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Darwin City Waterfront
Community Infrastructure – Stage 1
Macmahon Contractors Pty Ltd

Design Package 03C.0
Flushing System
Design Philosophy

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1. Executive Summary

Efficient engineering structures have been designed to ensure adequate secondary contact water quality levels are maintained in the lagoon water body adjacent to public domain, DCEC and residential precincts.

The water level of this lagoon is too high to rely on tidal flushing to maintain the water quality therefore a pressurised pipe system is utilised to deliver ocean water into the lagoon.

Residency time, temperature and salinity variations within the lagoon have been numerically modelled using Delft-3D for both wet and dry seasons. This model investigated the lagoon morphological water quality variations based on the optimal 20 % turnover and satisfactory modelling results were achieved which showed:

- Median residence time of the lagoon is low (10 to 80 hours)
- Lagoon bottom layer is well mixed with stratification not able to be induced into the system
- The turnover rate provided efficient flushing of the lagoon
- Sufficient circulation was achieved without the need to account for the proposed lock which would greatly assist the system

Further analysis of the hydraulic system proved that three delivery pipes optimised the operation costs over a 20 year period.

The final designed flushing system is inherently flexible to accommodate future changes that may be required to suit environmental conditions, adjustments to the system include:

- Pump run times can be reduced, saving on running costs to suit turn-over rates
- Supply pipe diameters have been sized to minimise operational costs but can take an additional flow of up to about 25 %
- Mixing fountains can be retro-fitted (if stratification does develop in the future due to change in conditions to that modelled)
- The lagoon water level can be adjusted with the installation of stop logs on the outlet weir structure

The risk of stingers (box jellyfish) and other fauna entering the lagoon may have to limit human contact during their active season. Beach signage should be erected to warn the public.

2. Scope

This report details the limits and assumptions used for the design of the lagoon flushing system for the maintenance of the water quality in the lagoon.

The flushing system is required for two key reasons:

– Maintain the water quality of the lagoon to secondary contact standards.
– Provide make-up water to negate losses due to seepage and evaporation and thereby keep the lagoon at the desired level of +3.0 m AHD.

To flush the lagoon and ensure water quality meets the required levels it is necessary to turn over up to 23,000 m^3/day (23 Ml). Due to the elevated level of the lagoon in relation to available tides, (refer Section 4.2), this volume of water has to be pumped in. The most economical way of doing this is to

pump continuously (or as long as is practically possible subject to tidal action). Thus there will effectively be a continuous flow of ocean water through the lagoon, draining out through the outfall structure in the breakwater.

Due to the fact that the lagoon water level is raised well above mean sea level, it is neither practical nor economical, to flush the enclosed water body tidally and therefore a fully pumped system has been designed.

This document is to be read in conjunction with the following referenced design drawings:

C0-000	Cover sheet and drawing index
C0-001	General arrangement
C0-002	Flushing structure outlet detail.
C0-003	Flushing pipe through breakwater detail
C0-004	Typical details sheet 1 of 2
C0-005	Typical details sheet 2 of 2
C0-006	Pump station plans
C0-007	Pump station sections
C0-008	Flushing detail layout
S0-001	Structural notes
S0-002	Pump station plan and sections
S0-003	Outlet headwall details

3. Background

3.1 Overall Project Description

The Darwin City Waterfront (DCW) site is approximately 27 Hectares of land in close proximity to the Darwin CBD that has become available for development following the withdrawal of industrial installations over a number of years. The land is situated beneath a natural escarpment which creates a geographical separation to the Darwin CBD and State Square above.

The development will include three main elements and will be staged over 15 years. It will include:

- The Darwin Convention and Exhibition Centre, with a 1500-seat capacity and 4000 square metres of exhibition space,
- Community facilities including seawall, wave pool, swimming areas, public promenade, parklands and picnic areas,
- Commercial developments, including a Medina hotel, harbour side cafes and restaurants and a staged residential development.

The $300 million Stage One of the development will be completed by 2008 and includes the Darwin Convention and Exhibition Centre, most of the community infrastructure and commercial developments, including the 141-room Medina hotel, 138 residential apartments, restaurants and cafes.

4. Introduction

4.1 Purpose of this Report

This report has been prepared to provide an understanding of the philosophies used in designing the flushing system for the lagoon.

This report outlines the developed design requirements for the maintenance of water quality within the lagoon. Note that the water quality in the wave and swimming pools is not covered in this report.

4.2 Aesthetic Philosophy

The tidal range in Darwin can be up to eight metres. This can create an environment where there are large distances between land-based structures and low water level giving a disconnection between the people and the water. Our vision is to create a standing body of water that would be at the highest possible level to create the close and enjoyable relationships between water, buildings and people. This will also create the body of water that will deliver the designers reflectivity requirements for the DCEC, further enhancing its iconic nature.

The overall objective of the breakwater is to maintain this closed body of water to form the lagoon and marina. This level of water inside the breakwater will be fixed at +3.0 m AHD, to be as close possible to the public.

4.3 Background

The lagoon and marina are formed by the construction of a breakwater across the bay. The objective of the breakwater is to maintain a constant body of water in front of the DCEC and residential development as well as providing a marina. The maximum level of this body of water has been set at +3.0 m AHD, with an allowable range of 0.5 m.

Due to the necessity to dredge up to 10 m of overburden (marine muds) prior to the construction of the breakwater, the lagoon will be relatively deep in sections. This has raised some concerns on water quality and the possibility of stratification. This report considers what would be required to ensure adequate water quality is maintained.

The water body is split into two sections, the marina and the boating lagoon (referred to as the lagoon in this report), by a submerged berm and the floating pontoon. There is no hydraulic separation between the two bodies of water and hence there can be no guarantee that spillages from the marina (fuel, sewerage) cannot enter the lagoon section. The design of the system takes this into account and attempts to mitigate the risks.

4.4 Level of Water Quality

Water quality guidelines are given in the publication “Australian and New Zealand Guidelines for Fresh and Marine Water Quality (2000)”. Chapter 5 – Guidelines for Recreational Water Quality and Aesthetics defines two categories of recreational sporting water activity as follows:

- Primary Contact: Sports in which the user comes into frequent direct contact with water, either as part of the activity or accidentally; for example swimming and surfing.
- Secondary Contact: Sports that generally have less frequent body contact with the water; for example boating or fishing.

The requirements to achieve primary contact water are very stringent. Under the guidelines, the water intake from the sea may not meet these criteria all the time without additional treatment. In addition, the open connection with the marina makes it very difficult to guarantee a high water quality with any confidence. Under these conditions, to treat the lagoon water to continually meet primary contact criteria would be impractical and therefore was not considered viable. This approach was considered prior to financial close and was confirmed in the Water Quality Aspects Report by CW-DC issued with the MI & CI Development Application.

The flushing system is therefore designed to flush the lagoon to achieve a minimum of secondary contact water quality.

5. Lagoon Water Quality

5.1 Design Philosophy

With the construction of the breakwater, the lagoon and marina will form a closed body of water with a relatively constant water level. This body of water varies in depth from zero at the edges to 14 m deep near the toe of the breakwater.

To ensure that the water quality within an enclosed water body is maintained at an adequate standard, it is necessary to flush it with ocean water. For marine lagoons such as this, flushing can typically be achieved by three methods:

- Tidal flushing – water in on high tide and out on low tides. Best suited to lagoons at or close to mean sea level.
- Pumped flushing – water is pumped in and flows out under gravity. Typically used for elevated lagoons.
- A combination of the two – used for intermediate level lagoons.

5.1.1 Tidal Flushing

The ideal method of turning over the water in a lagoon is via tidal flushing as this makes use of the natural tidal cycle to fill and drain the lagoon. Typically tidal flushing of such a lagoon can be achieved using an inflow and outflow piped systems constructed through the breakwater. The inlet would need to be controlled with valves to prevent over-filling during tide movements above the lagoon water level of 3.0 m AHD. These valves would normally be operated automatically with the aid of water level followers feeding information to a computer. Similarly the downstream ends of the outlet pipes would need to be fitted with Tideflex non-return valves to prevent water flowing in during high tides and over-filling the lagoon. This type of flushing is best suited to lagoons which have a mean water level at or around the mean sea level.

At this site, the water level is 3.0 m above MSL and therefore despite the large tidal range, the efficiency of tidal flushing to this system had to be investigated. Thus, to assess the practicality of this option, a model of the system was set up and run over the full range of predicted tides in a typical year. As expected this showed that tidal flushing could not always be achieved. In fact for a considerable period of time only partial exchanges can be achieved and on occasion, for periods of up to seven days, tides are not high enough to achieve any flushing at all. Figure 1 below shows a typical month. It can clearly be seen that the volume of water (the shaded area) above the minimum allowable water level in the lagoon (horizontal line at 2.5 m AHD) is quite minimal.

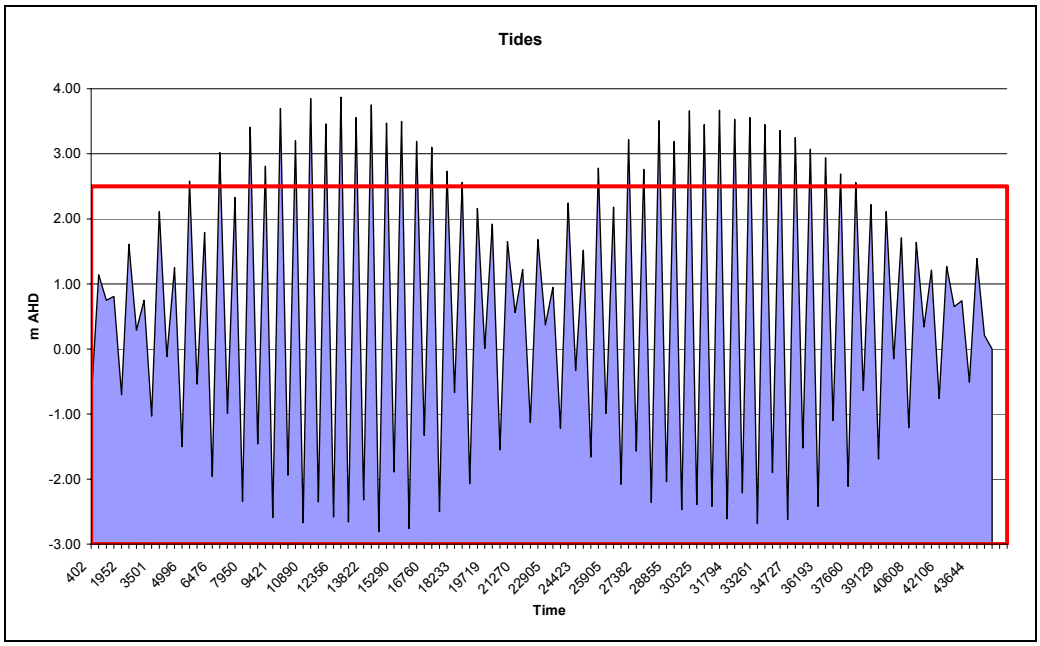


Figure 1: Typical Darwin Tidal Cycles

It is therefore concluded that the water level of this lagoon is too high to rely on tidal flushing to maintain the water quality at the desired level. In percentage terms, the tide is above the low water level approximately 11.6% of the time and above the high water level (when flushing has to cease to avoid flooding the lagoon) 6.4% of the time (refer Table 1 below, containing the data for the tidal cycles). Thus the available window for flushing is a mere 5.2 % of time, or about 38 hours a month. Even if large structures were constructed to allow tidal flushing with these tides, no flushing can take place for periods of up to about seven days at a time. This is obviously inadequate to guarantee the required water quality of the lagoon. Thus pumping needs to be considered to supplement or replace the tidal flushing.

Table 1: Available Flushing Times

Available Flushing Times			
	Above 2.50	Above 3.00	
Total month	5193	2837	mins
Monthly	**11.6**	**6.4**	%
Max period	6555	9510	mins
of no flush	4.6	6.6	days

Lagoon Operating Levels		
Upper level	3.00	m AHD
Lower level	2.50	m AHD

5.1.2 Combined Tidal and Pumped Flushing

Two options can be considered for the pumping system, firstly one that compliments the tidal flushing and secondly one that replaces the tidal system altogether. The first option, using tidal and pumping, requires that suitable pumping infrastructure needs to be available to pump the full volume of flushing water to cater for those periods when tidal levels are too low that tidal flushing cannot occur. Thus the design of the pumping infrastructure for the two systems would be similar, only the operation would vary.

Therefore combined tide and pump system would consist of the same pumping system as a pumped only system with the addition of the tidal flushing infrastructure. Without extensive additional infrastructure, the tidal flushing intakes would only be able to inject clean water close to their positions on the wall. This leads to very inefficient flushing of the lagoon and it may be necessary therefore to still pump to the remote parts of the lagoon to ensure a complete turnover of the entire volume of water. Given that additional infrastructure is required to have a combined tidal and pumping system, and the pumping system has to be capable of handling the total turnover in its own right (for the periods when no tidal flushing can be achieved), there is little advantage in maintaining both systems. It is uneconomical to justify the principal cost of this structure which could be only partly effective.

Therefore the flushing system has been designed to be a solely pumped system without any reliance on tidal effects. As a safety precaution, the pumps will have feed back from sensors in the lagoon preventing them from pumping if the lagoon water level exceeds +3.0 m AHD (or other levels set by the operators).

5.2 Circulation

Due to the shape of the lagoon and marina, several circulation cells will have to be working together to ensure complete mixing. To achieve this it is proposed that three points of discharge within the lagoon be used, one near the southern end of the DCEC, one near the wave pool and the third by the beach section (refer Figure 2 below.)

One set of outlets to drain the lagoon back to the sea is envisaged, this being sited at the western end of the breakwater. The sea outlets have been designed to handle the peak flow of the pumps at an external tide level of about +2.5 m AHD or lower. The downstream ends of the outlet pipes will be fitted with “Tideflex” non-return valves to prevent water flowing back into the lagoon during high tides.

If pumping (inflow to the lagoon) ceases, the level of the lagoon will drop relatively rapidly (approximately 24 hours) to that of the level of the weir sill on the outlet structure (2.87 m AHD). Thereafter the water level will drop very slowly as losses continue through seepage and evaporation (estimated at 5 to 10 mm a day).

The outlets have also been designed to cope with major rainfall events and wave overtopping of the breakwater. In the design storm event, it will take up to approximately 40 hours to drain the lagoon from the peak level of 5.5 m AHD back to normal supply level of +3.0 m AHD. This time period is on top of any time that the subsequent tides are high enough to restrict or cease drainage from the lagoon.

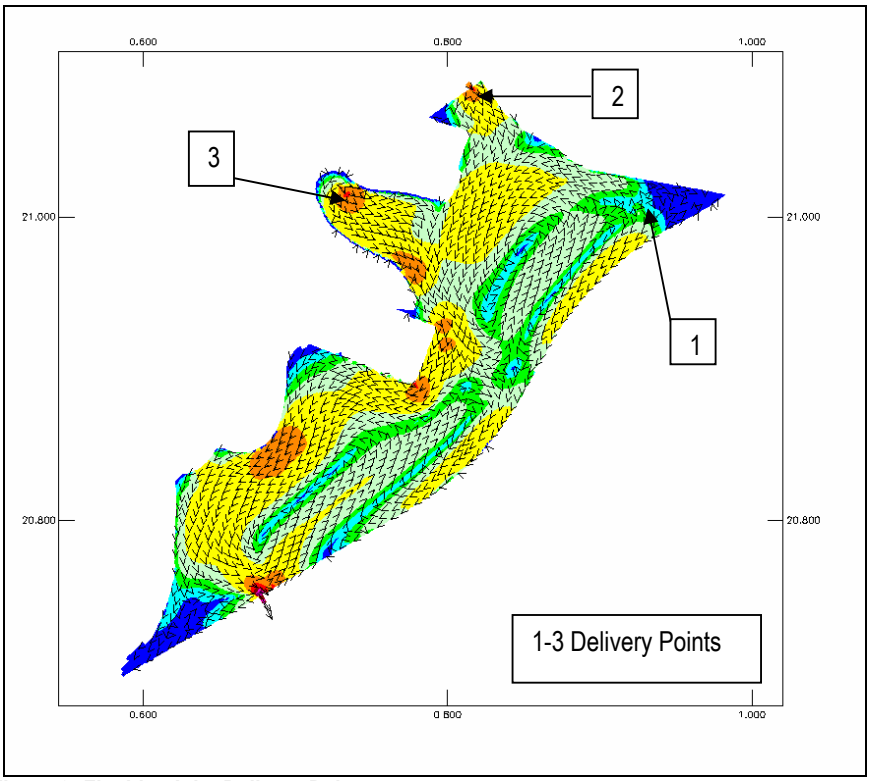


Figure 2: Flushing Inlet Delivery Points

5.3 Water Quality Issues

The operation of watercraft can involve a range of contaminating events (such as fuel leaks/spills, sewage leaks/spills, contaminated ballast water release, exotic flora/fauna deposition, litter, antifouling chemical release, etc.). Due to the possibility of spills of these pollutants within the marina section of the lagoon, it is necessary to maintain as much hydraulic separation between the marina and the lagoon as possible. To achieve this, it is necessary to maintain a positive flow from the lagoon to the marina and has therefore necessitated the need for the outlet structure to be positioned in the marina section of the breakwater wall.

All stormwater runoff from pavements (where practical) will be directed away from the lagoon, primarily to avoid influxes of contaminants into the lagoon. Water from roofs however, will be permitted to drain to the lagoon as it is unlikely to carry pathogens or significant pollutants.

5.4 Turnover Volumes

The required volume of water to be turned over (replaced) each day to maintain water quality within a lagoon is highly dependent on site conditions and usage. Recommended turnovers range from about 10 % to complete turnover per day. (The latter is obviously aimed at primary contact levels in swimming pools). The Coastal Engineering Manual, for instance, quotes the exchange coefficients of five small boat harbours as ranging between 14 and 22 %. Whilst no details are given that would allow a direct comparison with this lagoon, it is expected that the figures of 10 to 20 % turnover represent typical turnover rates to provide adequate water quality for secondary contact. Where primary contact is required, the more onerous quality criteria would be more dependent on local conditions, particularly the estimated public usage.

In this case we have a very large lagoon (in area) and a relatively large water body coupled with expected relatively low public usage (swimming is to be actively discouraged). It is estimated that the combined volume of the lagoon and marina is of the order of 300 000 m^3. However, as the water quality in the marina is less critical, the system strives to guarantee secondary water quality in the lagoon only. It is proposed that the marina be flushed with the water flowing out of the lagoon rather than additional flushing water from the sea. This reduces the volume of water required to be guaranteed to be kept at the high quality to a volume of about 126 000 m^3 (that of the lagoon).

Information from Lake Alexander a local Darwin lagoon, is probably the closest model for comparison purposes. We acknowledge that its depth is shallower than the deeper sections of the lagoon. As it is designed for swimming, Lake Alexander is required to achieve primary water quality rather than secondary as in this case. The lake has an area of 3.5 ha with a maximum depth of 2.8 m. The volume was not specified but is estimated to be 70 ML. CW-DC was advised that the residence time is 14 days and there is monthly checking of water quality. Pumps operate twice a day for a maximum of 6 hours as the tides allow, at a flow rate of 160 L/s. This represents a peak turnover of about 10 % of the volume per day. Rob Donovan, Operations Manager DCC, stated that the 14 day figure is pragmatic and reflects what the current pumping system can deliver, rather than what is desired (April 2004). Although apparently meeting the criteria for primary contact water, the turnover rate is not ideal. There is a foreign species of jellyfish in the lake that are apparently flourishing, which was the subject of study at the time, aimed at controlling numbers. There has also been the issue of biting fish in the lake.

In the absence of any further guidance in the aforementioned literature, the flushing system has been designed to pump a conservative 20 % turnover. To counter the conservatism, the system has the flexibility of being able to reduce the flow rate or pumping times if it is found that this volume of turnover can be reduced (refer Section 6.6). To finally confirm the estimates, mathematical modelling was undertaken and the methodology is discussed in Section 6.2. Whilst it is agreed that a nutrient balance model is ideal to model water quality with time, such models are very data hungry and require on-site calibration for the results to have any credibility. In this case there was neither the available data and as the lagoon has not been constructed, the means to undertake any calibration. Therefore, it was agreed at a meeting with the major stakeholders on the 09 May 2006 that CW-DC undertake basic hydrodynamic mathematical modelling. This modelling (refer Section 6.2) showed that **with the proposed 20 % turnover the median residence time of the lagoon is low, (10 to 50 hours) and the bottom layer is well mixed.** The lagoon residency time is similar to the turnover rate, resulting in efficient flushing of the lagoon.

5.5 Stratification Potential

Due to the necessity to dredge up to 10 m of overburden (marine muds) prior to the construction of the breakwater, the lagoon will be up to 14 m deep in places. This has raised some concerns on the possibility of stratification. To address this possibility it was initially proposed to include three fountain mixers in the lagoon and marina to increase vertical mixing of the water. However, others are of the opinion that the water body is large enough for natural mixing, aided by the flushing system, to be adequate to reduce the risk of stratification occurring, which would negate the need for the fountains.

Dr A Marshall wrote in her comments to the DCA in response to the project approval process, proposed flushing scheme for this project, “Aquatic Pest Management (APM) within Fisheries Group DBIRD have monitored water quality in all four Darwin marinas and selected sites in Darwin Harbour for the past five years and have seen a reduction in the degree to which stratification has developed in the marinas following Wet Season rains since 1999 as a result of discussions with marina management and lockmasters. The level of exchange in the order of 5 - 20 % over one to two weeks has assisted in preventing stratification from developing in the closed water bodies.” (18 February 2005).

Based on the possible risk, the design took the approach of "design for, but not with", whereby all infrastructure will be constructed to allow future installation of the mixers, but the mixers will only be retro-fitted if it is shown that they are required. This provided a direct saving to the project.

Following further discussions with the primary stakeholders, it was agreed on the 09 May 2006 that CW-DC undertake basic mathematical modelling to show that our design assumption, that stratification was unlikely to occur, was valid. This modelling (refer Section 6.2) was undertaken and showed that **no stratification was likely to occur in either the dry or wet seasons.**

6. Detailed Design

6.1 Pumping Volumes

The volume of water required to be pumped into the lagoon to provide the required flushing turnover per days consists of 20 % of the volume of the lagoon (refer Section 5.4). Estimates of seepage and evaporation are also taken into account for the design:

Flushing volume:	23 000 m^3/day (20 % of lagoon volume).
Seepage losses:	240 m^3/day (estimated based on area of lagoon).
Evaporation losses:	165 m^3/day (estimated based on Darwin typical evaporation figures).

(All figures rounded up)

Thus the total volume of water to be pumped in is 23 000 m^3/day and the total volume flowing out through the outlet structure is approximately 22 600 m^3/day. It is arguable as to whether the evaporation and seepage losses should be deducted, but at the end of the day their effect is minimal.

In addition to this inflow, CW-DC has been instructed to provide a source of seawater to the wave pool at a rate of 15 l/s. The wave pool designer confirmed that they would require 84 m^3 after a major rainfall (296 mm) and this needs to be pumped in a day. With a combined surface area of the lagoon and marina of approximately 46500 m^2, the required volume equates to about 2 mm in level. Therefore the pumping rates do not need to be altered to supply the wave pool make up sea water.

No allowance has been made in the calculations for any other additional water requirements such as basement flushing systems. To date CW-DC has not been briefed on the requirement to include any allowances for these.

6.2 Mathematical Modelling

CW-DC undertook a mathematical model study to confirm the flushing efficiency in the lagoon and to show whether stratification is likely to occur in this lagoon. A full model report is included as Appendix A.

The model consists of the development and exploitation of a Delft-3D model of the lagoon. The numerical model investigated the lagoon morphological water quality variations. The lagoon was modelled using a 3D curvilinear fit to the lagoon boundaries. Residency time, temperature and salinity variations were modelled. Two morphological scenarios were considered to represent both wet and dry seasons.

For each of the season simulations, tracers were put into the lagoon at the flushing points and the diffusion of the tracers examined as they were flushed through the lagoon. It was found that after the

system had established itself, the circulation throughout the lagoon was satisfactory for secondary contact levels.

The following conclusions were drawn from the numerical model study:

- The median residence time of the lagoon is low, (10 to 80 hours). The bottom layer is well mixed in the lagoon because the deep-water delivery point (No. 1) is advecting flow and induces additional turbulence;
- Approximately 3 days are necessary to replace 50% of the water intake, except in the Marina where the median residence time reaches approximately 8 days;
- Approximately 10 days are necessary to replace 90% of the water intake, except in the Marina where the significant residence time can reach approximately 25 days;
- The marina residence time is longer than the basin turnover rate and the residence time in the lagoon is smaller that the basin turnover rate. The water quality in the lagoon would be similar to an equivalent Plug Flow Reactor (PFC) system and in the marina to an equivalent Continually Stirred Tank Reactor (CSTR) system;
- The water quality at the sill is characteristic of a transition area and would be generally uniform over its depth;
- All results calculated without the effect of the lock which may greatly increase circulation;
- The deep-water delivery point (No. 1) has a significant influence on the deepest layer of the lagoon as it increases currents in the deep layers;
- **The model was unable to induce stratification in the system.**

6.3 Intake Pumps

The intake pumps were originally proposed to be housed in a purpose built pump station drawing water from the sea at the position of the old power station intake, (other positions for this form of intake were also considered). Due to the high tidal range, the pumping wells for this option would need to be in the order of 13 m deep. This also posed significant problems in construction of the intake pipeline from the submarine intakes to these wells.

To overcome these problems, and reduce construction costs, it was proposed to move the pump intakes to the Stokes Hill Wharf.

Two options were considered for the pump station;

- Surface mounted motors with hydraulic or shaft drives.
- Submersible pumps on rail systems.

Although the surface mounted motor option initially appeared attractive, the issue of noise on the public wharf would be problematic. To overcome this it would be necessary to construct a noise proof housing which then limited accessibility of the pumps and shafts. Along with this, the pump suppliers had concerns with the durability and maintenance requirements of the shafts which would have to be about 11 m long. Therefore this option was not considered further.

The submersible pump option also has the issue of noise, but from a fishing perspective. The wharf is a popular fishing site and it was imperative that the running of the pumps did not cause problems for fishermen. Therefore the siting of the pumps had to take into account the prime fishing points on the wharf to ensure that the pumps do not scare the fish away and also to avoid fouling of the pumps by fishing tackle. As the pumps are permanently underwater there will be no noise issues on the wharf.

Based on the above and advice of the pump suppliers, the preferred option is the submersible pump option. The pump station therefore will consist of three submersible pumps mounted on rails, braced to

new piles adjacent the wharf. A permanent working platform will be mounted above the structure to allow retrieval of the pumps for maintenance. Note that it was necessary to use independent piles rather than tie to the existing wharf piles because no guarantee could be given on the integrity of the existing wharf piles. The new structure will only connect to the wharf at the working platform, thus forming an independent structure.

6.4 Delivery Pipe System

Due to the shape of the lagoon, several circulation cells will have to be working together to ensure complete flushing of the water body. Thus it is not possible to have only one pumping delivery point into the lagoon to achieve complete flushing of the lagoon. It is proposed that three delivery points be used, one near the southern end of the DCEC, one by the swimming pools and the third by the beach section. All three delivery points are on the floor of the lagoon which helps in the mixing of the deeper water.

For each of the options for the outlet from the lagoon back to the sea, six different delivery pipe sizes were trialled. HDPE pipes are proposed throughout due to there robustness in marine applications (industry standard). Losses were calculated for fittings and pipe roughness. Due to the fact that the pipes are passing open marine water it can be expected that some marine growth may build up on the walls of the pipes if not maintained. For this reason the pipe system will need to be cleaned on a regular basis – estimated to be once every six months. For the detailed design a wall roughness factor of 1.0 mm was used in the calculations (compared to the recommended value of 0.007 mm for clean HDPE pipes, AS2200).

Consideration was given to chlorine dosing of the line to reduce marine growth, but aside from the problems associated with the chemical dosing itself, it is believed to be inefficient in these sorts of lines unless undertaken at very high doses. This would then lead to environmental problems and hence the option was not pursued further. Instead, provision has been made to clean the line with pigs. Pigging points have been incorporated at the start of the delivery pipes to enable this to be undertaken. It is envisaged that pigging would be undertaken using a portable high pressure pump drawing water directly from the lagoon.

Allowance for the delivery pipe to be closed at both ends has been incorporated should future users decide to undertake some form of chemical dosing.

6.5 Outlet Structure

The outlet structure from the marina back to the sea, consists of a HDPE outlet pipe through the breakwater from a weir inlet on the marina side to a headwall on the seaward side. The weir acts as a control for the water level in the lagoon. This level is set such that when the flushing system is operating at the desired flow rate the water level in the lagoon will be at its design level, namely +3.0 m AHD. Based on the flows derived above, the level of the weir of the outlet would be at +2.87 m AHD. The weir level can be adjusted, to account for settlement or changes in operation, by installing stop logs on the crest.

The weir inlet to this structure is covered with a safety grille to prevent people accidentally falling into the well. As such the grille bar spacing has been kept wide to allow minor debris to pass through and not interfere with the operation of the system. The grille has also been angled up at 30 degrees to reduce the chance of debris blocking the grille during operation. The structure of the grille has been designed to allow easy raking of debris up on to a maintenance platform. Calculations were undertaken to check what would happen if the grille openings became progressively blocked. Prior to flooding the system, the water level followers would switch off the pumps preventing inundation of the public domain.

The seaward end of the outlet pipe incorporates a "Tideflex" non-return valve to prevent water flowing back into the lagoon during very high tides. Tideflex valves have a quoted life of up to 50 years and require minimal maintenance due to their elastomer construction. Routine inspections will be required at six monthly intervals and after each major storm event.

Marine growth may build up on the walls of the pipe with time. This will need to be cleaned regularly to keep the system operating efficiently. It is envisaged that this cleaning will be undertaken from the downstream end using rodding techniques or pigs. Again this should be done at six monthly intervals.

6.5.1 Optimisation of the Outlet System

The desired volume of flushing water has to be pumped into the lagoon during the period when the tide levels permit gravitational drainage through the outlets. This period of time was calculated over an entire tidal cycle for six different options of the outlet structure pipe diameter to determine the average pumping time. For the outlet pipe options considered the average pumping time was found to range between 82.0 % and 93.1 %.

Table 3 below shows a typical extract from the outlet calculations sheet, based on the Darwin tides.

Table 3: Outlet Pipe Sizing

Parameters		Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
Volume	m^3	**115175**	**115175**	**115175**	**115175**	**115175**	**115175**
Turnover		**20%**	**20%**	**20%**	**20%**	**20%**	**20%**
INPUT							
Nominal pipe size	m	0.450	0.500	0.560	0.630	0.630	0.710
Pipe internal diameter:	m	0.3658	0.4065	0.4555	0.5123	0.5123	0.6031
Est seepage losses	m^3/hr	10	10	10	10	10	10
Est evaporation loss	m^3/hr	6.8	6.8	6.8	6.8	6.8	6.8
Total flowrate:	m^3/hr	1022	1027	1020	1016	1039	1022
Elevation upstream:	m RL	3.0	3.0	3.0	3.0	3.0	3.0
Elevation downstreamoutlet:	m RL	0.5	0.5	0.5	0.5	0.5	0.5
Length main:	m	18.0	18.0	18.0	18.0	18.0	18.0
Roughness 'k':	mm	1.5	1.5	1.5	1.5	1.5	1.5
No outlet pipes	no.	3	2	2	2	1	1
HYDRAULIC DETAILS							
Percent time tide > level	%	7.7	8.1	7.4	7.0	9.1	7.6
Flowrate per outlet:	m^3/s	0.095	0.143	0.142	0.141	0.289	0.284
Velocity:	m/s	0.90	1.10	0.87	0.68	1.40	0.99
friction factor 'f ':		0.0291	0.0281	0.0273	0.0265	0.0263	0.0252
Friction losses K		1.90	1.90	1.90	1.90	1.90	1.90
Head Required 'h'	**m**	**0.14**	**0.19**	**0.11**	**0.07**	**0.28**	**0.13**
Max Tide level for pumping	**AHD**	**2.86**	**2.81**	**2.89**	**2.93**	**2.72**	**2.87**

Six delivery pump and pipe configurations were considered for each of the above outlet pipe options and capital and estimated running costs put to the solutions. The resultant matrix of 36 options was then used to determine the most optimum solution for the project.

Allowing for the requirement that a minimum flow velocity through the outlet of one metre per second is required to limit slime/marine growth (refer standard guidelines for rising mains), the optimum delivery pipe size is either a single 630 or 560 mm diameter or three 450 mm diameter pipes, coupled with a single 710 mm diameter outlet pipe. Note that HDPE pipe sizes refer to outside diameters. However, the smaller single delivery pipe has higher headlosses, requiring more powerful pumps and higher running costs. Therefore the 630 mm delivery pipe was chosen as the most economical solution. This also allows a small margin to pump more water in if this is desired for some reason, eg compensation for water losses due to the future lock. Further optimisation with actual construction costs showed that three delivery pipes were in fact better than the single delivery pipe option for a variety of reasons (joint welding, piggability availability etc). Therefore the final pipe size option was three off, 450 mm diameter HDPE delivery pipes. As shown in the extract from the calculation tables above, this is equivalent to the single 630 mm pipe.

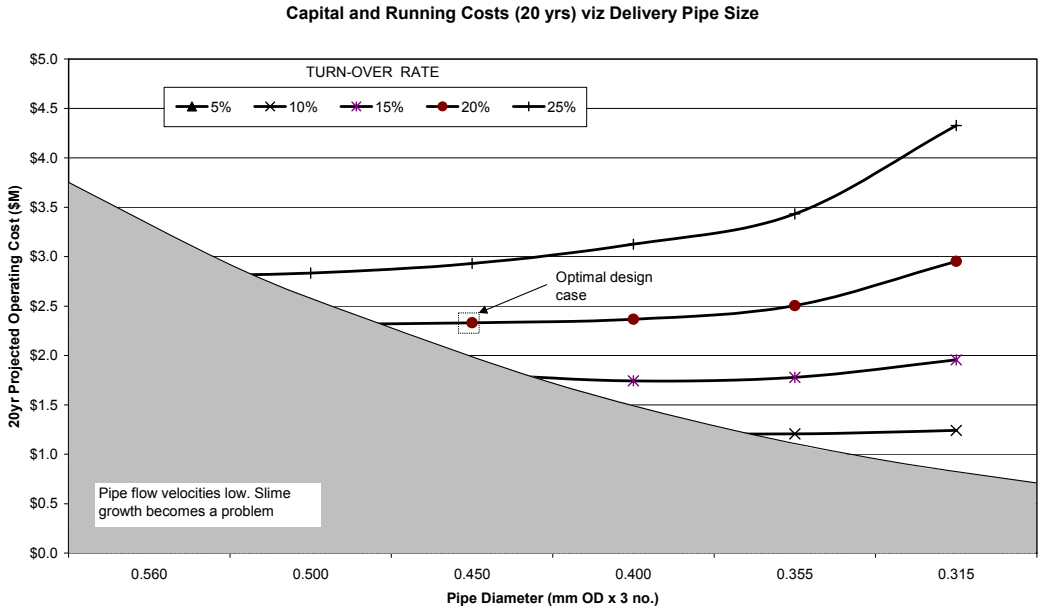


Figure 5: Capital and Running Costs viz. Delivery Pipe Size

Comparative running capital and costs over a twenty year cycle are shown the graph above for the chosen option of three 450 mm diameter delivery pipes and a single 710 mm diameter outlet pipe. The graph shows the relative costs for different volumes of flushing above and below the design value of 20 % of the volume. It must be noted that the costs quoted in this figure are for comparison purposes between individual options only and do not represent the final costs of each option.

6.6 Flexibility of the Design

The flushing system design has the following inherent flexibility to allow for any changes that may be required for what ever reasons during its life.

- The pumps have been designed to run continuously whilst tides allow. On average this equates to about 90% of the time and will then turnover 20% of the lagoon water per day. If it is found that this turnover is not required to maintain an adequate water quality, the time that the pumps run for can be reduced, saving on running costs.

- Should for some reason it becomes necessary to pump more water into the lagoon (such as for make up water to replace losses from the lock instead of a pump at the lock) the pumping rate can be increased. The supply pipe diameters have been sized to minimise operational costs but can take an additional flow of up to about 25 % if required.

- Mixing fountains can be retro-fitted if it is found that natural mixing of the lagoon is not adequate to prevent stratification.

- Water level can be raised with the installation of stop logs on the outlet weir. This may be required if the rate of pumping into the lagoon is reduced and it is desired to keep the lagoon level as high as possible.

7. Control of Pollutants

7.1 Siltation

An estimation of the expected siltation rate was made for the system. Based on an average input load of 15 mg/l, and assuming that all the sediment drops out in the lagoon, the expected rate of siltation would be approximately 150 m^3 per annum. Actual figures from the report "The Water Quality of Darwin Harbour" give median total suspended solids (TSS) of 6 mg/l. This is therefore a conservative estimate but gives an indication of the siltation rate.

It is expected that at this rate of siltation, the build up of silts will not have any significant effects on the lagoon.

7.2 Stormwater Runoff

Where possible, all stormwater runoff from roads, paved areas and landscaped areas, which by its nature may carry pollutants, will be captured in the underground stormwater drainage system. This system passes the runoff water through gross pollutant traps to remove the pollutants prior to disposal into the sea to the west and east of the development.

Stormwater captured directly off the roof of the DCEC building will be relatively clean and this can be directed into the lagoon. The volumes of water from this roof compared to that falling directly onto the lagoon are very small and insignificant in comparison with the volume of water in the lagoon.

Rainfall runoff from some small areas of landscaping close to the lagoon edge cannot be captured and will enter the lagoon. These areas are very small and none are likely to bring oils or greases with them. Therefore these should not create a problem in maintaining the water quality.

7.3 Swimming Pool Water

The wave pool produces three sources of wastewater that need to be disposed of.

1) Backwash from the filter systems. This will carry with it a large quantity of pollutants including possible pathogens and therefore must not be released to the lagoon and will be discharged to the sewer via a gravity connection with appropriate waste discharge licenses required to be in place by the operator.

2) Emergency overflow condition when a pipe is ruptured in the basement plant-room. This water will be very high quality, albeit containing some residual chlorine from its treatment process. Subject to further studies, it is expected that this water may be released to the lagoon as its quality will exceed that of the water being pumped into the lagoon from the sea. The residual chlorine will help to improve the water quality although an assessment of the expected quantity and life of this chlorine will need to be made. It would need to be shown that the chlorine will all have decayed prior to the water entering the ocean for this water to be dumped directly into the lagoon. The alternative is to provide a post treatment plant to remove the residual chlorine prior to dumping the water.

3) A basement pore water pressure relief system which runs around the perimeter of the wave pool structure and drains directly to the lagoon.

7.4 Groundwater

During the initial geotechnical investigations, hydrocarbons were detected in the ground which could be mobilised by groundwater movement. Should these hydrocarbons find there way into the lagoon in any significant volumes, they will create an oily sheen on the water surface and drop the water quality below acceptable standards.

To ensure that these contaminants cannot be mobilised into the lagoon, cut-off drains are being constructed at critical sections around the lagoon. CW-DC cannot guarantee the effectiveness of these drains in ensuring that no hydrocarbons enter the lagoon.

7.5 Fauna

The flushing system draws water directly from the sea and therefore does not reduce the risk of stingers (box jellyfish) entering the lagoon, in either adult or juvenile form. Similarly the future lock will also allow marine life to enter the lagoon. Therefore, use of the lagoon may have to be limited during their active season, approximately October to May. Beach signage will be necessary, consistent with signage erected at all public beaches around Darwin.

Similarly the lagoon can not be guaranteed to be free from crocodiles (although this is unlikely) and once the lock is operational, there is always the risk that stray sharks may enter the waters. Thus the recreational lagoon will carry a similar risk to using the open sea for similar activities with regard to these issues.

Appendix A

Mathematical Modeling

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Macmahon Contractors Pty Ltd

Design Package 03C.0
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Mathematical Modelling

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1. Model Set-up

1.1 Model Description

A Delft-3D numerical model of the Darwin Waterfront Lagoon has been developed to investigate the hydrodynamics of the lagoon and whether significant stratification is likely. The computation grid used was 10 layers deep, all layers being 10% of the water-depth. Flushing water inlets (pump outlets) have been placed on the bottom layer and the discharged water is controlled by a Q-H weir relationship in the Marina (the weir outlet). The calculation of turbulence was made with the k-ε model to maximise opportunities to develop stratification in the numerical model if this was to occur. Table 1 indicates the selected default numerical model parameters.

Table 1: Default Darwin Waterfront Delft3D Water Quality Model Parameters

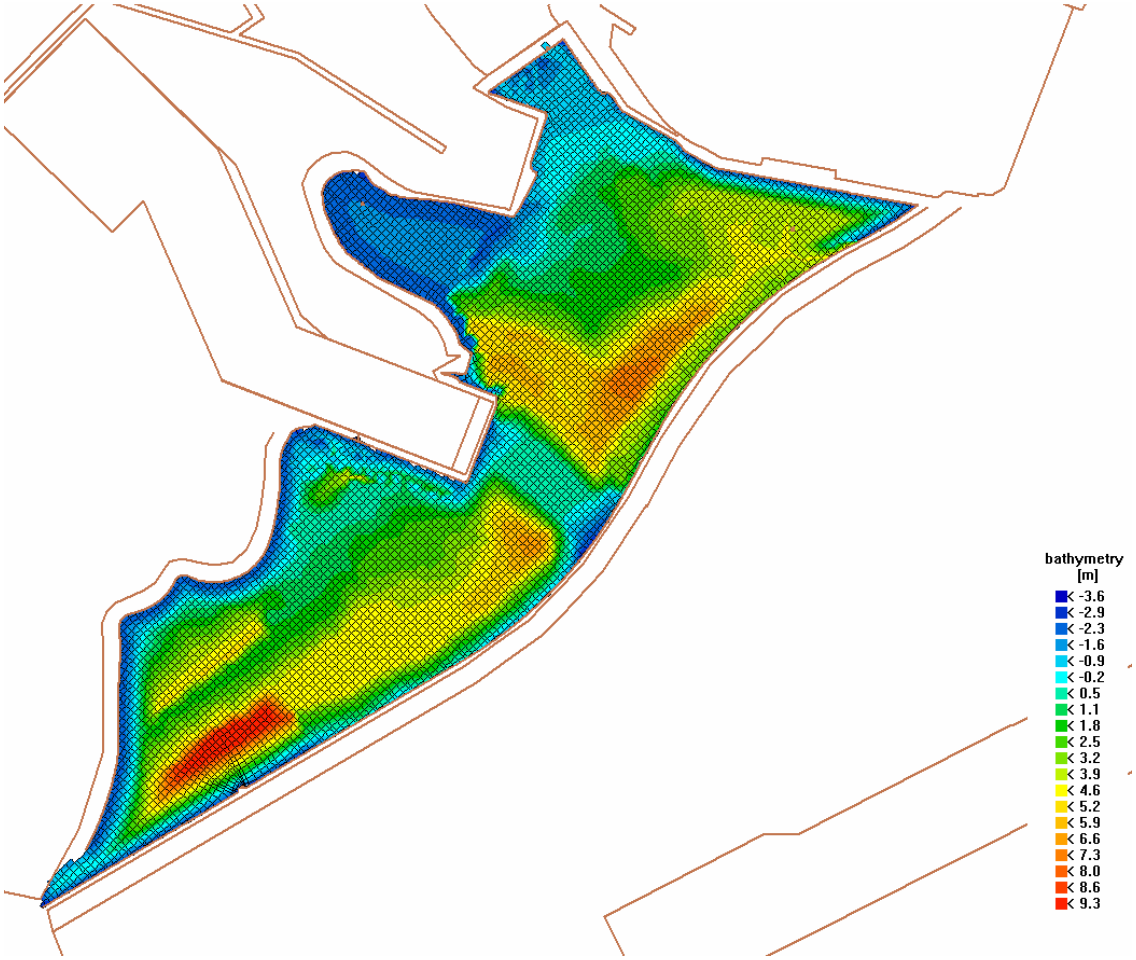
Item	Description
Computation Grid Size	101 x 42 x 10
Layering	Uniform
Typical Grid Width (m)	5 x 5 x 0.5
Time Step	3s
Boundary Condition: Intake	3 x pump outlets on the seabed 107 l/s/outlet (O1, O2, O3)
Boundary Condition: Outflow	QH relationship for Weir 2.7m wide at +3.0m AHD
Bed Roughness	N = 0.02
Ambient Eddy Viscosity	0.1 m^2/s
Vertical Diffusion	Corrected for Sigma-Coordinate
Transport Model	Salinity, Temperature, Tracer
Turbulence Model	K-ξ
Heat Flux Model	Excess Temperature
Advection Method	Cyclic

Figure 1 shows the extent of the computation domain. The Darin Waterfront basin physical extent is characterised by:

- The northern basin, called the “lagoon”, with its beaches and water contact sports areas in the shallower areas and a deep-water area to the east, along the breakwater.
- The southern basin, called the “marina”, that is progressively becoming deeper from the onshore development to the breakwater foundation trench.
- An underwater earth-bund which separates the two bodies of water, called the “sill”, that is a relatively shallow area (approx. 3.5m deep, when the water level is a +3.0m AHD).

Non-dimensional analysis suggests that wind-shearing at the lagoon surface is a key mixing parameter. Therefore, this scenario was expected to be conservative, as the absence of wind impacts in the numerical solution would increase the mixing capacity of the lagoon, the k-e turbulence and excess temperature numerical sub-models.

Figure 1: Lagoon Computation Domain (View From Top)



1.2 Wind Analysis

Wind data from was taken from the station at Stokes Hill Wharf and analysed. Wind speeds were recorded hourly from May 1991 to October 2004. The average wind speed was found to be 2.75 m/s. Table 2 summarises the wind speed exceedance for this station, while Figure 2 and 3 shows daily averaged and monthly averaged wind variations.

Table 2: Wind Speed Exceedance at Stokes Hills Wharf (1991-2004)

Wind Speed	Exceedance
Calm	12.8 %
Calm to 1 m/s	12.0 %
1 to 2 m/s	14.4 %
2 to 3 m/s	17.1 %
3 to 4 m/s	16.3 %
4 to 5 m/s	12.2 %
5 to 6 m/s	6.9 %
6 m/s or more	8.2 %

Figure 2: Daily Wind Speed Average

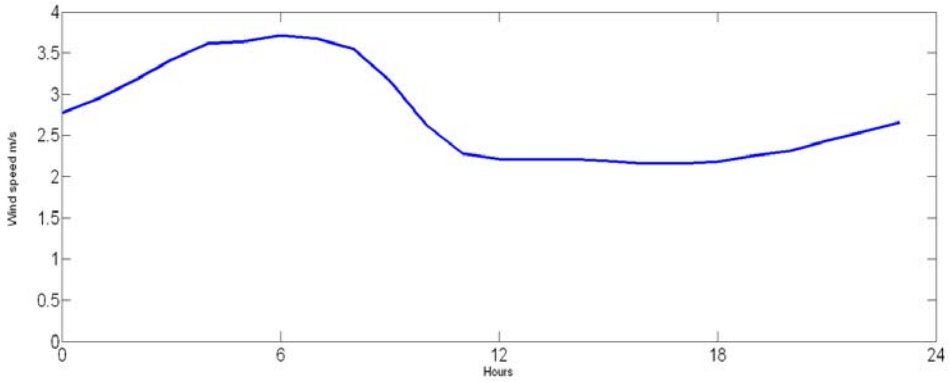


Figure 3: Monthly Wind Speed Average

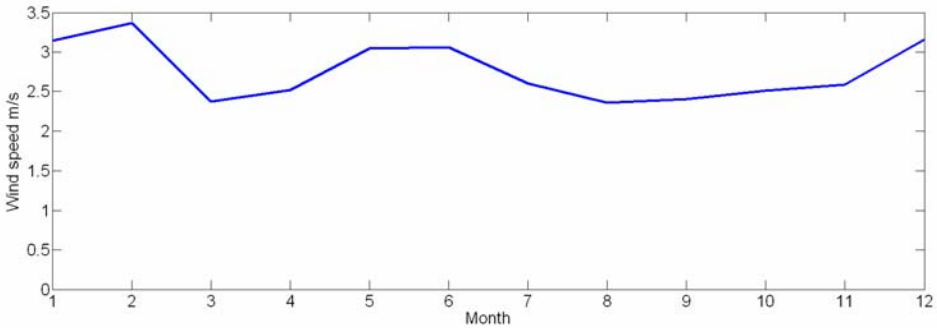


Table 3 below shows the analysis of the calm-wind duration exceedance. For instance, 52.7% of the calm wind condition lasts for approximately one hour. 95% of the time, periods of calm-wind last less than 10-hours. The longest period that the wind has been calm in the record was for 16 hours from 12 am on 15 March 1997, to the 4 am on 16 March 1997.

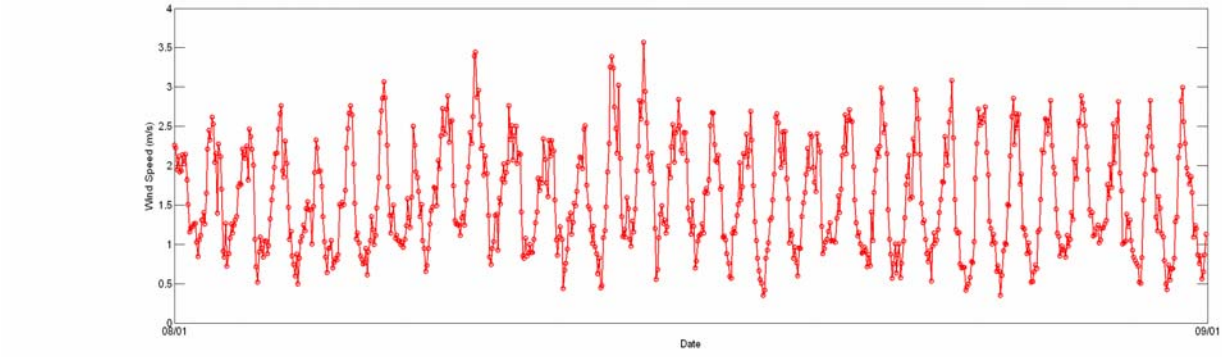
Table 3: Calm Wind Duration Exceedance (Stokes Hills Wharf)

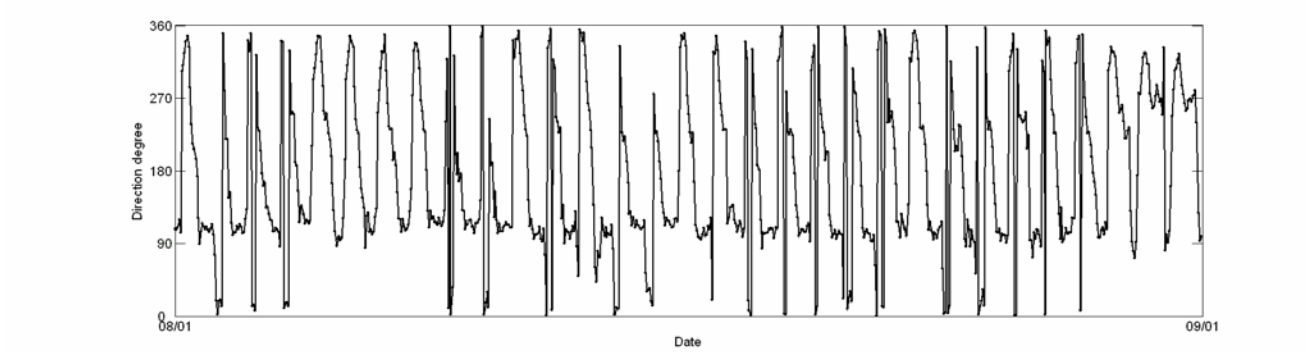
Duration (hours)	Exceedance (%)
1	52.7 %
2	22.4 %
3	11.1 %
4	5.8 %
5	3.3 %
6 to 10	4.3 %
10 or more	0.4 %

Considering that the wind is calm for only 13% of the time and that when the wind is calm it stays calm for more than 10 hours less than 5% of the time, it seems that the nil-wind speed for 45 days used in the initial modelling was an over-conservative assumption.

Therefore, it was proposed to force the model with a morphological wind. It was proposed to use the vectorial-averaged wind over the calmest month of the year (August). This wind was then corrected for terrain conditions due to obstructions (buildings, breakwater). In this respect the wind speed was decreased by a factor of 0.67 to account for wind reduction from Category 1 terrain to Category 4 terrain (Australian Wind Standard AS1170.2, Table 4.1(A)). The hydrodynamic model used wind-drag coefficients at 10 m above the water level and therefore it was not necessary to reduce the wind speed further to take into account wind-height transformation. The derived morphological wind boundary conditions for the model are plotted in Figure 4.

Figure 4: Model Morphological Wind




360
270
180
90
0
Direction degree
08/01
09/01
Date

2. Model Verification

2.1 Mass Balance

Steep slopes are apparent in the basin geometry at the toe of the breakwater. The vertical diffusion was increased artificially in the model numerical solution to account for this. It is possible to obtain stable mass-balance in true sigma coordinates for highly diffusive tracers, where stratification is not physically possible. However, for relatively low diffusive tracers the Delft3D sigma coordinate correction is activated to reduce this behaviour. Such a limitation is intrinsic to Delft-3D modelling and all the calculations were performed with this option.

The mass-balance error between the pipe outlet flow and the weir was found to be less than 0.1% at any time, which is not significant. Diffusive tracers were added instantaneously at each of the three supply pipe outlets to verify the stability of the transport model. The quantities of tracer lost in the basin and exiting through the weir were compared. At any time, the difference was less than 1% of the total mass of tracer remaining in the basin.

2.2 Turbulence Model and Sensibility Analysis

Delft-3D-Flow horizontal and vertical eddy viscosity parameters are coefficients multiplying the intensity of the Reynolds stresses, which account for flow momentum loss due to viscosity. Eddy viscosity influences the “flow turbulence”. Delft-3D horizontal and vertical eddy diffusivity parameters are coefficients multiplying the rate of diffusive transport for a given tracer (whether the tracer is heat, salinity, suspended sediment etc…). Eddy diffusivity influences the “tracer mixing capacity” in an advective flow corrected for viscous turbulence losses. A very small eddy viscosity has similar characteristics to “laminar flow” as the flow is purely advective and there is no energy loss through viscosity. A tracer set-up having small eddy diffusivity would mix principally due to turbulence and advection. Both parameters influence the rate of transport of tracers.

Both parameters are used as inputs to a selection of turbulence sub-models. For the k-ε model, viscous parameters are calculated directly by the turbulence model and only minimum viscosity-input-parameters are required. The Ozmidov length scale can be set-up to increase the vertical turbulence for buoyant flow. This parameter was optimised to increase the opportunity to build stratification in the model while producing a stable turbulence model.

The residency time is a numerical model statistical result, which indicates the time spent by a particle within the basin or “reactor”. Residence time depends on the hydrodynamics of the reactor that is numerically modelled by advection, turbulence, eddy viscosity and eddy diffusivity. The mixing parameterisation of a real tracer depends on parameters that should be calibrated to real on-site measurements (eddy viscosity and eddy diffusivity calibration). In this case study, no measurements are available for calibration.

However, the influence of these parameters on real-flows can be explored with two idealised systems: the “Plug Flow Reactor” (PFR) and the “Continuously Stirred Tank Reactor” (CSTR). When particles enter a PFR they exit the system linearly through advection. In this case, turbulence is not a major driver of the transport characteristics. When particles enter a CSTR they exit exponentially through instantaneous dilution. In this case turbulence significantly influences the flow characteristics.

The flushing efficiency of a real basin can be assimilated to a composition of both ideal reactor systems. The sensitivity of a PFR system to diffusion is small, while it would be important for a CSTR system.

2.3 Hydrodynamic Model

The basin flow was calculated for a steady wind condition, where the fluxes in the basin are due to the pump-weir actions, turbulence and surface wind shear. The morphological wind presented in Section 1.2 represented the model wind boundary condition. The residence time at the top, averaged over the water depth, and at the bottom of the basin were calculated (Figure 5) and Table 4 summarises the typical residence times calculated at various locations across the basin.

Table 4: Estimated Typical Residence Time (hours)

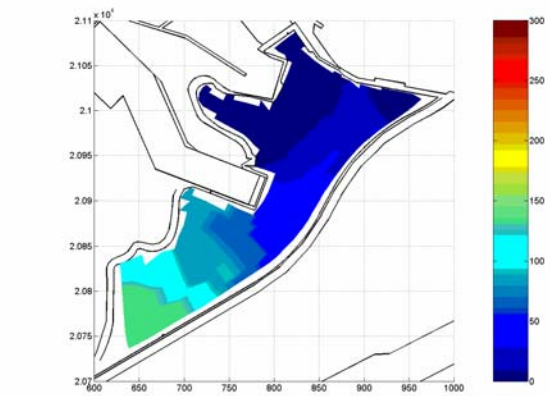
Location	$T_{50\%}$ (hours)			$T_{90\%}$ (hours)		
	Top	Average	Bottom	Top	Average	Bottom
Lagoon (d<5m)	10	10	20	50	50	70
Lagoon Deep Water (d>5m)	30	30	50	200	200	320
Sill	50	70	100	300	300	370
Marina (d<5m)	90	100	150	370	370	450
Marina (d>5m)	100	100	150	370	370	420

The turnover-rate is the time required for the basin volume to be discharged through the weir, and is approximately 190 hours. The "median residence time" ($T_{50\%}$) is the time required for a peak concentration of tracers to decrease by 50%. The "significant residence time" ($T_{90\%}$) is the time required for peak concentration of tracer to decrease by 90%.

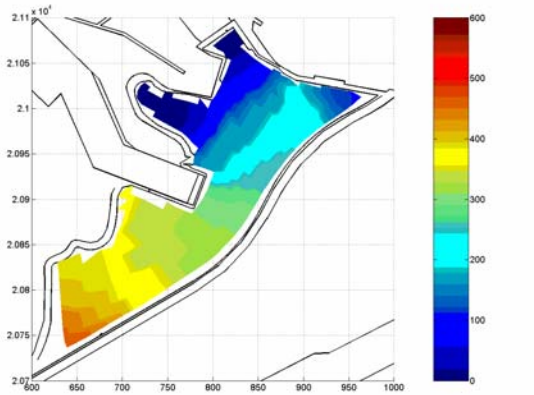
The basin median residency time is typically less than 150 hours. The median residence time of the lagoon is low, (10 to 50 hours). The bottom layer is well mixed in the lagoon because the deep-water outlet O1 is advecting flow and induces additional turbulence. As there is no such source of current in the marina and the median residence time reaches 100 to 150 hours. Half of the initial tracer peak concentration is lost within 150 hours in the Marina. The significant residence time of the lagoon is approximately equal to the turnover rate of the basin. The significant residency time is substantially higher than 180 hours in the Marina. In the deepest area of the marina there are only 10% of particles remaining after 450 hours.

As the turnover rate in the lagoon is approximately equal to the significant residence time, the system equates to a PFR model for this area. Particles advected in the lagoon are flushed very efficiently through the sill. As the turnover rate in the marina is approximately equal to the median residence time, the CSTR system approximates the best flow condition in the marina. The marina fluid circulation is largely influenced by dilution: thus it would take longer to flush this area. The sensitivity to the turbulence modeling of the lagoon part of the model is low, while it is higher in the marina.

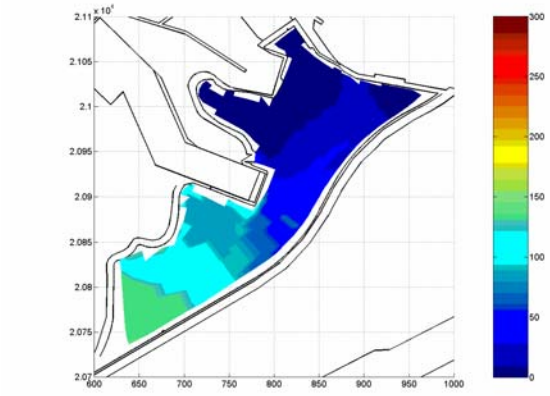
Thus the lagoon is well flushed with low residency times. The marina has higher residency times but not to the extent that problems occur – refer stratification modeling following. Note that the marina was modeled without the lock. The operation of the lock will also improve the flushing of the marina.



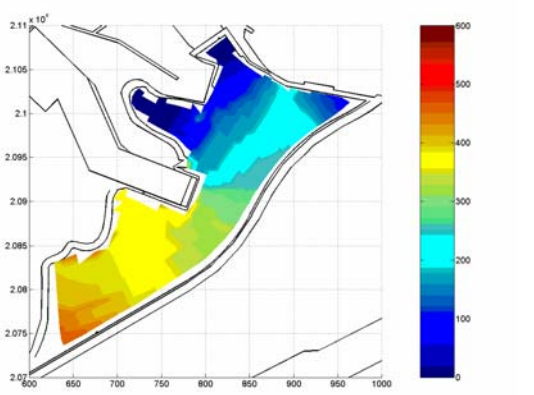
Top Layer T_{50}



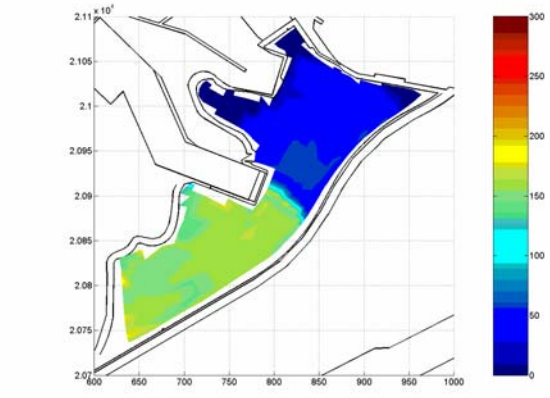
Top Layer T_{90}



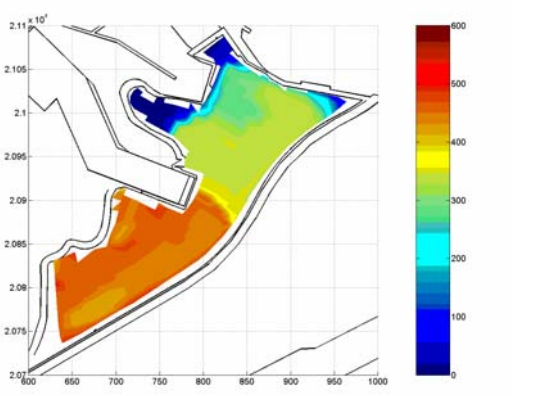
Average over all Layers T_{50}



Average over all Layers T_{90}



Bottom Layer T_{50}



Bottom Layer T_{90}

Figure 5: Residence Time (Hours) –Morphological Wind Forcing – Composite Bathymetry

3. Heat Exchange Modelling

The Excess Temperature Heat Flux model was set-up to evaluate thermal stratification of the lagoon. It is a robust, yet relatively sophisticated heat exchange model. Model inputs consist only of the natural background air temperature and an activation surface, which was taken to be the full lagoon surface. When the background temperature is equal to the water surface temperature the heat flux is nil, otherwise the heat is transmitted from the warmer fluid to the colder fluid following a quadratic law, the efficiency of which increases with the wind speed (Sweers 1976). The heat mixing capacity of the lagoon was modelled for the dry and wet seasons.

3.1 Dry Season

Table 5 contains the model inputs corresponding to the dry season. The model was run for a period of 45 consecutive days using the morphological wind boundary condition, to be conservative. It was found that equilibrium developed over approximately three morphological days.

Table 5: Dry Season Inputs

Input	Value
Background Air Temperature	Oscillating from 21.0°C (morning) to 30°C (evening)
Background Water Temperature	26.5°C uniform in basin and at the pump outlets
Background Salinity	36ppt uniform in basin and at the pump outlets
Cullen Bay Typical Water Temperature	26 to 28°C uniform (no stratification)
Cullen Bay Typical Water Salinity	36ppt uniform (no stratification)

Salinity: The model salinity was found to be uniform over the water column, approximately 36ppt. The deeper water is slightly more saline approaching 36.1ppt, and top layer salinity oscillates ±0.05ppt following the diurnal air temperature variations.

Temperature: The model surface temperature varied from 25.8°C to 26.7°C following the diurnal temperature cycle. The deepest layer temperature was nearly constant, approximately 25.9°C.

Under these dry conditions the model showed no significant stratification. Existing marina basins in Darwin are typically unstratified during the dry season.

3.2 Wet Season

Table 6 contains the model inputs for the wet season modelling. The model was run for 45 days using the morphological wind boundary condition, to be conservative. It was found that equilibrium developed over approximately 10 morphological days.

Table 6: Dry Season Inputs

Input	Value
Background Air Temperature	Oscillating from 27°C (morning) to 30.2°C (evening)
Background Water Temperature	30°C uniform in basin and at the pump outlets
Background Salinity	28ppt uniform in basin and at the pump outlets
Cullen Bay Typical Water Temperature	Bottom 35°C, Surface 31°C
Cullen Bay Typical Water Salinity	Bottom 20ppt, Surface 10ppt

Salinity: The salinity in the lagoon oscillates between 27.9ppt and 28.1ppt. The deepest water salinity was approximately 28.1ppt, whilst the salinity in the top layers oscillated between 27.9ppt and 28.0ppt following diurnal background temperature variations.

Temperature: The surface temperature changed from 29.1°C to 29.5°C degrees overnight but the bottom temperature remained at approximately 29.0°C degree.

During the modelling, two layers of water could be distinguished in the model: a slightly colder, fractionally more saline body of water at the bottom (26°C, 28.5ppt) and a slightly warmer, fractionally fresher body of water at the surface (27°C, 27.5ppt). However, diurnal water quality variations were similar to the difference in temperature magnitude. Compared with Cullen Marina, this “stratification” is negligible. The model therefore showed that during the wet season no thermal significant stratification occurs.

4. Results and Recommendations

The following conclusions can be drawn from the numerical model study:

- The median residence times of the lagoon are low (10 to 50 hours). The bottom layer is well mixed in the lagoon because the deep water outlet O1 is advecting flow and induces additional turbulence.
- Approximately 2 days are necessary to replace 50% of the water intake in the lagoon, in the marina the median residence time reaches approximately 6 days;
- Approximately 8 days are necessary replace 90% of the water intake, except in the marina where the significant residence time reaches approximately 20 days;
- The marina residence time is longer than the basin turnover rate and the residence time in the lagoon is smaller than the basin turnover rate. The water quality in the lagoon would be similar to an equivalent PFR system and in the marina to an equivalent CSTR system;
- The water quality at the sill is characteristic of a transition area and would be relatively uniform over its depth;
- The model was unable to produce a stable stratification.

The following recommendations can be made, based on the results of the modelling:

- If water quality becomes an issue, mitigation measures including a redirection of pump outlets and additional mixers may be envisaged;
- Because mixers principally influence turbulence and the turbulence is naturally low in the marina, it is likely that these could reduce the possibility of any salinity stratification. In comparison, re-direction of the pump outlets would influence both turbulence and advection and could have a more substantial effect;

In using this report, it is important to consider that this numerical model has the following limitations:

- The model is not calibrated as no prototype measurements were available at the time of the study, assumptions are made on a number of model parameters, as noted above; and
- The sensitivity to diffusion of the model is relatively important.

Therefore the model is a useful tool in providing a general picture of the basin performance and relative tendencies. However the model outputs at this stage are more of qualitative value than quantitative.