provide designers with recommendations to provide the most feasible solution.

Figure 9.2 illustrates the design procedure and items to be addressed for the design of a brine outfall. The various Sections refer to the specific design guidelines or general regulations. The design guidelines presented focuses on an ocean outfall since it is the most common and normally most cost effective.

10 CONCLUSIONS & RECOMMENDATIONS

10.1 CONCLUSIONS

The design approach for seawater intakes, brine outfalls and marine pipelines of large scale desalination plants which was developed from reviewing available literature, investigating the marine components of existing plants and specific analysis of certain components, are provided in the form of flow diagrams (Figures 9.1 and 9.2) with the reference to the relevant sections in this thesis.

Taking into account the growing strain on the natural fresh water resources together with the advanced technologies in the field of seawater desalination processes, it is expected that South Africa's fresh water resources for domestic, industrial and agricultural uses will have to be supplemented by desalinated seawater in the near future. Although this thesis does not claim to be a design guideline as such, it could contribute to more effective design of direct offshore subsurface seawater intakes and offshore brine discharge systems for large desalination plants. And in future, provide input to the development of South African Guidelines for the marine components of seawater desalination plants.

10.2 RECOMMENDATIONS

It is recommended to use the design approach developed in this thesis as a framework for designing large scale seawater intake systems and brine outfalls in order to save on initial costs & time.

It was concluded throughout this thesis that the marine fouling, such as mussel growth, could adversely affect the desalination filters and membranes, as well as the overall hydraulics. Furthermore, the extent of marine fouling components and subsequent mitigation measures is currently more of a trail-and-error approach. Therefore it is recommended that this aspect should be

investigated more thoroughly in future work in order to contribute to the optimization of seawater desalination plants.

It is further recommended that the affect of the brine effluent stream, taking into account not only the salinity concentrations, but also the overall characteristics of all the constituents (e.g. cleaning chemicals, chemicals incorporated in the desalination process, etc.) should be determined through laboratory testing. Subsequently, specific guidelines should be set which developers must adhere to in order to protect the marine environment for future generations.

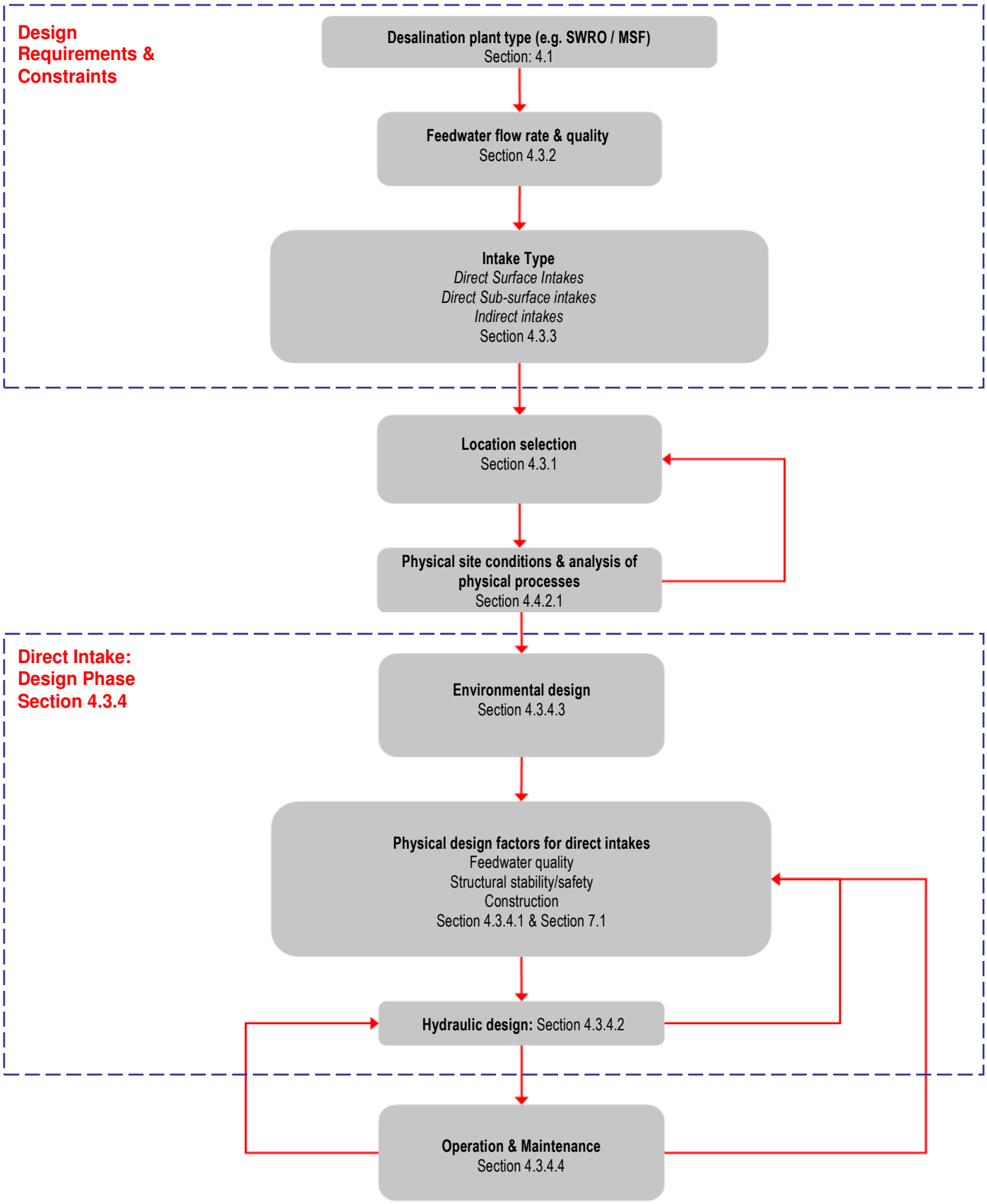


Figure 9.1: Design procedure for direct intakes

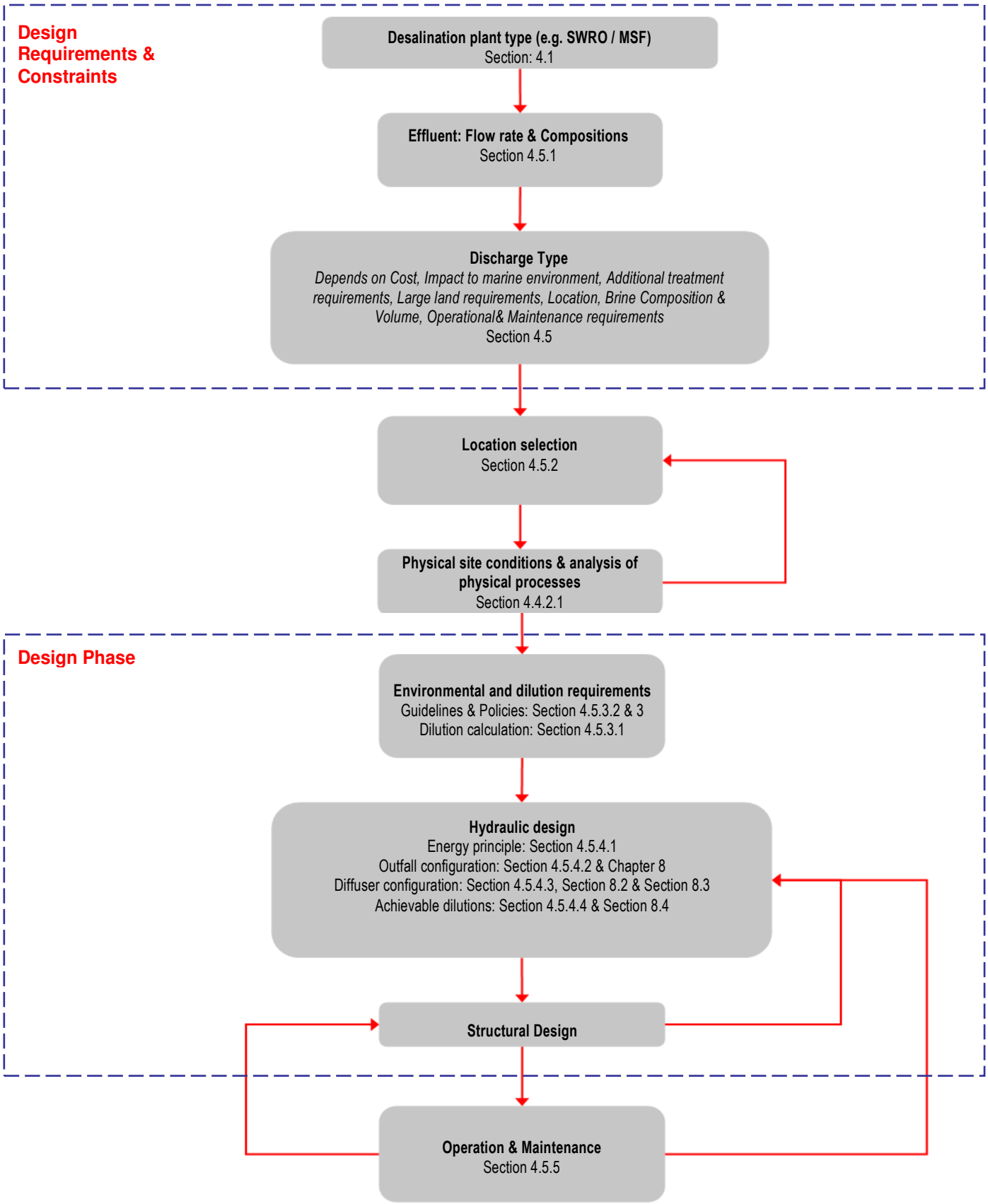


Figure 9.2: Design procedure for brine outfall

REFERENCES

ABDEL-JAWAD, M. (2001). Energy source for coupling with desalination plants in the GCC countries. Consultancy report prpaterd for ESCWA

AMTA. (2007). American Membrane Technology Association. Application of Membrane Technologies. Fact Sheets

APPLIED WATER MANAGEMENT GROUP (AMWATER). American Water. Fact sheet: AMER0158_Project%20Sheets_Tampa-5.2. www.appliedwater.amwater.com

AREIQAT, A. & MOHAMED, K.A. (2005). Optimization of the negative impact of power and desalination plants on the ecosystem. Elsevier 2005.

BECK, R.W. (2002). Demineralization Treatment Technologies fro the Seawater Demineralization Feasibility Investigation. Special Publication SJ2004-SP7. Feasibility Investigation Contract #SE459AA. December 2002.

BEVAN, J. (2005). The Professional Diver's Handbook. Second Edition.

BOSMAN, D.E. (2008) Report on Pressure Transient Analysis of Seawater Abstraction System and Comments on Marine Fouling (Rev C). URAMIN Desalination Plant, Namibia. May 2008

CHRISTIE, W.A.M (2009). Developer of WAMTech ocean outfall model. Personal communication.

BROOKS, N H (1960) Diffusion of sewage effluent in an ocean current. Proceedings of the First International Conference on Waste Disposal in the Marine Environment. Pergamon Press. pp 246-267

BUROS, O.K. (1982). The USAID desalination manual. Englewood, J.J., IDEA Publications.

BUROS, O.K. (2000). The ABCs of Desalting. 2nd Edition. International Desalination Association.

CHADWICK, A & MORFETT, J (1998). Hydraulics in civil and environmental engineering. Third Edition

CHRISTIE, S. (2009). Senior Engineer. Perth Seawater Desalination Plant. Personal communication via email

CRAIG, B., CIOFFI, S., RYBAR, S. (2008). Long term experience with membrane performance at the Larnaca desalination plant. Elsevier

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF) (1995a). South African water quality guidelines for coastal marine waters. Volume 1. Natural Environment. Pretoria

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF) (1995b). South African water quality guidelines for coastal marine waters. Volume 2. Recreation. Pretoria

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF) (1995c). South African water quality guidelines for coastal marine waters. Volume 3. Industrial use. Pretoria

DEPARTMENT OF WATER AFFAIRS AND FORESTRY (DWAF) (2004). Operational policy for the disposal of land-derived water containing waste to the marine environment of South Africa. Water Quality Management Series. Edition 1.

DESALINATION ISSUES ASSESSMENT REPORT (2003). Desalination, with a grain of salt - a California perspective. May 2003.

DIXON, A. (2007). Subsea Pipeline Construction. Port & Coastal Engineering. University of Stellenbosch. August 2007

DONEKER, R.L. & JIRKA, G.H. & Hinton, S.W. (2006). CORMIX: A Hydrodynamic Mixing Zone Model and Decision Support System for Pollutant Discharges into Surface Waters.

EINAV, R. & HARUSSI, K. & PERRY, D. (2002). The footprint of the desalination processes on the environment. Elsevier 2002.

EM 1110-2-3001 (1995). Planning and Design of Hydroelectric Power Plant Structures. Coastal Engineering Manual. April 1995

HAMANO, T (2004). Feature and New Technology for Fukuoka Desalination Plant. Assoc. of Fukuoka Water-Supply Enterprise

HEATON, K (2005). The Winston Churchill Memorial Trust of Australia.

KWI HYURK, K. (2009) General Manager. Doosan Heavy Industries & Construction. Personal communication via email

LATTERMANN, S. & HÖPNER, T. (2007). Environmental impact and impact assessment of seawater desalination. Elsevier 2007.

LAVAL, B. & HODGES, B.R. (2000). The CWR Estuary and Lake Computer Model. User Guide. The University of Western Australia. September 2000.

LE ROUX, M (2005). Development of a Procedure for an Ocean Effluent Outfall Model for application as an Analysis and Management Tool. Final year project. University of Stellenbosch

MELBOURNE WATER, GHD. 31/20622/132863 Seawater Desalination Feasibility Study

MISSIMER, T.M. (2008). Alternative Subsurface Intake Designs for Seawater Desalination Facilities. Alden Reserch Laboratory. Desalination Intake Solutions Workshop. Worcester, Massachusetts.

MOSTERT, H. (2009). Pipe material selection. Pipeline and Pumpstation Design Short Course. University of Stellenbosch. April 2009

PANKRATZ, T. (2008). An Overview of Seawater Intake Facilities for Seawater Desalination. Alden Desalination Intake Solutions Workshop, Holder, Massachusetts. October 2008.

PETERS, T. & PINTO, D. (2008). Seawater intake and pre-treatment/brine discharge – environmental issues. Elsevier 2008.

POSEIDON RESOURCES CORPORATION (2004). Carlsbad Seawater Desalination Project, Alternatives to the Proposed Intake (March, 2004).

ROBERTS, P.J.W. & TOMS, G. (1987). Inclined dense jets in a flowing ambient. ASCE Journal of Hydraulic Engineering, Vol. 113, No. 3.

SA NAVY (2009). South African Tide Tables. Published by the Hydrographer South African Naby. ISBN 97809584817-4-8

SANZ, M.A. & BONNELYE, V. & CREMER, G. (2007). Fujairah reverse osmosis plant: 2 years of operation. Elsevier.

SAUVET-GOICHON, B (2007). Ashkelon Desalination plant - A successful challenge. Elsevier

SMITH, J.C. (2004). Hold The Salt, The Promise of Desalination For Texas. Research Report, October 2004. Texas Plublic Policy Foundation

STANIMIROV, M. Pipelife Norge AS. Personal communication via email

STRATEGEN (2004). Metropolitan Desalination Proposal Section 46 Review. Prepared for Water Corporation by Strategen. February 2004.

SW455 04/08 (2008). Sydney Water Factsheet

SW8 04/08 (2008). Sydney Water Factsheet - Cosntruction work

SYDNEY COASTAL COUNCILS GROUP INC. (). Desalination Fact Sheet

SYNTHESIS PAPER – Management Practices for Feedwater Intakes and Concentrate Disposal for Seawater Desalination.)

THE MAGAZINE OF THE SCIENTIFIC CHRONICLES NO. 4 (2005). Research and Development (July-August 2005).

TM 5-814-3 (1988). Domestic Wastewater Treatment. Department of the Army Technical Manual

TS-01A (2005). Sydney's Desalination Project. Scope of work D&C Separable Portion 3 (Rev 3)

TSIOURTIS, N.X. (2004). Desalination: The Cyprus Experience. European Water 7/8: 39-45, 2004.

INSTITUTE OF WATER AND ENVIRONMENTAL ENGINEERING DEPARTMENT CIVIL ENGINEERING, UNIVERSITY OF STELLENBOSCH (W&E US) (2008a). Report on Hydraulic Model Tests on Proposed Seawater Intake Structure for the Wlotzkasbaken Desalination Plant. Part 1: Stability Tests on Caisson Structure Under Wave Action. September 2008

INSTITUTE OF WATER AND ENVIRONMENTAL ENGINEERING DEPARTMENT CIVIL ENGINEERING, UNIVERSITY OF STELLENBOSCH (W&E US) (2008b). Report on Hydraulic Model Tests on Proposed Seawater Intake Structure for the Wlotzkasbaken Desalination Plant. Part 2: Intake Hydraulics: Energy losses and Chlorination flow. September 2008

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY (US-EPA) (1985) Initial mixing characteristics of municipal ocean discharges. Environmental Research Laboratory, Naragansett, Report No EPA/SW/MT-86/012(a & b).

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA Final Rule (2001). 40 CFR Parts 9, 122, et al. National Pollutant Discharge Elimination System: Regulations Addressing Cooling Water Intake Structures for New Facilities; Final Rule. December 2001.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA Final Rule (2004). 40 CFR Parts 9, 122 et al. National Pollutant Discharge Elimination System—Final Regulations To Establish Requirements for Cooling Water Intake Structures at Phase II Existing Facilities; Final Rule. July 2004.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA. Final Rule (2006). CFR Parts 9, 122, 123, et al. National Pollutant Discharge Elimination System; Establishing Requirements for Cooling Water Intake Structures at Phase III Facilities; Final Rule. June 2006.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA. TDD (). EPA-821-R-06-003. Technical Development Document for the Final Section 316(b) Phase III Rule.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA TDD (2001). EPA-821-R-01-036. Technical Development Document for the Final Regulations Addressing Cooling Water Intake Structures for New Facilities. November 2001.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY. EPA TDD (2004). EPA 821-R-04-007; DCN 6-0004. Technical Development Document for the Final Section 316(b) Phase II Existing Facilities Rule. February 2004.

WANGNICK, K. (2004). 2004 IDA Worldwide Desalting Plants Inventory No. 18", June 2004, published by Wangnick Consulting

WATER MANAGEMENT CHALLENGES IN THE LORATO REGION. Baja California, Mexico.

WATER RESEARCH COMMISSION (WRC, 1992). Guide for the Marine Disposal of Effluents through pipelines

WSP (2009). Marine Intake Works and Brine Discharge for Desalination Plant. Trekkopje Uranium Mine. Design Report. Areva Resources Namibia. March 2008

www.tampabaywater.org

www.water-technology.net The website for water and wastewater industry

APPENDICES

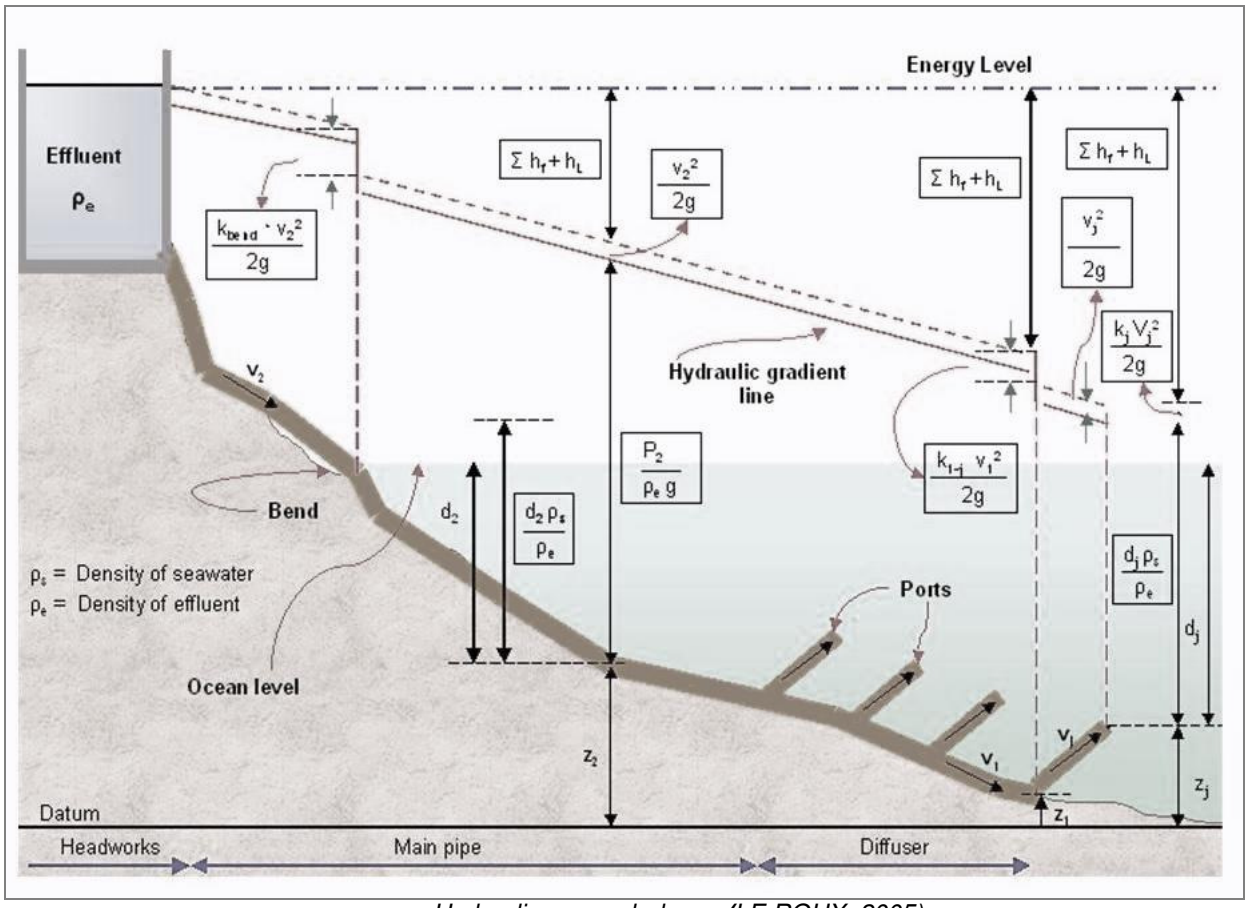
APPENDIX A: MARINE PIPELINES - HYDRAULIC DESIGN

Energy Principle

The energy principle is represented by the Bernoulli equation for pipe flow:

$$v_1^2/2g + p_1/\rho g + z_1 = v_2^2/2g + p_2/\rho g + z_2 + h_f + h_L$$

where:
- v = Q/A average velocity
- p = intensity of pressure at centre-line
- ρg = specific weight
- z = centre line elevation
- h_f = frictional head loss (in terms of Chezy or Manning or Darcy Weisbach - Colebrook White)
- h_L = local head loss (losses occurring where the flow velocity changes in magnitude or direction)



Hydraulic energy balance (LE ROUX, 2005)

Continuity Equation

The continuity equation for steady incompressible flow is of use for effluent discharge:

$$\Sigma Q_{IN} = \Sigma Q_{OUT}$$

Determine Pipe Diameter for Estimated Flow

*The following equation is used to determine the estimated flow (**A. Chadwick and J. Morfett - 1999**):*

Q = VA

Where Q = flow m/s^2
V = 0.7 m/s
$A = \pi D^2/4$
D = pipe diameter (m)
Thus: $D = (4.A/\pi)^{1/2}$

Frictional Head Loss
The Darcy-Weisbach, Colebrook-White is the appropriate (generally accepted most accurate) formula to be used to determine friction losses in pipes *A. Chadwick and J. Morfett - 1999).*

Darcy-Weisbach:

$$h_f = (\lambda L\ v^2) / (2gD)$$

where: h_f = headloss due to friction
λ = friction factor
v = effluent velocity
D = diameter of pipe
L = length of pipe
g = acceleration due to gravity

λ is called the pipe friction factor and can be determined from the Moody Diagram or by applying the Colebrook-White formula:

Colebrook-White formula:

$$\lambda = 0.25\{\log10\{k_s / 3.7D + 2.51 / (R_e\lambda^{1/2})\}]^{-2}$$

where: k_S = roughness height (mm)
R_e = Reynolds number

Reynolds number

$$R_e = \rho vD/\mu$$

where: ρ = Fluid density (kg/m^3)
v = Velocity (m/s)
D = Diameter (m)
μ = Viscosity (kg/m.s)

Local head loss h_L

For fully turbulent flow in pipes, the headloss due to friction and turbulence at inlets, converging sections and bends in the pipe can be determined with the following equation (WRC – Water Research Centre - 1990):

$$h = kv^2/2g$$

where: h = headloss across the fitting
v = mean upstream velocity
k = non-dimensional loss coefficient (Appendix 7)

Calculating the Froude number

Fr = vp/[g.dp(Δρ/ρs)]1/2

Where

vp = port velocity (m/s)
Δρ = ρs - ρe
ρs = seawater density (kg/m3)
ρe = density of wastewater (kg/m3)
dp = port diameter (m)

APPENDIX B: MARINE PIPELINES – STRUCTURAL DESIGN

Wave force calculations (*WSP, 2009*)

The hydrodynamic forces acting on the pipe can be represented by the ***Morrison equations***. These equations are functions of time since the orbital velocity and acceleration change throughout a wave cycle and are 90° out of phase:

$$F_m = \tfrac{1}{4}\pi D^2 C_m \rho_w (a_s \sin\alpha)$$
$$F_d = \tfrac{1}{2} C_d \rho_w D (u_s \sin\alpha + U_c) \left| (u_s \sin\alpha + U_c) \right|$$
$$F_l = \tfrac{1}{2} C_l \rho_w D (u_s \sin\alpha + U_c)^2$$

where

F_m	is the inertia force as function of time (N/m)
F_d	is the drag force as function of time (N/m)
F_l	is the lift force as function of time (N/m)
C_m	is the inertia force coefficient
C_d	is the drag force coefficient
C_l	is the lift force coefficient
ρ_w	is the density of sea water (1025 kg/m^3)
D	is the effective pipe diameter (m)
a_s	is the significant horizontal orbital acceleration as a function of time (m/s^2)
u_s	is the significant horizontal orbital velocity as a function of time (m/s)
U_c	is the steady current velocity perpendicular to the pipe axis (m/s)
α	is the angle of the wave orthogonal relative to the pipe axis (degrees)

According to DNV's quasi-static Simplified Method of Analysis (DNV,1988) the stability of the pipeline can be determined, assuming a ***Simple Coulomb Friction model***, using the following expression:

$$(W_s / f_w - F_l)\,\mu \geq F_d + F_m$$

where

W_s	is the submerged weight of the pipe (N/m)
f_w	is a calibration factor
μ	is the soil friction factor

The limiting value of the submerged weight can then be found from:

$$W_s = [(F_d + F_m + \mu F_l) / \mu]_{max}\ f_w$$

The calibration factor f_w varies between a value of 1,0 and 1,6 and includes a safety factor of 1,1 according to DNV (1988). It is a function of two variables: ***the Keulegan Carpenter number (K)*** and the ***current to wave velocity ratio (M)***. These are given by the following expressions:

$$K = u_s T_u / D$$
$$M = U_c / u_s$$

where
T_u is the zero-upcrossing wave period
and the other parameters are as defined above.

Allowable stresses

The ***equivalent stress*** is defined using **von Mises equation**:

$$\sigma_{eq} = [\sigma_L^2 + \sigma_H^2 - \sigma_L \sigma_H + 3\tau 2]\ ½$$

where σ_{eq} = equivalent stress
σ_L = hoop stress
σ_H = longitudinal stress
τ = shear stress.

Equivalent stresses at various positions along the pipeline span can be determined using the relevant stresses σ_L, σ_H and τ at each position. The ***biaxial bending*** of the pipe can be checked to satisfy the equation:

$$M_{ux}/M_r + M_{uy}/M_r < 1.0$$

Where M_{ux} = bending moment about x-axis (in horizontal plane)
M_{uy} = bending moment about y-axis (in vertical plane)
M_r = bending moment required to cause buckling

Loadings

The loadings acting on the pipeline are:

- selfweight of pipeline and the water inside the pipe
- internal negative pressures caused by the suction of pumps
- transient pressure effects from waves entering the pipe were found to be less than the loading from a power failure
- transient pressure effects from power failure
- hydrostatic pressure from the external water pressure equivalent to 100kPa for 10m water depth
- accidental anchor loads
- horizontal inertia wave loading
- Loadings acting on the pipe at the same time were combined into the following load cases:
    - Selfweight of pipe + water inside pipe + lateral wave loading + internal negative pressures for normal operation + hydrostatic external pressure
    - Selfweight of pipe + water inside pipe + lateral accidental anchor loading + internal negative pressures for normal operation + hydrostatic external pressure
    - Selfweight of pipe + water inside pipe + wave loading + pipe internal negative pressure due to power failure + hydrostatic external pressure

APPENDIX C: EXISTING DESALINATION PLANTS

Cyprus: Larnaca

The plant extends over an area of 160 x 100 metres and comprises of the following treatment processes:

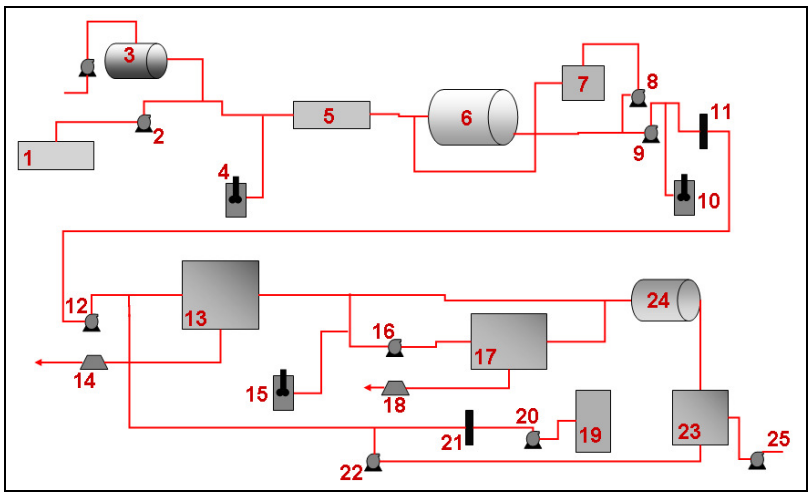


Figure C.1: Cyprus SW desalination plant processes (Refer to Table C.1)

Components of Cyprus SW desalination plant

1	Seawater intake	14	Energy recovery turbine first pass
2	Seawater pumps	15	Anti-scalant dosing system for second pass
3	Sulfuric acid dosing system	16	High-pressure pumps for trains in second pass
4	Coagulant dosing system	17	RO trains in second pass
5	Mixer room	18	Energy recovery turbine second pass
6	Open gravity sand filters	19	Chemical cleaning tank
7	Backwash tank for sand filters	20	Chemical cleaning pumps
8	Booster pump for sand filters backwash tank	21	Cartridge filter (from chemical cleaning system)
9	Booster pumps	22	Diesel pump for train flushing in case of energy power failure
10	Anti-scalant dosing system for first pass	23	Permeate water tank
11	Cartridge filters	24	Limestone gravel reactors
12	High-pressure pumps for trains in first pass	25	Permeate pump for distribution to the city
13	RO trains in first pass		

- *Preliminary treatment*: The raw feed water has a salinity of 40 500 ppm and intake temperature between 18 and 26 °C. Particles larger than 2 mm are removed from the inflow by a rotary screen.
- *Pre-treatment:* The feed water is dosed with flocculent together with sulphuric acid while being pumped to the flocculation chamber in order to ensure the seawater arrives at the flocculation chamber in optimum condition for treatment. The feed water is pre-treated by means of two flocculation chambers and 12 dual-media gravity filters.
- *RO process:* After pre-treatment, the filtered water collets in a tank from which four booster pumps drive the water through 10-micron filters before it passes to the desalination membranes via high pressure pumps. The plant comprises of five RO trains with a rated production of 9 000 m^3/day each and consists of 120 pressure vessels arranges in 12 rows and containing eight SWRO membranes each.

- *Final treatment:* The desalinated water is further treated by adding dissolved calcium carbonate in order to adjust the calcium and bicarbonate ions and improve the taste. Finally, the water's pH is adjusted and chlorinated before being pumped the a storage tank.
- *Power consumption:* Since power consumption is one of the major factors which affects the product water pricing, the plant has been designed to ensure that the power consumption for the desalination process itself is reduced to 3.4 kWh/m³ of product water with an overall power consumption, including all pumping, of 4.35kWh/m³.

Large pipes were imported to filter the water. The output capacity was upgraded to 54,000m³/day

The seawater pumping station under construction (© Copyright 2009 SPG Media Limited)

The water plant at Larnaca was protected with epoxy coated reinforcement

Picture courtesy of Trygve Blomster; Export Manager ;Pipelife Norge AS

Picture courtesy of Trygve Blomster; Export Manager ;Pipelife Norge AS

Picture courtesy of Trygve Blomster; Export Manager ;Pipelife Norge AS

Japan: Fukuoka

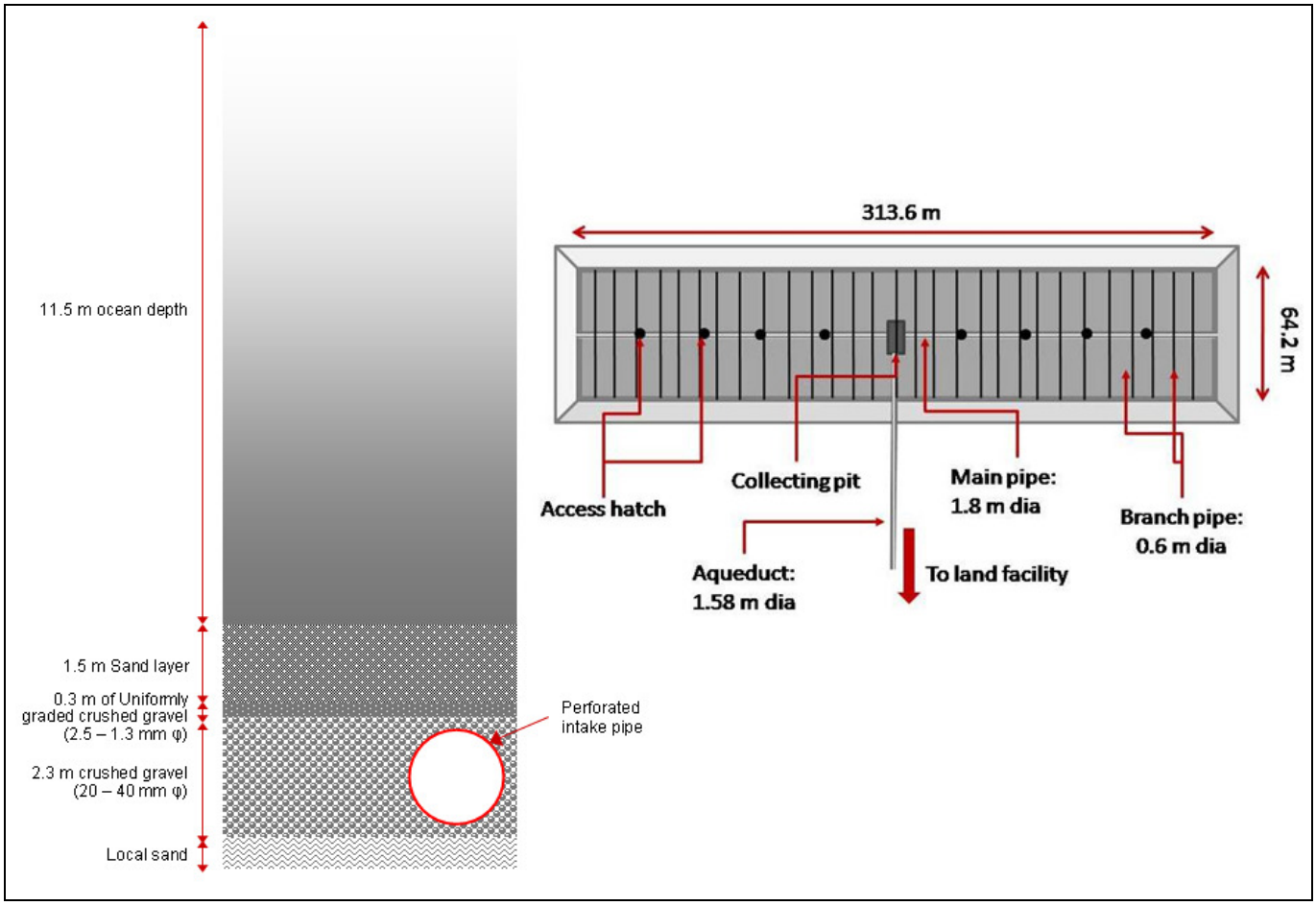
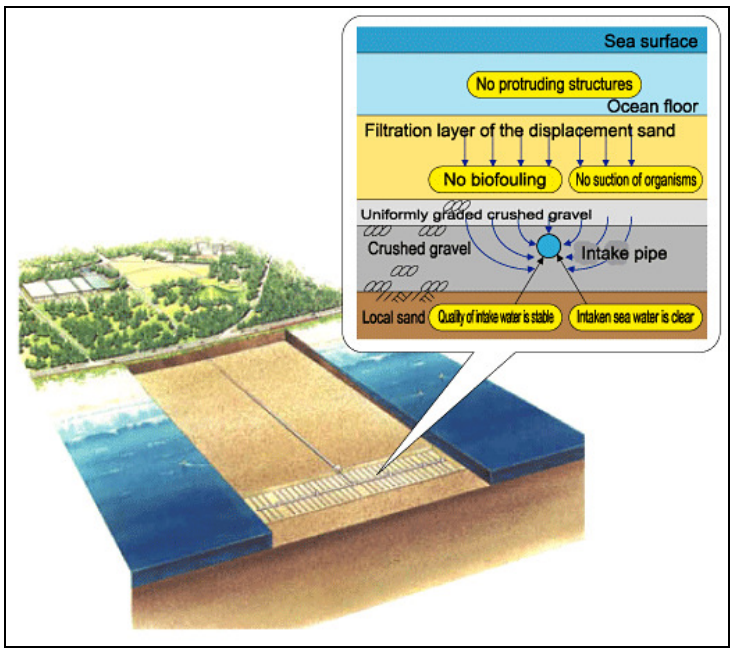


Figure C-2: Cross section and plant view of infiltration gallery

The facility consists of the following treatment processes:

- *Pre-treatment*: UF (Ultrafiltration) membranes remove microorganisms and ultrafine particles, therefore the need to add coagulants are not neccesary. The membranes produce no waste and require less space for installation.
- *RO process:* The two-stage reverse osmosis system consists of a high-pressure and low-pressure system in order to produce a higher quality of water.

The only main operational problem which was encountered was that the stainless steel (SS316) pipes which were designed to transport the brine stream, which is about 2.5 times the salt concentration of the ambient seawater, corroded. The corrosions occurred mainly in flanges and at "stagnant" areas in the pipeline (i.e. dead ends). The problem was addressed by replacing the stainless steel pipes with coated steel pipe sections and using titanium on the flange faces.



Fukuoka indirect intake structure (© F.D.W.A. All Rights Reserved)

Saudi Arabia: Shoaiba

 The overall development at the Shoaiba site also includes an oil-fired power station, together with a port and a tanker terminal (© Copyright 2009 SPG Media Limited)	 One of the 20 vertical mixed-flow brine re-circulation pumps. In all, over 100 pumps of varying designs and sizes were used in the plant (© Copyright 2009 SPG Media Limited)
 Model test of marine outfall structures (COWI Marine and coastal engineering)	 Outlet weirs (COWI Marine and coastal engineering)

UAE: Umm Al Nar

The Umm Al Nar Power Company is the largest single producer of water in the United Arab Emirates (© Copyright 2009 SPG Media Limited)

A total of 72 pumps were supplied, including 25 small flow rated units like this one (© Copyright 2009 SPG Media Limited)

The acquisition of 40% interest by the International Power led consortium is part of the privatisation process of Abu Dhabi's power and water industries (© Copyright 2009 SPG Media Limited)

Israel: Ashkelon

A dedicated gas turbine power station was built adjacent to the desalination plant; an overhead line provides a second supply from the Israeli national grid (© Copyright 2009 SPG Media Limited)

Tow departure from Norway (Pipelife Norge AS)

Installation (Pipelife Norge AS)

UAE: Fujairah

An effective pre-treatment process of the feed-water is required since mineral and organic micro-pollutants can clog the filtering membranes of the RO system. The pre-treatment process comprises of three filter layers: one pumice stone filter and two sand filters of different densities which filters the water at a rate of 12 m^3/second

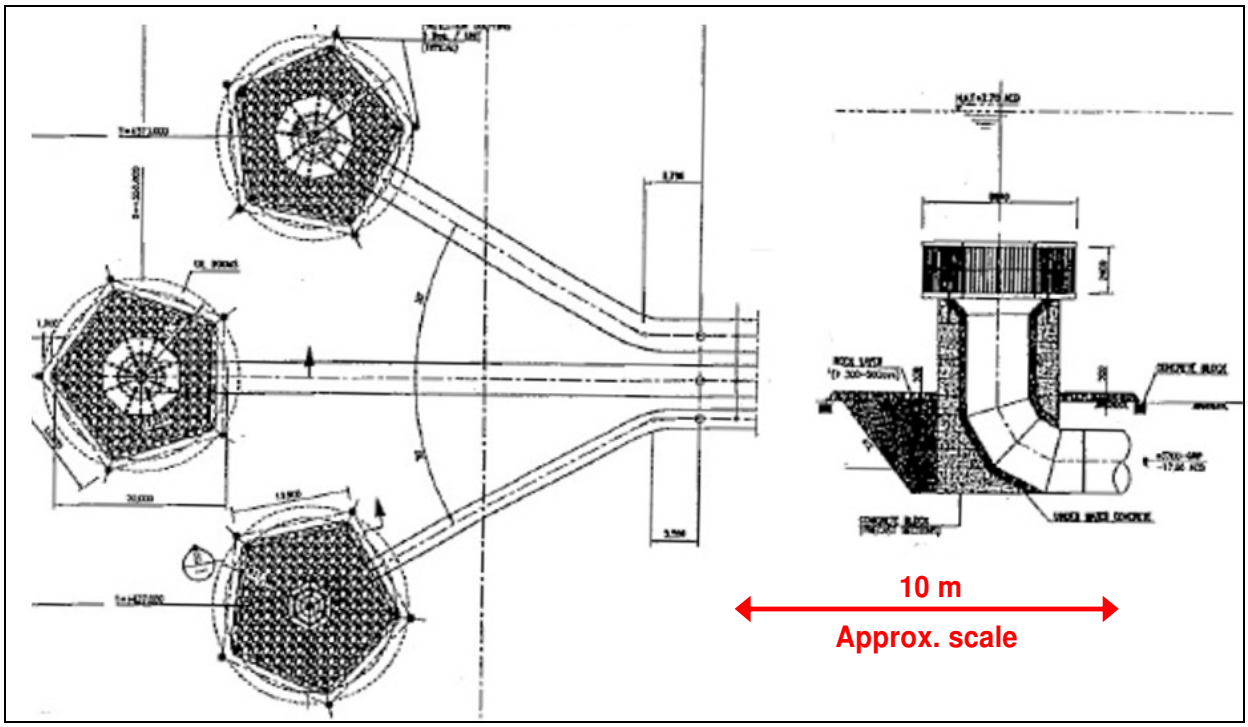


Figure C-3: Intake design (PANKRATZ, 2008)

Tom Pranktz

Molluscs and marine organisms being removed from the intake pipeline at Fujairah Desalination Plant by divers air-blasting the internal pipe walls. a) Three of these baskets were filled during a 20 minute period of observation of the cleaning operation. b) Assorted molluscs retrieved from the baskets with evidence of reproduction (Heaton 2005)

Australia: Perth

The feed water first passes through a pre-treatment filter to protect the pores of the membranes, before being forced through the spiral wound membrane elements of the RO treatment trains. After treatment, the product water is treated with lime, chloride and fluoride before being stored and subsequently integrated with the municipal supply system.

Electricity for the desalination plant comes from a Wind Farm located 30km east of Cervantes.

RO Process:

Pre-treatment:

- Phase 1: Filtration through 24 large duel media pressure filters
- Phase 2: Filtration through final nutrient/contaminant removal through 14 cartridge filter vessels

RO

- 1st pass reverse osmosis through 12 racks operating at pressures up to 64 bar
- 2nd pass reverse osmosis through 6 racks operating at pressure up to 15 bar

Final treatment

- Disinfection facility (chlorine)
- Stabilisation facility using lime and carbon dioxide injection
- Fluoridisation using fluorosilicic acid

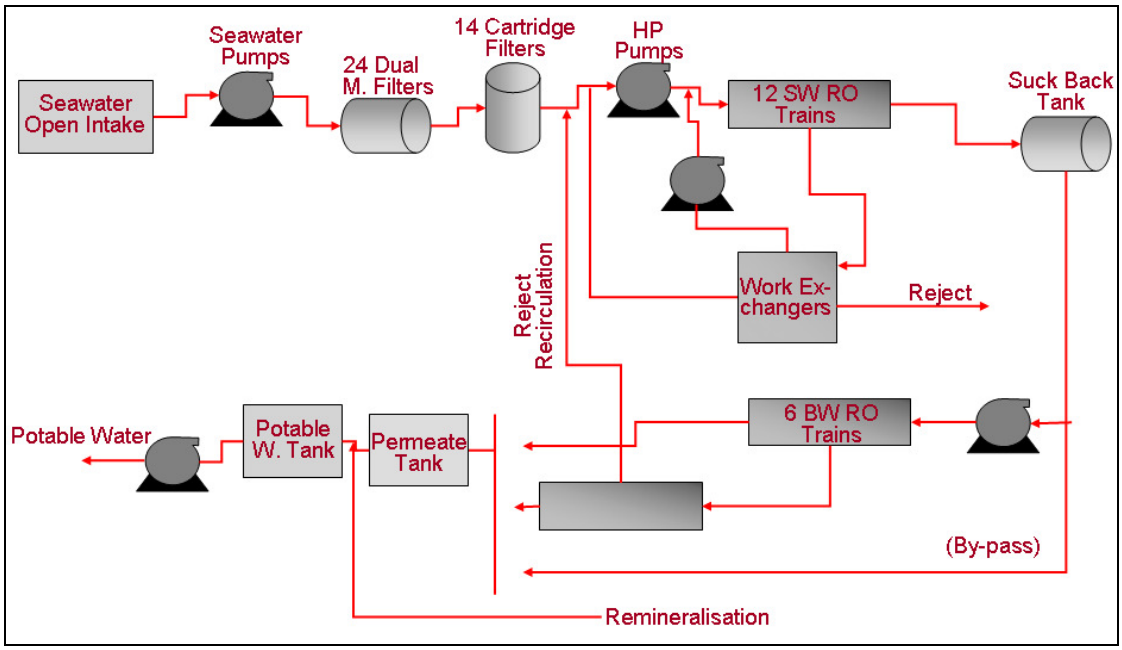


Figure C-4: Desalination process of Perth Desalination –Degremont

Aerial view of the new plant during construction. The completed facility supplies 17% of Perth's needs and is the largest single contributor to the area's integrated water supply scheme (© Copyright 2009 SPG Media Limited)2009 SPG Media Limited)

Pump station (Tom Pranktz)

Pump station (Tom Pranktz)

Installation of intake structure (Tom Pranktz)

Installation of intake structure (Tom Pranktz)

Extent of Marine growth on Intake structure

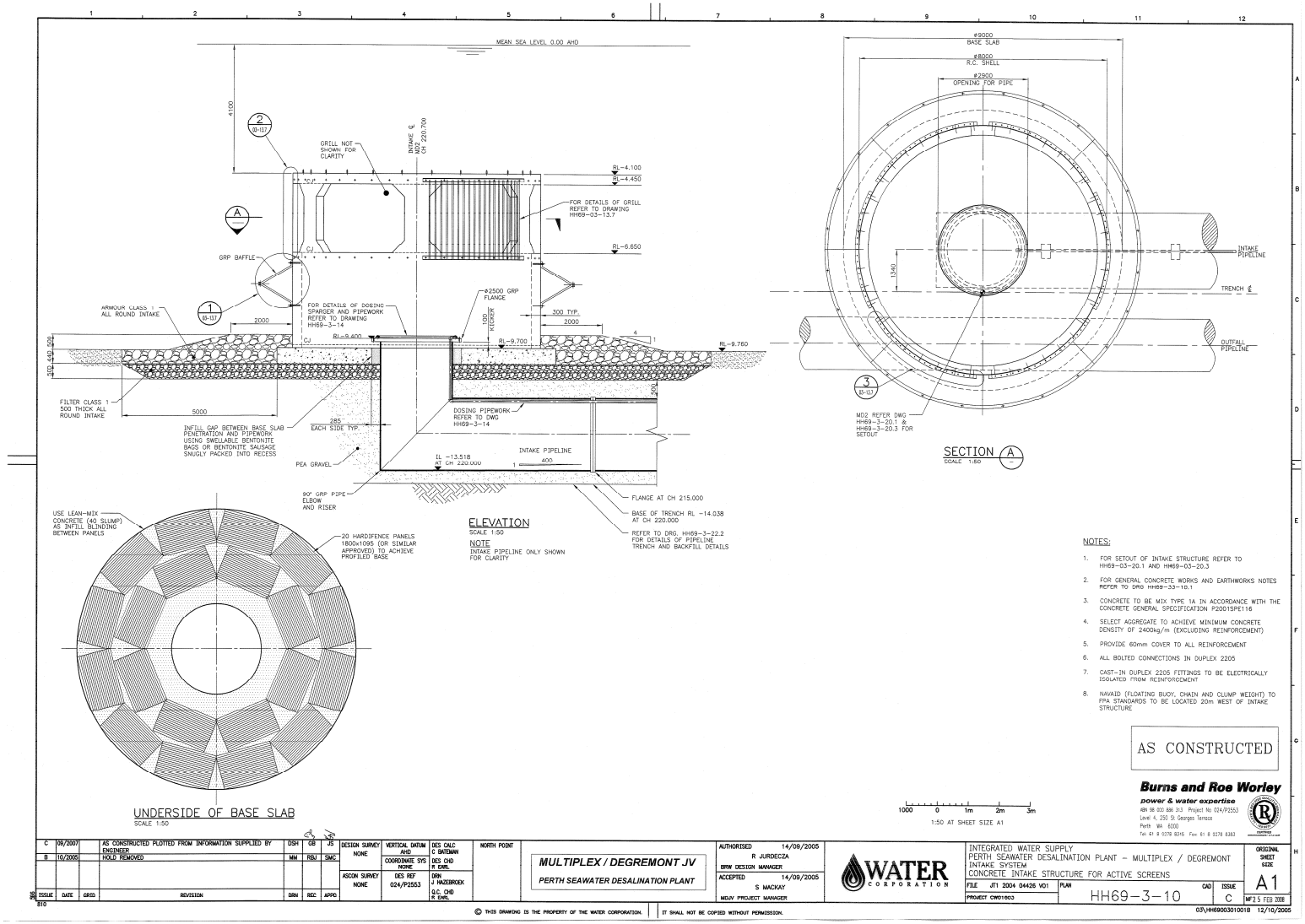


Figure C-5: Intake structure of Perth desalination plant

Australia: Sydney

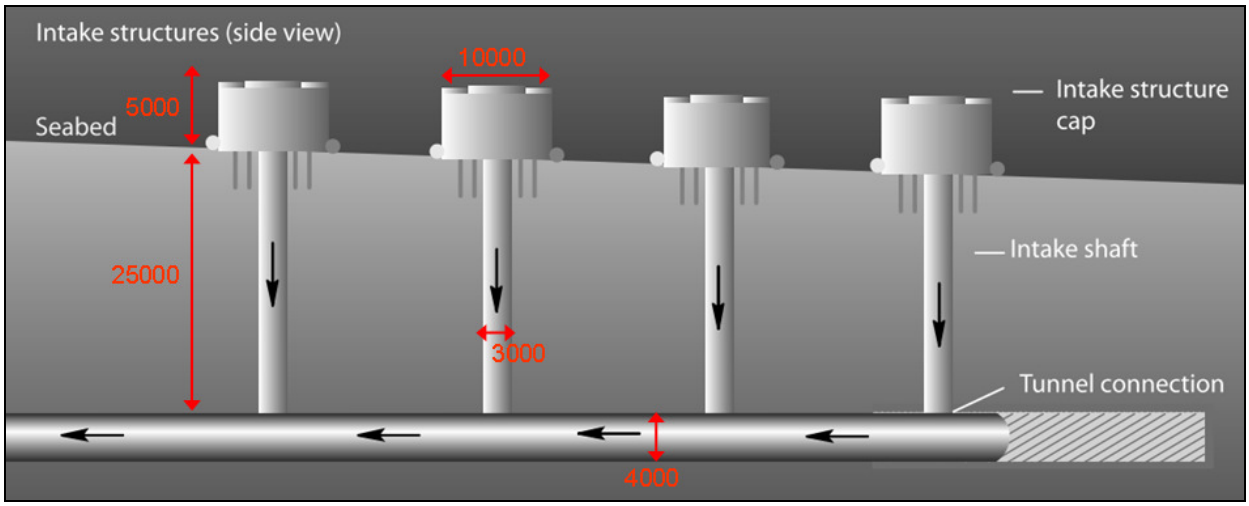
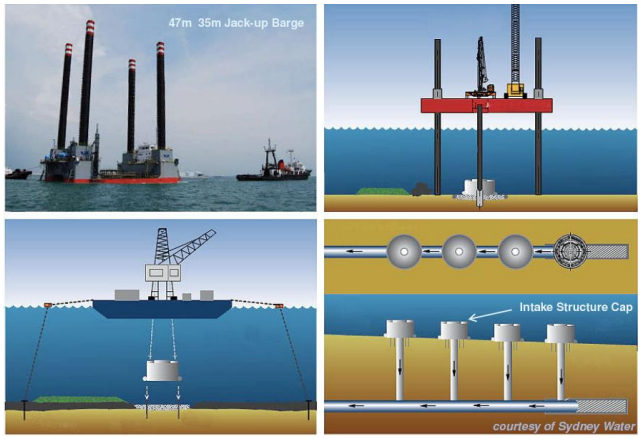


Figure C-6: Intake configuration of Sydney desalination (SW8 04/08, 2008)

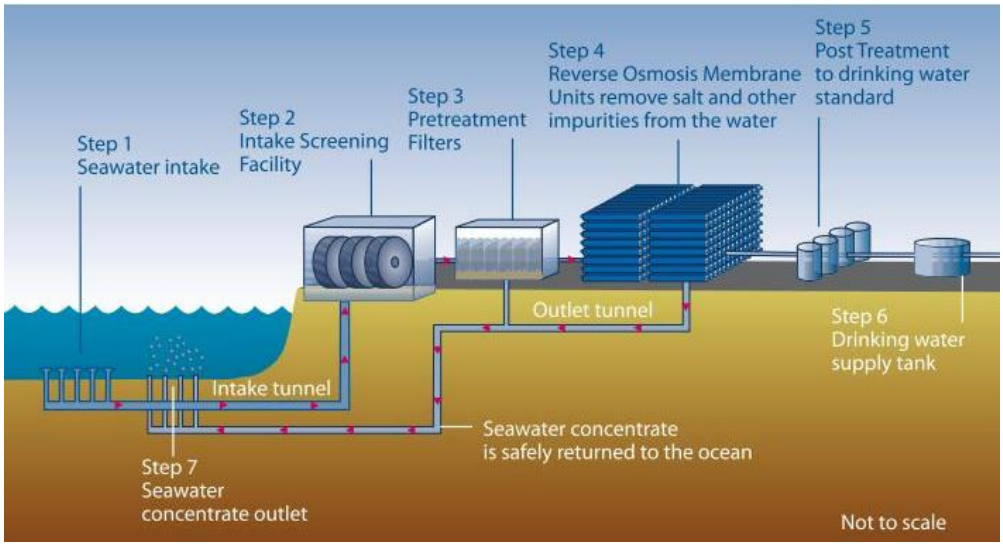
Desalination plant (Sydney water website)

Intake structure (Sydney water website)



Installation method (Sydney water website)

The jack-up barge off Cape Solander (Sydney water website)	



Schematic presentation of desalination process (Sydney water website)

USA: Tampa Bay

The membrane facility comprises of a two-stage sand filtration intake adjacent the neighboring power plant's four discharge tunnels, a concentrate discharge system, various chemical storage and dosing facilities, a 24 km product water transmission main, pre-treatment, flocculation and sedimentation systems.

Pre-treatment: The power plant provides fine grating screens for its cooling system. The feedwater for the desalination facility travels through a multi- step screening and settling process to clear out shells and other debris. Subsequently, chemicals are added to the water before it travels through sand filters to remove smaller particles, and finally through diatomaceous earth filters and cartridge filters to eliminate microscopic materials. *(AMWATER)*

RO process: To remove the salt, the facility uses a RO process, using high pressures to force the pre-treated water through semi-permeable membranes that trap salt and other minerals and allow the purified water to move to the final treatment stage.
Final treatment: The final phase of the process puts minerals back into the treated water and pump it to Tampa Bay Water's facilities site where it is combined with treated water from other sources.

APPENDIX D: TREKKOPJE: MARINE STEEL PIPES

Marine steel pipes:

A batch (between 9 and 12 pipes) of 1.2 m OD steel pipes was delivered on a weekly basis. In total, 145 marine steel pipe sections were delivered to site for the 3 marine pipelines. The 18 metre steel pipes were welded together to from about 160m pipe strings. A corrosion coating and lining were applied to the steel pipes at the pipe manufacturer and any damages during transportation, or at the field joints were repaired at site. After the welding and coating repairs, a 150mm thick concrete weightcoat were casted around the pipes. Finally, sacrificial anode bracelets were also bolted at certain intervals along the pipes for additional corrosion protection.

Drilling, blasting and excavation of pipe trench:

The land-side pipe trench area was surveyed before the blasting operations commenced and the rock level while drilling the blast holes in order to determine the soil and rock volumes. The area of blasting was set out by a surveyor and the top soil was excavated prior to drilling. Holes were drilled on a 1.5m x 2m grid to about 1 m deeper than the design level of -3.55 m MSL and charged with explosives. The trench through the surf zone was drilled and blasted from a cantilever bogey resting on a jetty, which extended over the entire width of the pipe trench. After blasting, the trench was excavated to the required depth with a dredging grab.

Diffuser:

The offshore diffuser section was manufactured in Johannesburg and arrived on site in November 2008. The diffuser's main pipe was coated at the manufacturer and all ports was welded and coated on site.

GRP pipe:

GRP Vectus pipes will be laid for the pipeline section from the high water mark up to the pump station. The pipes were delivered to site and stored on a level area supported by wooden piles as specified.

5 June 2008: Welding machine - Invertec STT10

12 June 2008: Trial reinforcement section

12 August 2008: Welding plinths

21 November 2008: Stringing yard

19 November 2008: Stringing yard

26 November 2008: Weightcoated steel pipes

APPENDIX E: TREKKOPJE: CAISSON

Caisson construction:

A 24m x 10m x 4m concrete caisson placed about 1.2 km offshore, will serve as the offshore anchor during the pipe-pull operation and afterwards function as the intake headworks of the seawater system. The caisson was floated from a floating NamPort dry-dock in Walvis Bay where it was constructed in the dry. In middle November it was berthed at the NamPort quay. A week later, the caisson was towed with a tugboat from Walvis Bay to its final position at Wlotzkasbaken (about 1.2 km offshore). On arrival, it was noticed that the caisson lost about 200 mm of freeboard on the sheave side (offshore side) while being towed from Walvis Bay. Since the structure seemed stable enough, the contractor decided to sink the caisson as close as possible to the intake location and perform the final repositioning at a later stage. Following a number of unsuccessful re-floating and re-positioning operations, the caisson was placed in its correct position in April 2009. After the caisson placement, eight of the chambers were filled with sand and grout bags were placed underneath the bottom of the caisson for support. The manufacturing of the stanchions and intake screens, which will be placed on top op the caisson, was manufactured in Walvis Bay and installation is scheduled for November/December 2009.

29 September 2008: Caisson construction

4 November 2008: Final concrete cast

15 November 2008: Floating (Walvis Bay)

21 November 2008 (Berthed at NamDock)

23 November 2008 (Tow towards Wlotzkasbaken)

23 November 2008 (Being sunk at intake location)

APPENDIX F: TREKKOPJE TEMPORARY WORKS

Temporary works:

- Cofferdam: Since the pipelines are buried through the surf zone to a level below the lowest expected scour line, a cofferdam was constructed to protect the beach-crossing, avoiding sandy materials to re-silt the trench after excavation. The cofferdam also serves as a ramp for the excavation plant to access the jetty.
- Jetty: A 300 m steel jetty was constructed to serve as a working platform for excavation of the pipe trench through the surf zone. The jetty piles were driven into the seabed using a 3.2 ton drop hammer until it reached hard bedrock. A hole was then drilled into the bedrock to anchor the pile. The deck of the jetty consists of 47 sections, each 6 m long and wide, which was placed on the piles with a crane and then welded to each other.
- Pipe stringing yard: The pipe stringing/storage yard is the area where the individual pipe lengths were welded together, concrete coated and stored.
- Pipe launch-way and pulling arrangement: The launch-way's function is to maintain the design profile of the pipeline during the launching (pipe-pull) operation. The launch-way consists of concrete roller bases with steel rollers to support the pipeline.

25 October 2008: Jetty

 14 November 2008: Constructing cofferdam	 14 November 2008: Surf zone
 2 February 2009: Launch-way rollers	 20 January 2009: Pipe launch-way

APPENDIX G: BRINE RESERVOIR DESGIN

$h_{f\ (pipe)}$ (Darcy-Weisbach)	$h_{f\ (diffuser)}$ (Darcy-Weisbach & local losses)	$h_{f\ (pipe)}$ + $h_{l(diffuser)}$ (m)	q (m^3/s) Input	z_1 (m to MSL)	v (m/s)	$v^2/2g$	z_2 (m to MSL) Actual level of tank
0.39	**1.45**	**1.84**	**1.25**	**1.004**	**1.13**	**0.07**	**2.91**
0.42	1.455	1.88	1.30	1.004	1.18	0.07	2.95
0.45	1.46	1.91	1.35	1.004	1.22	0.08	2.99
0.48	1.465	1.95	1.40	1.004	1.27	0.08	3.04
0.52	1.47	1.99	1.45	1.004	1.31	0.09	3.08
0.55	1.475	2.03	1.50	1.004	1.36	0.09	3.12
0.59	1.48	2.07	1.55	1.004	1.40	0.10	3.17
0.62	1.485	2.11	1.60	1.004	1.45	0.11	3.22
0.66	1.49	2.15	1.65	1.004	1.49	0.11	3.26
0.69	1.495	2.19	1.70	1.004	1.54	0.12	3.31
0.73	1.5	2.23	1.75	1.004	1.59	0.13	3.36
0.77	1.505	2.28	1.80	1.004	1.63	0.14	3.42
0.81	1.51	2.32	1.85	1.004	1.68	0.14	3.47
0.85	1.515	2.37	1.90	1.004	1.72	0.15	3.52
0.89	1.52	2.41	1.95	1.004	1.77	0.16	3.58
0.94	1.525	2.46	2.00	1.004	1.81	0.17	3.63
0.98	1.53	2.51	2.05	1.004	1.86	0.18	3.69
1.03	1.535	2.56	2.10	1.004	1.90	0.18	3.75
1.07	1.54	2.61	2.15	1.004	1.95	0.19	3.81
1.12	1.545	2.66	2.20	1.004	1.99	0.20	3.87
1.16	1.55	2.71	2.25	1.004	2.04	0.21	3.93
1.21	1.555	2.77	2.30	1.004	2.08	0.22	3.99
1.26	**1.56**	**2.82**	**2.35**	**1.004**	**2.13**	**0.23**	**4.06**
1.31	1.565	2.88	2.40	1.004	2.17	0.24	4.12
1.36	1.57	2.93	2.45	1.004	2.22	0.25	4.19
1.41	1.575	2.99	2.50	1.004	2.26	0.26	4.25
1.47	1.58	3.05	2.55	1.004	2.31	0.27	4.32
1.52	1.585	3.11	2.60	1.004	2.35	0.28	4.39
1.58	1.59	3.17	2.65	1.004	2.40	0.29	4.46
1.63	1.595	3.23	2.70	1.004	2.44	0.30	4.53
1.69	1.6	3.29	2.75	1.004	2.49	0.32	4.61
1.74	1.605	3.35	2.80	1.004	2.53	0.33	4.68
1.80	1.61	3.41	2.85	1.004	2.58	0.34	4.75
1.86	1.615	3.48	2.90	1.004	2.62	0.35	4.83
1.92	1.62	3.54	2.95	1.004	2.67	0.36	4.91
1.98	1.625	3.61	3.00	1.004	2.71	0.38	4.99
2.04	1.63	3.67	3.05	1.004	2.76	0.39	5.06

Level of reservoir (m to MSL)	Volume of reservoir (m^3)	Volume (m^3)	Duration of discharge for each time step		Q = 0.278 m^3/s				Q = 0.627m^3/s				Duration (minutes) to fill reservoir to required level	
z2	Cylindrical reservoir (Dia = 42 m)	For each time step	min	cum min	Add volume from inflow	Total volume	Volume for each time step	cum min (inflow)	Add volume from inflow	Total volume	Volume for each time step	cum min (inflow)	Q = 0.278 m^3/s	Q = 0.627m^3/s
2.91	17.95	17.95	0.24	0.24	3.98	21.93	21.93	0.29	8.97	26.92	26.92	0.36	1.08	0.48
2.95	73.14	55.20	0.71	0.94	11.77	84.91	62.99	1.10	26.54	99.68	72.76	1.29	4.39	1.94
2.99	129.98	56.84	0.70	1.64	11.67	141.65	56.74	1.79	26.32	156.31	56.62	1.98	7.79	3.46
3.04	188.46	58.48	0.69	2.34	11.58	200.04	58.38	2.49	26.11	214.57	58.27	2.68	11.30	5.01
3.08	248.56	60.10	0.69	3.03	11.49	260.06	60.02	3.18	25.92	274.48	59.91	3.36	14.90	6.61
3.12	310.29	61.73	0.68	3.71	11.41	321.70	61.64	3.86	25.73	336.02	61.54	4.05	18.60	8.25
3.17	373.63	63.34	0.68	4.39	11.33	384.96	63.26	4.54	25.56	399.19	63.16	4.72	22.40	9.93
3.22	438.58	64.95	0.67	5.06	11.26	449.84	64.87	5.21	25.39	463.97	64.78	5.40	26.29	11.66
3.26	505.13	66.55	0.67	5.74	11.19	516.31	66.48	5.88	25.23	530.36	66.39	6.06	30.28	13.43
3.31	573.27	68.15	0.67	6.40	11.12	584.39	68.08	6.55	25.07	598.35	67.99	6.73	34.37	15.24
3.36	643.01	69.74	0.66	7.06	11.05	654.06	69.67	7.21	24.93	667.94	69.59	7.39	38.55	17.09
3.42	714.33	71.32	0.66	7.72	10.99	725.32	71.26	7.87	24.79	739.12	71.18	8.05	42.83	18.99
3.47	787.23	72.90	0.66	8.38	10.93	798.16	72.84	8.52	24.65	811.88	72.76	8.70	47.20	20.93
3.52	861.70	74.47	0.65	9.03	10.87	872.58	74.42	9.17	24.52	886.23	74.34	9.35	51.66	22.91
3.58	937.74	76.04	0.65	9.68	10.82	948.56	75.99	9.82	24.40	962.14	75.92	10.00	56.22	24.93
3.63	1015.35	77.60	0.65	10.32	10.77	1026.12	77.55	10.47	24.28	1039.63	77.49	10.65	60.87	26.99
3.69	1094.51	79.16	0.64	10.97	10.71	1105.23	79.11	11.11	24.17	1118.68	79.05	11.29	65.62	29.09
3.75	1175.23	80.72	0.64	11.61	10.67	1185.90	80.67	11.75	24.05	1199.29	80.61	11.93	70.46	31.24
3.81	1257.50	82.27	0.64	12.24	10.62	1268.12	82.22	12.38	23.95	1281.45	82.16	12.56	75.39	33.43
3.87	1341.31	83.81	0.63	12.88	10.57	1351.88	83.77	13.02	23.84	1365.15	83.71	13.19	80.41	35.65
3.93	1426.66	85.35	0.63	13.51	10.53	1437.19	85.31	13.65	23.74	1450.41	85.25	13.82	85.53	37.92
3.99	1513.55	86.89	0.63	14.14	10.48	1524.04	86.85	14.28	23.65	1537.20	86.79	14.45	90.74	40.23
4.06	**1601.97**	**88.42**	**0.63**	**14.76**	**10.44**	**1612.42**	**88.38**	**14.90**	**23.55**	**1625.53**	**88.33**	**15.08**	**96.04**	**42.58**

4.12	1691.92	89.95	0.62	15.39	10.40	1702.33	89.91	15.53	23.46	1715.38	89.86	15.70	101.43	44.97
4.19	1783.40	91.47	0.62	16.01	10.36	1793.76	91.44	16.15	23.37	1806.77	91.39	16.32	106.92	47.41
4.25	1876.39	93.00	0.62	16.63	10.32	1886.72	92.96	16.77	23.29	1899.68	92.91	16.94	112.49	49.88
4.32	1970.91	94.51	0.62	17.24	10.29	1981.19	94.48	17.38	23.20	1994.11	94.43	17.56	118.16	52.39
4.39	2066.93	96.03	0.61	17.86	10.25	2077.18	95.99	18.00	23.12	2090.05	95.94	18.17	123.92	54.94
4.46	2164.47	97.54	0.61	18.47	10.22	2174.68	97.50	18.61	23.04	2187.51	97.46	18.78	129.76	57.54
4.53	2263.51	99.04	0.61	19.08	10.18	2273.69	99.01	19.22	22.97	2286.48	98.96	19.39	135.70	60.17
4.61	2364.05	100.54	0.61	19.69	10.15	2374.20	100.51	19.83	22.89	2386.95	100.47	20.00	141.73	62.84
4.68	2466.10	102.04	0.61	20.30	10.12	2476.22	102.01	20.43	22.82	2488.92	101.97	20.61	147.85	65.55
4.75	2569.64	103.54	0.60	20.90	10.09	2579.72	103.51	21.04	22.75	2592.39	103.47	21.21	154.06	68.31

APPENDIX H: OPTIMIZATION OF DIFFUSER CONFIGURATION

Roberts & Toms: $S_t = 0.38F_r$ (Refer to Section 4.5.4.4)

$F_r = v_p/[g.d_p(\Delta\rho/\rho_s)]1/2$ (Refer to Appendix A)

ρs	1026	kg/m^3
ρe	1046	kg/m^3
St	20	Required dilutions

Nr of ports	Q = 1 m^3/s		Q = 1.5 m^3/s		Q = 2 m^3/s		Q = 2.5 m^3/s		Q = 3 m^3/s	
	Port flow (m^3/s)	Port diameter (m)	Port flow (m^3/s)	Port diameter (m))	Port flow (m^3/s)	Port diameter (m))	Port flow (m^3/s)	Port diameter (m))	Port flow (m^3/s)	Port diameter (m)
1	1.000	0.314	1.500	0.369	2.000	0.415	2.500	0.453	3.000	0.488
2	0.500	0.238	0.750	0.280	1.000	0.314	1.250	0.343	1.500	0.369
3	0.333	0.202	0.500	0.238	0.667	0.267	0.833	0.292	1.000	0.314
4	0.250	0.180	0.375	0.212	0.500	0.238	0.625	0.260	0.750	0.280
5	0.200	0.165	0.300	0.194	0.400	0.218	0.500	0.238	0.600	0.256
6	0.167	0.153	0.250	0.180	0.333	0.202	0.417	0.221	0.500	0.238
7	0.143	0.144	0.214	0.170	0.286	0.190	0.357	0.208	0.429	0.224
8	0.125	0.137	0.188	0.161	0.250	0.180	0.313	0.197	0.375	0.212
9	0.111	0.130	0.167	0.153	0.222	0.172	0.278	0.188	0.333	0.202
10	0.100	0.125	0.150	0.147	0.200	0.165	0.250	0.180	0.300	0.194
11	0.091	0.120	0.136	0.142	0.182	0.159	0.227	0.174	0.273	0.187
12	0.083	0.116	0.125	0.137	0.167	0.153	0.208	0.168	0.250	0.180
13	0.077	0.113	0.115	0.132	0.154	0.149	0.192	0.162	0.231	0.175
14	0.071	0.109	0.107	0.129	0.143	0.144	0.179	0.158	0.214	0.170
15	0.067	0.106	0.100	0.125	0.133	0.140	0.167	0.153	0.200	0.165
16	0.063	0.104	0.094	0.122	0.125	0.137	0.156	0.150	0.188	0.161
17	0.059	0.101	0.088	0.119	0.118	0.133	0.147	0.146	0.176	0.157
18	0.056	0.099	0.083	0.116	0.111	0.130	0.139	0.143	0.167	0.153
19	0.053	0.097	0.079	0.114	0.105	0.128	0.132	0.140	0.158	0.150
20	0.050	0.095	0.075	0.111	0.100	0.125	0.125	0.137	0.150	0.147
21	0.048	0.093	0.071	0.109	0.095	0.123	0.119	0.134	0.143	0.144
22	0.045	0.091	0.068	0.107	0.091	0.120	0.114	0.132	0.136	0.142
23	0.043	0.090	0.065	0.105	0.087	0.118	0.109	0.129	0.130	0.139
24	0.042	0.088	0.063	0.104	0.083	0.116	0.104	0.127	0.125	0.137
25	0.040	0.087	0.060	0.102	0.080	0.114	0.100	0.125	0.120	0.135
26	0.038	0.085	0.058	0.100	0.077	0.113	0.096	0.123	0.115	0.132
27	0.037	0.084	0.056	0.099	0.074	0.111	0.093	0.121	0.111	0.130
28	0.036	0.083	0.054	0.097	0.071	0.109	0.089	0.120	0.107	0.129
29	0.034	0.082	0.052	0.096	0.069	0.108	0.086	0.118	0.103	0.127
30	0.033	0.081	0.050	0.095	0.067	0.106	0.083	0.116	0.100	0.125
31	0.032	0.080	0.048	0.094	0.065	0.105	0.081	0.115	0.097	0.123
32	0.031	0.079	0.047	0.092	0.063	0.104	0.078	0.113	0.094	0.122
33	0.030	0.078	0.045	0.091	0.061	0.102	0.076	0.112	0.091	0.120
34	0.029	0.077	0.044	0.090	0.059	0.101	0.074	0.111	0.088	0.119
35	0.029	0.076	0.043	0.089	0.057	0.100	0.071	0.109	0.086	0.118
36	0.028	0.075	0.042	0.088	0.056	0.099	0.069	0.108	0.083	0.116
37	0.027	0.074	0.041	0.087	0.054	0.098	0.068	0.107	0.081	0.115
38	0.026	0.073	0.039	0.086	0.053	0.097	0.066	0.106	0.079	0.114
39	0.026	0.073	0.038	0.085	0.051	0.096	0.064	0.105	0.077	0.113
40	0.025	0.072	0.038	0.084	0.050	0.095	0.063	0.104	0.075	0.111