# RINGSEND WASTEWATER TREATMENT WORKS

# DESIGN REVIEW REPORT
# FOR SUBMISSION TO EPA



Jan 2010 Report

Dublin City Council
Comhairle Cathrach Bhaile Átha Cliath

# Document Control Sheet

| | | | | |
|---|---|---|---|---|
| Client | Dublin City Council | | | |
| Project | Ringsend Wastewater Treatment Works Extension | | | |
| Report | Design Review Report for Submission to EPA | | | |
| Date | January 2010 | | | |
| Project No: 67511 | | Document Reference: DG28 | | |
| Version | Author | Checked | Reviewed | Date |
| 1 | Robert Gaudes | Ken McIntyre<br>Al Pincince | Robert Gaudes | 4 Jan 2010 |
| | | | | |
| | | | | |
| | | | | |

# Table of Contents

# Appendices

Appendix A Ringsend Wastewater Treatment Works Extension Baseline Report

Appendix B Overview of Geology in Dublin Bay

Appendix C Ringsend WwTW Proposed Tunnelling Outfall Cost

# List of Tables

# List of Figures

# Section 1
# Executive Summary

The overriding purpose of the Ringsend WwTW Extension project is to extend the Works from its present capacity to the maximum achievable within the curtilage of the existing site and to achieve the required discharge standards.

Dublin Bay Project Contract 2 provided facilities to treat 1.64 million PE to secondary standards, specifically: 25 mg/L BOD; 125 mg/L COD; 35 mg/L TSS; and 18.75 mg/L Ammonia Nitrogen. An open space of 0.8 hectares was reserved to extend the Works to 2.15 million PE, assuming the same effluent standards.

The average influent loading to the Works is currently approximately 1.8 million PE. A flow and loading analysis estimates a 2025 design year average daily loading of 2.2 million PE and a maximum weekly loading of 3.3 million PE. Projections include an average industrial loading of 400,000 PE and a growth rate of 0.7% per annum from a datum of 1.8 million PE. While the planning period to 2025 is rather short, it is anticipated that a regional treatment works will be constructed to the north of Dublin in the next decade, providing relief to the Ringsend Works.

The projected loadings are similar to those planned for Contract 2. However, effluent standards have become more stringent. In 2001, the Liffey River Estuary was declared to be a sensitive water body under the Urban Waste Water Treatment (UWWT) directive, requiring nutrient removal to achieve 10 mg/L Total Nitrogen and 1 mg/L Total Phosphorus for continued discharge into the estuary.



The Works, as currently configured, has limited ability to remove nutrients. If denitrification filters are installed downstream of the Sequencing Batch Reactors, SBRs, approximately 1.5 million PE can be treated to the UWWT Total Nitrogen Standard. Without the denitrification system, the Works can only treat about 1.0 million PE.

Chemically Enhanced Primary Treatment (CEPT) was considered to decrease loadings to the SBRs and to control Phosphorus. If applied, it could increase the overall Works' capacity to 1.9 million PE (with denitrification filters). Treatment to remove nitrogen from sidestreams of sludge processing was also considered. While not increasing the Works' capacity, it would reduce oxygen consumption in the SBRs and decrease nitrogen loading on denitrification filters. Several treatment scenarios including CEPT and/or sidestream treatment in addition to extended biological treatment were considered.

The restricted space available for biological treatment reduced the viable alternatives to deep shaft aeration and membrane bioreactors (MBRs). Deep shaft systems utilize concentric pipelines drilled to approximately 100 m depth as aeration basins. Flotation clarifiers would be used to separate the solids prior to discharge. The system would be designed to nitrify and denitrify. Depending upon the degree of pretreatment, deep shaft systems could be provided for 0.3 million PE or 0.7 million PE. The former would be preceded by CEPT. The latter would not require pretreatment, but would require construction on both sides of Pigeon House Road. MBRs could be for 0.3 million PE, but would require CEPT.

Chemical sludge produced from CEPT and/or phosphorus control in the nitrification filters would exceed the capacity of the sludge stream, requiring further expansion beyond its (year 2010) capacity of 120 tonnes per day.

Alternatives including deep shaft aeration have present worth costs ranging between €211 million and €223 million. Alternatives including MBRs had present worth costs of €244 million and €257 million.

Ocean outfalls, discharging at secondary treatment standards of 25 mg/L BOD, 125 mg/L COD and 35 mg/L, were considered as alternatives to continued discharge to the Liffey River Estuary at UWWT limits. If the SBRs were to be operated in a manner that would avoid nitrification, the existing facilities would be capable of treating 2.2 million PE with only the addition of blower capacity. No chemical sludge would be produced and the sludge stream would not require further extension. By decreasing the average mixed liquor concentration to approximately 2,200 mg/L, sedimentation, and therefore effluent TSS quality, should improve. As added comfort, covers that would eliminate wind effects on the upper SBR level, would be provided.



Two outfall scenarios, at lengths of 7.5 km and 10 km, were considered for the discharge of secondary effluent. Based on initial dispersion modelling and environmental assessments, there is confidence that an outfall terminus falling within this range of lengths is likely. Present worth costs are heavily influenced by outfall length, with present worth costs ranging from €176 million and €218 million, or between 83% and 103% of the next lowest cost alternatives.

Non-cost factors heavily favored the secondary treatment/ocean outfall alternative. Power consumption, directly attributed to wastewater treatment and indirectly derived from sludge treatment, was 50% to 90% greater for the nutrient removal alternatives. Chemical consumption, which is zero for the secondary treatment/ocean outfall alternative, ranges from 10,000 $m^3$/yr to 20,000 $m^3$/yr. Sludge production for nutrient control alternatives is estimated to be 3,400 tonnes per year or 9,600 tonnes per year, depending upon how phosphorus is removed. Power, chemicals and sludge all generate greenhouse gases.

In addition to being the low cost alternative, secondary treatment with an ocean outfall is the low-risk alternative. It consumes less energy and chemicals and produces less sludge and greenhouse gases. It requires no new unit process and is, therefore, much simpler to operate and maintain. The discharge, while not treated to the same levels as the other alternatives, would meet water quality standards and be more protective of existing Natura 2000 sites and bathing waters.

Pending an environmental impact assessment, it appears that providing secondary treatment with an ocean outfall discharge would be the most beneficial option for the Ringsend WwTW Extension.

# Section 2 Introduction

A Preliminary Report for the Ringsend Wastewater Treatment Works (WwTW, Works), dated May 1993 included a recommendation that the Works be commissioned in two stages. Stage I was to be designed and constructed immediately on the basis of a design horizon of 2015. The site for Stage II extension was proposed to be within an area set aside on the Stage I site. Thus, the requirement for a Preliminary Report has been satisfied. This Design Review Report addresses the needs to expand the Stage I facilities to the ultimate capacity that can be achievable on the current site.

Section 2 sets the background for the Extension, reviewing design standards for the current Works and historic compliance with them, followed by the economic factors that are used in evaluating alternatives.

Section 3 provides a review of current flows and loadings to the Works and projects forward to the Design year of 2025.

Section 4 evaluates the capacity of the Works to comply with standards for discharge into the Liffey River Estuary as well as for discharge beyond Dublin Bay into less sensitive waters that would not require nutrient removal or effluent disinfection. A number of alternative unit processes, and combinations of unit processes, are evaluated. Those alternatives that can achieve effluent compliance within the curtilage of the existing site are carried forward.



Section 5 investigates the feasibility and cost of long sea outfalls to convey wastewater treated to secondary treatment standards.

Section 6 provides cost estimates and discusses non-cost factors for those alterantives surviving initinal screening in Sections 4 and 5.

Section 7 provides a comparison between cost and non-cost factors for each of the alternatives, draws conclusions and recommendation the preferred option for the Ringsend WwTW Extension

## 2.1 Existing Facilities

The Ringsend WwTW was extended to its current configuration under the Dublin Bay Project Contract No. 2. Contract No. 2 was procured under a design/build/operate scheme. The Works was officially handed over to the operator in May 2005.

The parameters listed in Table 2.1 constitute the Basis of Design for Contract 2 Works.

*Table 2.1 Basis of Design, Contract No. 2*

| **Description** | |
|---|---|
| **Average Daily Flow (ADF)** | 5.7 $m^3$/sec |
| **Flow to Full Treatment (FFT)** | 11.1 $m^3$/sec |
| **Peak Instantaneous Flow** | 23.0 $m^3$/sec |
| **Influent BOD load**<br>Average<br>95th Percentile | <br>98,400 kg/day (200 mg/L)[1]<br>156,700 kg/day |
| **Effluent BOD**<br>95th Percentile<br>Not to be Exceeded | <br>25 mg/L<br>50 mg/L |
| **Influent COD load**<br>Average<br>95th Percentile | <br>225,100 kg/day (445 mg/L)[1]<br>383,300 kg/day |
| **Effluent COD**<br>95th Percentile<br>Not to be Exceeded | <br>125 mg/L<br>250 mg/L |
| **Influent TSS load**<br>Average<br>95th Percentile | <br>101,100 kg/day (205 mg/L)[1]<br>194,300 kg/day |
| **Effluent TSS**<br>95th Percentile<br>Not to be Exceeded | <br>35 mg/L<br>87.5 mg/L |
| **Influent Nitrogen load**<br>Total N - Average<br>Total N – 95th Percentile<br>Ammonia N – Average<br>Ammonia N – 95th Percentile | <br>15,600 kg/day (31.7 mg/L)[1]<br>21,400 kg/day<br>9,500 kg/day (19.3 mg/L)[1]<br>12,800 kg/day |
| **Effluent Ammonia Nitrogen**<br>95th Percentile<br>Not to be Exceeded | <br>18.75 mg/L<br>47 mg/L |
| **Influent Total Phosphorus**<br>Average<br>95th Percentile | <br>3,700 kg/day (7.5mg/L)[1]<br>5,600 kg/day |

*1. As computed from ADF*



In addition to achieving effluent limits on BOD, COD, TSS and Ammonia Nitrogen, the Works must disinfect during the bathing season to achieve 100,000 Faecal Coliform bacteria per 100 ml sample (100,000 FC/100 ml) on an 80 percentile basis.

The Year 2020 design BOD loading to the Works, as expressed in population equivalents (PE), is 1.64 million PE. The design envisaged expansion to 2.15 million PE by constructing two more sequencing batch reactors (SBRs) on 0.8 hectares of open space within the curtilage of the existing site.

Pollutant loadings to the Works have exceeded the Year 2020 design projections ever since Contract 2 entered the operations phase. Notwithstanding the adverse loading conditions, the Works has regularly achieved its effluent limits for BOD, COD, Ammonia Nitrogen and Faecal Coliform. There are infrequent exceedances of upper limits, but the Works has met the respective 95$^{th}$ percentile and 80$^{th}$ percentile compliance limits for these parameters. Effluent TSS, however, has achieved compliance with the 95$^{th}$ percentile standard of 35 mg/L only 82 percent of the time The upper level limit of 87.5 mg/L is exceeded on average about once per month.

After Contract No. 2 was signed, the Liffey River Estuary was designated as Nutrient Sensitive Waters under the Urban Waste Water Treatment (UWWT) directive. Consequently, annual mean limits of 10 mg/L total nitrogen (TN) and 1 mg/L total phosphorus were set on effluent from the Works. As currently configured, the Works are incapable of meeting the UWWT standards at the Year 2020 design loading.

Storm tanks receive flows in excess of 11.1 $m^3/s$ and store the wastewater for treatment when influent flows subside. On infrequent occasions the storm tanks overflow to the Liffey Estuary. There is a limit of 3,000,000 FC/100 ml in the storm water discharge, which has never been exceeded.

A more detailed discussion of existing facilities may be found in Appendix A, *"Ringsend Wastewater Treatment Works Extension Baseline Report"*.



There have been a number of modifications to the Works subsequent to the taking over of Contract No. 2. Most of these modifications were related to odour control and solids processing, with little direct impact on wastewater treatment capacity or efficiency. There is an ongoing project, designated "Sludge Stream Expansion Option 11A", which will increase the Works' digestion capacity by 30 tonnes per day, add a third thermal hydrolysis train, one new centrifuge and three Surplus Activated Sludge (SAS) thickeners. The SAS thickeners should provide sufficient capacity to thicken all the SAS generated in the SBRs. This is significant in that co-settling of SAS in the primary clarifiers can be virtually eliminated (save a small amount that is deemed beneficial to settling), improving removal efficiency in the primary clarifiers and, thereby, reducing solids loading to the SBRs. It is hoped that the reduction of solids loading to the SBRs will improve effluent TSS quality. The SAS thickeners were commissioned in December 2009 and so their effect on effluent TSS should become apparent shortly.

## 2.2 Discharge Monitoring

The effluent discharge (SW1) is located in a cooling water channel north of the ESB Ringsend Power Station. The storm water overflow pipe (SW2) is to the north of the storm tanks. Influent and effluent sampling locations as well as outfalls are shown on Figure 2.1.


*Figure 2.1 Primary Monitoring Points*

## 2.3 Project Objectives

The Over-Riding Purpose of the Ringsend WwTW Extension Project is to extend the Works from its present capacity to the maximum achievable within the curtilage of the existing site and to achieve the required discharge standards.

Further, the proposed extension works shall not result in the diminution of the capacity of the existing Works to function, at any stage, during the construction and commissioning of the proposed works.



## 2.4 Economic Factors

The economic factors presented in Table 2.2 are used throughout this report. It is understood that the Discount Rate is subject to change over time.

Given the lack of definition at the planning level stage, a contingency of 35% is placed on capital costs.

*Table 2.2 Project Economic Factors*

| | |
|---|---|
| **Electricity** | €0.0125 per kWh |
| **Natural Gas** | €0.04 per kWh |
| **Alum** | €0.13/litre @ 54.6 gr Aluminium/litre |
| **Methanol** | €0.42/litre |
| **Sludge Disposal Costs** | €140 per dry tonne |
| **Discount Rate** | 4.49 percent |
| **Term** | 20 years |
| **Uniform Series Present Worth Factor** | 13.02 |
| **Capital Recovery Factor** | 0.0768 |

## 2.5 Site Plan

An overall plan of the existing site is presented in Figure 2.2. In addition to the existing structures, boundaries and roadways, the figure shows the locations of major underground pipelines and channels.

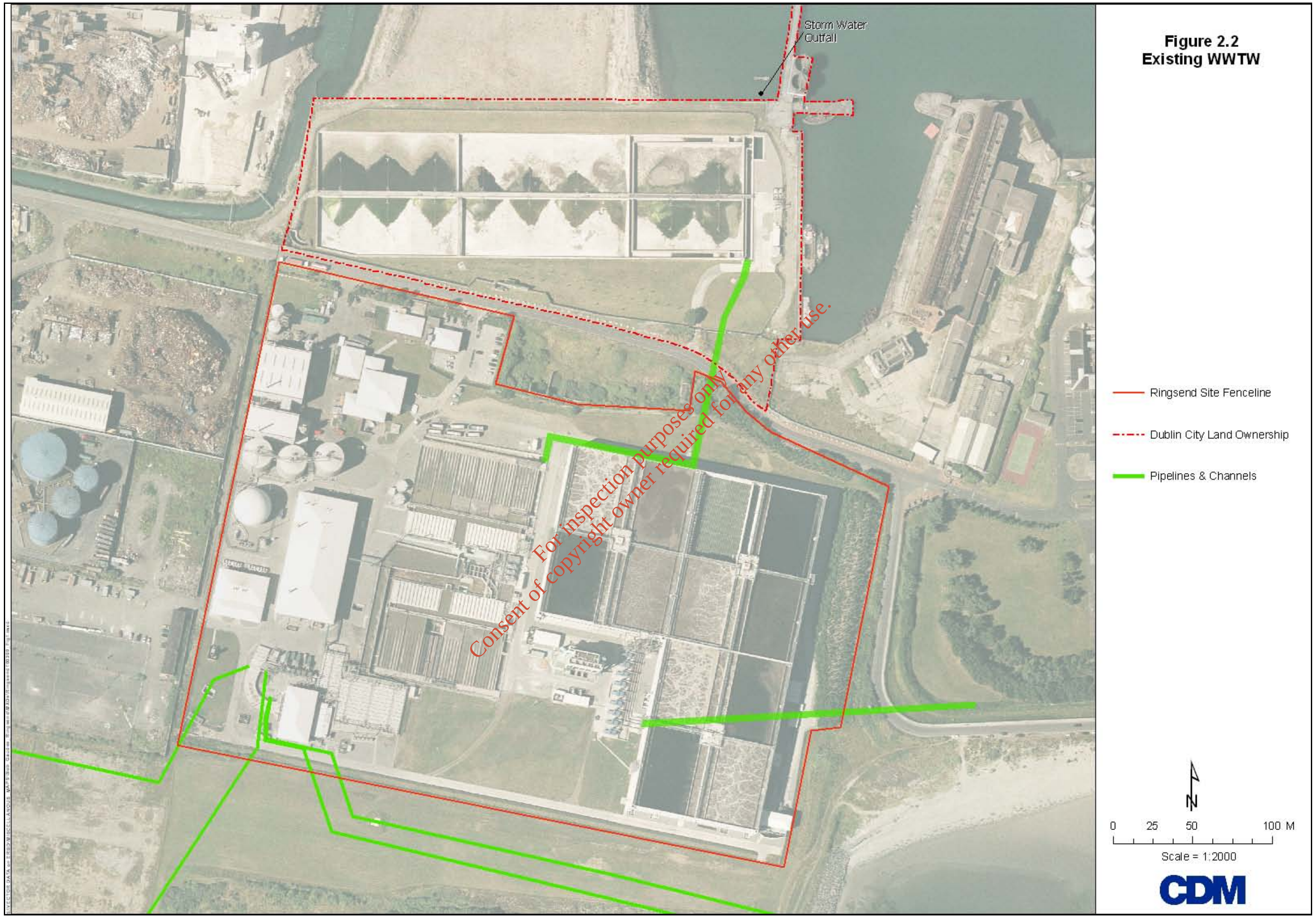




*Figure 2.2 Site plan of the existing WWTW*

# Section 3
# Loading Analysis

## 3.1 Current Loading

In this section the characteristics of the wastewater received at the Ringsend Waste Water Treatment Works (WwTW), in the Period 2003 to December 2008, is assessed. Analytical data was collected from the contractor, Celtic Anglian Water (CAW) and their subcontracted analytical Laboratory City Analysts as well as from the Dublin City Council's Central Laboratory.

### 3.1.1Flow Analysis

#### 3.1.1.1 Background & Design

Flows are received to the WwTW from the following catchment areas:

- Main Lift Pumping Station (MLPS);
- West Pier Dun Laoghaire Pumping station;
- Sutton Pumping Station; and
- Dodder Valley Siphon.



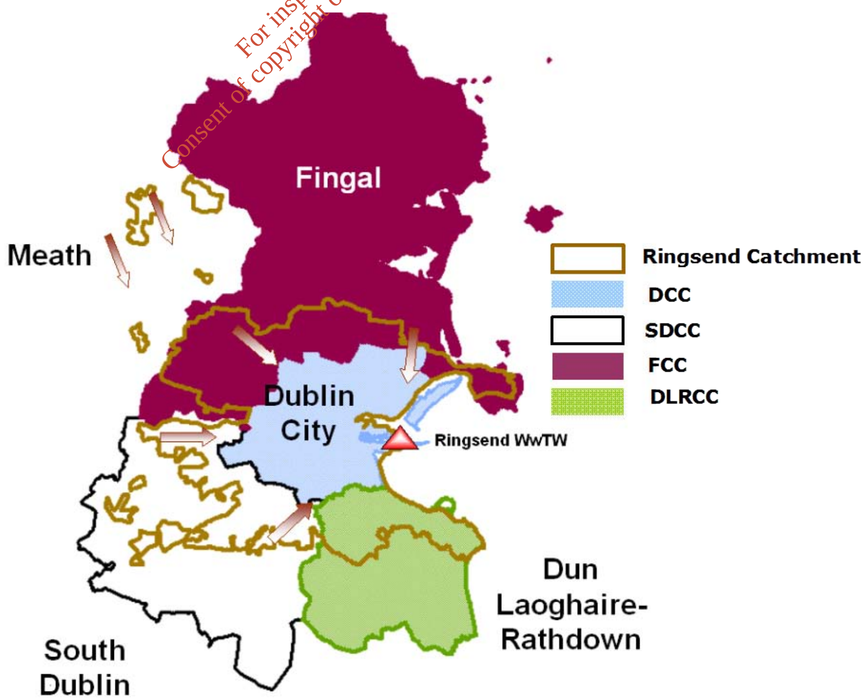



*Figure 3.1 Catchments to Ringsend (Source: GDSDS 2008)*

The peak storm flow to Ringsend is 22.6 $m^3/s$ and storm holding tanks cater for flows in excess of full flow to treatment (FFT), which is 11.1 $m^3/s$. Storm holding tanks are also provided at Sutton to cater for severe storm conditions. Design flows are shown in Table 3.1.

*Table 3.1 Current Ringsend Design (Source: Tender Documents Vol. 2 Employers Requirements 1998)*

| Design Flow Basis for Ringsend WwTW year 2020 and ultimate Design Year | | | |
|---|---|---|---|
| | **Estimated 2001 Flows** | **Design Year 2020 Average Design** | **Ultimate Design Year Average Design** |
| **Design Parameter** | **($m^3$/s)** | **($m^3$/s)** | **($m^3$/s)** |
| Dry Weather Flow (DWF) | 3.8 | 4.6 | 5.5 |
| Average Daily Flow | 4.8 | 5.7 | 6.9 |
| Full Flow to Treatment (FFT) | 11.1 | 11.1 | 13.8 |
| Peak Flow | 22.6 | 22.6 | 23.5 |

### 3.1.1.2 Measured Flows

Flows are measured by Dublin City Council (DCC) for Sutton, Dodder Valley and the West Pier Dun Laoghaire Pumping station in addition to the recorded flows at Ringsend WwTW.

Average influent flows have increased continuously since 2003 and the 2008 average daily flow (ADF) rate (470,480 $m^3$/d) is just over 95% of the design ADF of 492,480 $m^3$/d (Table 3.2).



*Table 3.2 Measured Flows ($m^3$/d)*

| **Parameter** | **Average** | **95%ile** | **Maximum** | **Count** | **Standard Deviation** |
|---|---|---|---|---|---|
| **Period** | | | | | |
| Aug '03-Dec '08 | 401,881 | 636,338 | 1,352,012 | 1,978 | 122,862 |
| 2003 | 330,116 | 480,267 | 1,017,421 | 153 | 113,046 |
| 2004 | 387,343 | 601,063 | 1,352,012 | 365 | 118,426 |
| 2005 | 393,205 | 585,354 | 991,310 | 365 | 103,265 |
| 2006 | 381,316 | 593,488 | 922,703 | 365 | 106,449 |
| 2007 | 407,154 | 630,737 | 1,114,190 | 365 | 114,077 |
| 2008 | 470,480 | 794,244 | 1,102,283 | 365 | 141,337 |
| Design | 492,480 | | | | |

### 3.1.1.3 Storm Flows

Storm discharges to the Estuary outfall, account for less than 1% of all measured inflows to the plant. Influent flow data indicates that a high proportion of all incoming flows arriving at the works are receiving full treatment (Table 3.3). August 2008 was a particularly wet month with average daily flows of 599,112 $m^3$/d to the plant (maximum 974,208). During this peak flow period, greater than 96% of all flows received full treatment.

*Table 3.3 Ringsend Storm Flows*

| | Average Daily Influent ($m^3$/d) | Average Daily Storm Flow to Liffey ($m^3$/d) | % of Total Flow Treated |
|---|---|---|---|
| 2003 (from 1st Aug) | 330,116 | 6793.1 | 97.9 |
| 2004 | 387,343 | 4363.5 | 98.9 |
| 2005 | 393,205 | 3108.8 | 99.2 |
| 2006 | 381,316 | 829.9 | 99.8 |
| 2007 | 407,154 | 3142.8 | 99.2 |
| 2008 (to end Aug) | 457,302 | 6976.0 | 98.5 |

## 3.1.2 Load Analysis

### 3.1.2.1 Background & Design Load

The current plant was designed to treat a population equivalent (PE) of 1.64 million with 2020 design year average BOD and TSS loads of 98.4 t/d and 101.1 t/d, respectively (Table 3.4). Domestic design average pollutant loads were estimated based on per capita contributions of 60 g BOD/c/d and 75g TSS/c/d, 8g AmmN/c/d, 12g TN/c/d and 3 g TP/c/d (ref: Employers Requirements Design-Build Works 1998).



*Table 3.4 Current Ringsend Design (Source: Employers Requirements Design-Build Works 1998)*

| Design Load Basis for Ringsend WwTW year 2020 | | | | |
|---|---|---|---|---|
| | Estimated 2001 Loads | | Design Year 2020 | |
| | Average Load | 95%ile | Average Load | 95%ile |
| Design Parameter | (kg/d) | (kg/d) | (kg/d) | (kg/d) |
| BOD | 88,300 | 141,400 | 98,400 | 157,600 |
| TSS | 89,000 | 171,000 | 101,100 | 194,300 |
| Ammonia (N) | 8,100 | 10,900 | 9,500 | 12,800 |
| Total Nitrogen (TN) | 13,600 | 18,600 | 15,600 | 21,400 |
| Total Phosphorus (TP) | 3,200 | 4,800 | 3,700 | 5,600 |

### 3.1.2.2 Measured BOD Load

Since the plant was commissioned in 2003, the measured annual average BOD loads to the WwTW have been continuously higher than the 2020 design figure of 98.4 t/day (Table 3.5). Further, the 95 percentile load was higher than the stipulated 2020 design load for all years except 2007.

The Operational data also indicate a high variability in influent BOD load being received at the WwTW. Figure 3.2 shows the trend in BOD loadings to the plant in 2007 along with the 2020 design average values. This figure also illustrates the variability of the incoming load.

*Table 3.5 BOD Load (t/d)*

| Parameter | Average | 95%ile | Maximum | Count | Standard Deviation |
|---|---|---|---|---|---|
| **Period** | | | | | |
| Aug'03-Dec '08 | 112.5 | 168.4 | 361.5 | 1385 | 33.6 |
| 2003* | 111.0 | 157.3 | 221.8 | 107* | 26.8 |
| 2004 | 118.8 | 162.8 | 237.6 | 257 | 26.8 |
| 2005 | 117.9 | 162.3 | 257.6 | 257 | 26.4 |
| 2006 | 117.3 | 195.1 | 293.3 | 258 | 40.0 |
| 2007 | 101.5 | 140.3 | 361.5 | 249 | 32.2 |
| 2008 | 107.4 | 181.7 | 289.8 | 256 | 39.0 |
| Design | 98.4 | 157.6 | | | |

**Data for 2003 begins 1st August*

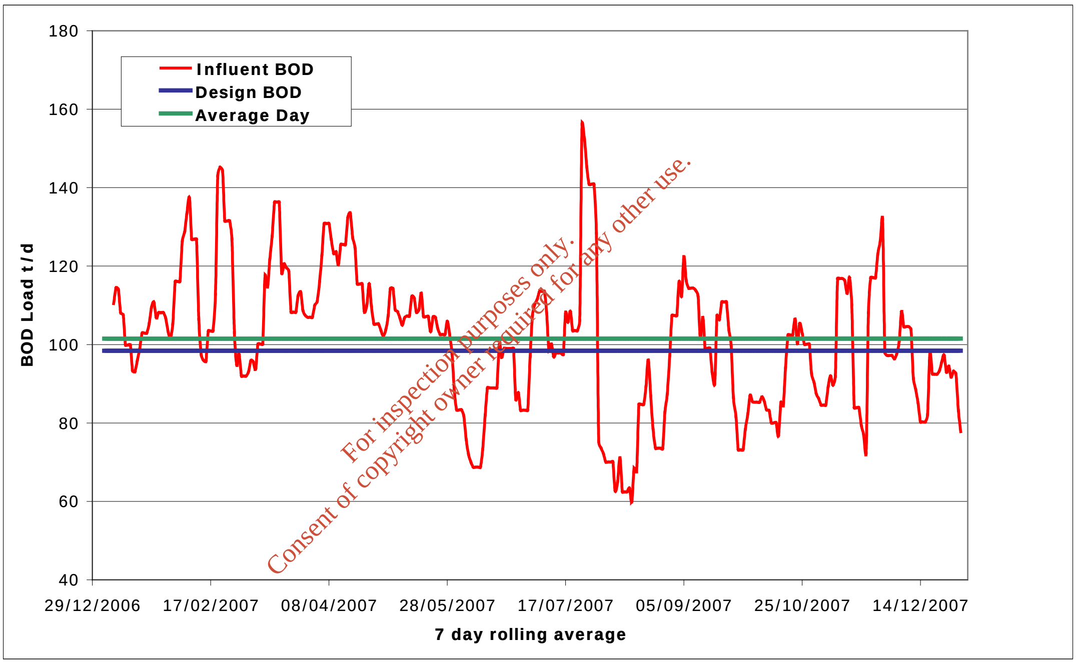





*Figure 3.2 Sustained BOD loads received at the WwTW in 2007. BOD(t/d) vs time.*

The 2008 data also showed periods of high load. The period April-May 2008 was a particularly stressed month on the plant. Influent BOD loads to the works averaged 188.3 t/d over a 30-day period. This corresponds to an average of 3.14 million PE for the period or 192% of the design basis, with a peak daily load of 4.83 million PE.

### 3.1.2.3 Measured TSS Load

Influent TSS loading has increased steadily since 2003 (Table 3.6). The average daily TSS loading in 2008 exceeds the 2020 design year loading and is currently 11% higher than the design.

*Table 3.6 TSS Load (t/d)*

| Parameter | Average | 95%ile | Maximum | Count | Standard Deviation |
|---|---|---|---|---|---|
| **Period** | | | | | |
| Aug '03-Dec '08 | 99.4 | 151.2 | 860.2 | 1959 | 45.0 |
| 2003 | 89.8 | 128.8 | 211.1 | 153* | 23.7 |
| 2004 | 94.0 | 135.6 | 244.4 | 364 | 24.3 |
| 2005 | 96.2 | 142.7 | 530.7 | 363 | 35 |
| 2006 | 96.2 | 143.3 | 748.4 | 365 | 59.2 |
| 2007 | 102.3 | 146.7 | 742.1 | 352 | 47.5 |
| 2008 | 112.5 | 175.5 | 860.2 | 364 | 54.1 |
| Design | 101.1 | 194.3 | | | |

**Data for 2003 begins 1st August*

### 3.1.2.4 Measured Nutrient Load

The 2008 average total nitrogen (TKN + Nitrite + Nitrate) loading to the plant amounts to approximately 17 t/d. Figure 3.3 illustrates the increasing Ammonia and TKN loads to the site from 2003 to 2008 relative to the design loads. It is noted that from 2005 the TN load has been higher than the 2020 design of 15.6 t/d and since 2006 the Ammonia Load has exceeded the design figure of 9.5 t/d.



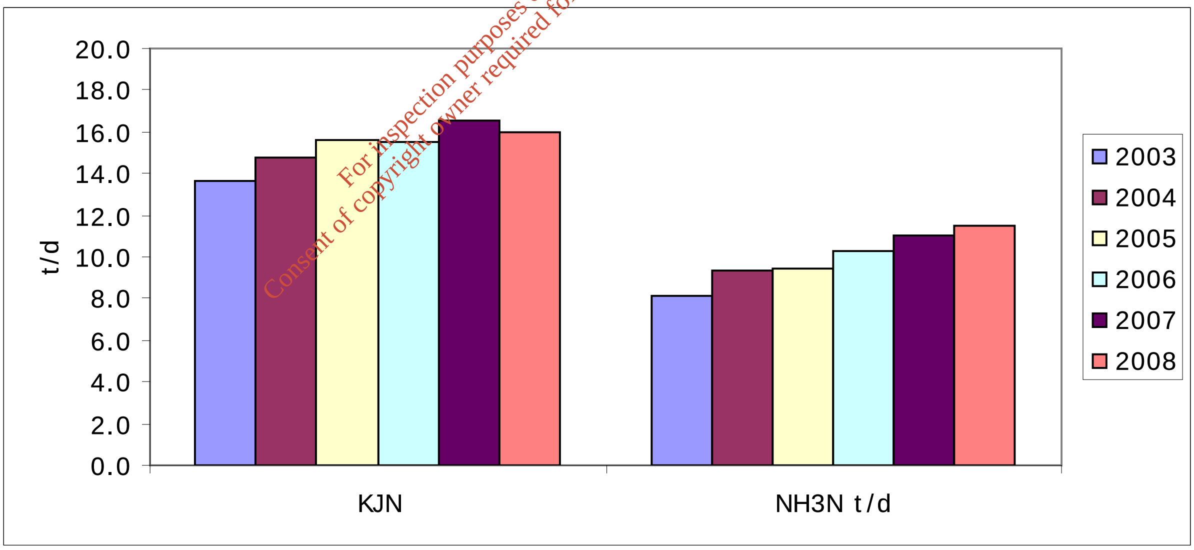



*Figure 3.3 Average Nitrogenous Load to the Ringsend WwTW 2003 to 2008*

**Note: Design load is Total Nitrogen whilst TKN is actually measured in the Influent*

The 2020 design Phosphorus Load for the Ringsend WwTW is 3.7 t/d (5.6 t/d 95 percentile). There is no dedicated phosphorus removal in operation at Ringsend. However, some phosphorus (i.e. c35%) will be removed as a result of sedimentation and biological P uptake and as part of the solids removal process. Table 3.7 shows current influent P concentrations.

*Table 3.7 Ringsend WwTW Influent Phosphorus concentrations*

| | Total P (mg/l) | Reactive P (mg/l) |
|---|---|---|
| 2005 | 6.5 | 3.5 |
| 2006 | 6.4 | 3.4 |
| 2007 | 5.6 | 3.5 |

## 3.1.3 Loading Variability

According to the Urban Wastewater Directive, Article 4.4, the (designated) load of a treatment plant expressed in PE "shall be calculated as the basis of the maximum weekly average load entering the plant during the year, excluding unusual situations such as those due to heavy rain".

Based on the above definition, the PE of the Ringsend WwTP for 2008 (to end August) was 3,697,696 PE and the variability over the past five years is summarised in Table 3.8.

*Table 3.8 Loading Variability (PE)*

| | 2004 | 2005 | 2006 | 2007 | 2008 |
|---|---|---|---|---|---|
| Average Annual PE | 1,980,405 | 1,965,830 | 1,955,033 | 1,691,486 | 1,790,678 |
| Maximum Weekly PE Load* | 2,624,265 | 2,553,775 | 3,111,220 | 2,602,621 | 3,697,696 |
| Ratio-Average to Maximum Load | 1.33 | 1.30 | 1.59 | 1.53 | 2.06 |
| Measured 95%ile PE | 2,713,793 | 2,704,603 | 3,251,191 | 2,337,972 | 3,027,917 |

*As per EPA Definition*



The difference between the average loads and the maximum weekly load being received at the plant is significant and has been increasing over time. In 2008 the maximum load was over 200% of the annual average and this load was sustained over an entire week. This increased loading exerts significant pressure on the WwTW in terms of maintaining effluent quality and processing increased loads of sludges on site. This variability needs to be considered carefully and factored into the design of the expanded works.

Table 3.8 includes the measured 95 percentile load over the period 2004 to 2008. The current design included for a 95 percentile load of 157.6 t/d, which equates to a PE of 2,626,666 PE using 60 g/c/d. This 95 percentile has been exceeded in all years except 2007 and the maximum weekly PE load exceeds the 95 percentile in 2006 and 2008.

## 3.1.4 Current Loading Breakdown

This section looks at the breakdown of the load being received at the Ringsend WwTW by analysis of the catchment, both domestic and non-domestic contributors. CSO census data was used to provide data for the domestic population and Local Authority trade license and IPPC license information was used to asses the load from the Industrial sector.

Other load contributions e.g. the commercial sector, are difficult to accurately assess due to the lack of legislation in place to provide complete monitoring and licensing of this sector (i.e. office blocks etc.). In addition, there are other variables specific to the Dublin region, such as high levels of commuters into the area and a high level of tourism that contribute to the uncertainties in this measurement technique. Various sources were used to provide additional information and references are included as footnotes to Table 3.9.

*Table 3.9 Current Load to Ringsend WwTW*

| | BOD kg/d | COD kg/d | TSS kg/d | PE |
|---|---|---|---|---|
| Measured [a] | 107,441 | 229,983 | 112,478 | 1,790,678 |
| Residential [b] | 63,979 | | 79,973 | 1,066,311 |
| Industrial [c] | | | | 233,853 |
| Commercial [d] | | | | 170,610 |
| Institutional [e] | | | | 7,672 |
| Tourism [f] | | | | 25,795 |
| Commuter [g] | | | | 36,913 |
| Tanker Discharges [h] | | 449 | 429 | 13,227 |
| Total Calculated | | | | 1,552,383 |

*a Based on average loads for 2008*
*b CSO 2006 census data for Dublin City and Greater suburbs (and some additional small population centres i.e. Ashbourne, Dunboyne and Clonee in County Meath; Saggart, Rathcoole, Newcastle and Baldonnel in South County Dublin)*
*c 2008 measured data from IPPC and Trade Licence discharges received from the four Local Authorities*
*d Estimated at 16% of the Domestic Load*
*e Based on 5,674 Hospital beds (HSE 2008)(1,998 Prison beds (Irish Prison Service Annual Report 2007) not included as it is assumed (from correspondence with CSO) that prisoners are accounted in Residential figures*
*f Tourism figure (15,795 PE) based on Failte Ireland Published Report 'Tourism Facts 2007'. Daily visitor figure (10,000 PE) based on Tourism Satellite Account (TSA) Project and the First Steps TSA report figures and a daily BOD load of 20g/visitor.*
*g Based on data from the Dublin Transportation office 2008 (Census data 2006) for people travelling into Dublin City & Suburbs from outside the administrative area. A BOD load of 20g/c was used.*
*h Based on data from Dublin City Council for 2008 tanker discharge volumes*



The calculated data presented above is almost 14 % different to that measured at the Treatment works. There are a number of possible explanations for this e.g. incorrect unit loads used (60g/c/d); underestimation of the industrial discharges; illegal dumping; underestimation of the contribution from commercial sources.

## 3.2 Projected Loading

The Ringsend WwTW was originally designed for a PE of 1.64 million and it was envisioned that it would ultimately be expanded to treat 2.15 million PE. A portion of the site, comprising 0.8 hectares, was set aside for the expansion. This ultimate design capacity was considered to include for secondary treatment and seasonal disinfection only.

There are various restrictions on the capacity that can be achieved on the current Ringsend site but if it is assumed that the planned North Dublin Plant will be in place by earliest 2020, then the design year for the upgrade should be at least 2025 (including for buffer period).

### 3.2.1 Residential Load Projection

The baseline for domestic populations is the 2006 Census data. The most recent publication from the CSO 'Regional Population Projections 2011-2016' (Dec 2008), indicate some variations in recent population trends and project the population for the Dublin Area using a number of different scenarios. These projections are based on future trends in fertility, mortality, migration (international & internal).

For the Dublin area, the following population projections have been provided in Table 3.10. Targets from the National Spatial Strategy are also included for comparison.

*Table 3.10 CSO Population Projections for Dublin (population in thousands) (ref: CSO Regional Population Projections 4th Dec 2008)*

| Scenario | 2006 | 2011 | 2016 | 2021 | 2026 | Annual Average Increase % |
|---|---|---|---|---|---|---|
| M2F1 Recent | 1,183 | 1,279 | 1,345 | 1,380 | 1,365 | 0.7 |
| M2F1 Traditional | 1,183 | 1,302 | 1,464 | 1,563 | 1,659 | 1.7 |
| M0F1 Recent | 1,183 | 1,178 | 1,164 | 1,132 | 1,080 | -0.5 |
| M0F1 Traditional | 1,183 | 1,199 | 1,246 | 1,298 | 1,343 | 0.6 |
| NSS Target* | 1,183 | | | 1,484 | | |

**The National Spatial Strategy: The DEHLG is responsible for the implementation of the National Spatial Strategy (NSS) which is aimed at promoting more balanced regional development and harnessing the potential of all regions.*



### 3.2.2 Industrial Load Projection

Data in Table 3.9 notes that the load from industrial discharges in 2008 was 233,853 PE. This equates to approximately 23% of that allocated or licensed. Given the current economic situation both nationally and internationally, it is likely that this industrial load will decrease further, in the short term at least. It is also policy within DCC for new and amended trade licence applications to reduce Industrial discharges to domestic strength.

Although the current strategy within the Local Authorities is to reduce the Licensed Industrial PE load to Ringsend, there is currently significantly more PE licensed than is actually used. It is prudent to look at the actual allocation and consider the total loadings if License holders increased their discharges. It is equally prudent to plan for the inclusion of future industrial development in the catchment.

A figure of 400,000 is included for Industrial PE loads for the design year 2025.

### 3.2.3 Commercial & Other Non-Domestic Loads

The other non-domestic loads to the Ringsend WwTW have proven difficult to quantify accurately and therefore the following formula has been used to estimate a load figure for total non-domestic sources (excluding Industrial):

Other non-Domestic Loads = (Total Measured Load – (Residential Load + Industrial Load))

It is reasonable to use the same growth rate projections for the non-domestic populations as for the domestic so these loads were projected forward to the design year 2025 using the growth rates discussed below.

### 3.2.4 Total PE Projections

Growth rates from the most recent CSO publication (Dec '08) were discussed in Section 3.2.1. All of the four annual growth rates discussed in this publication 1.7%, 0.7%, 0.6% and -0.5%, were used to project the PE for the Ringsend WwTW as shown in Figure 3.4. These growth rates were used to project Domestic and other Non-Domestic loads (excluding Industrial Load) forward to the design year 2025 from 2008 actual plant loading data. A figure of 400,000 PE is included for the contribution from Industrial sources.

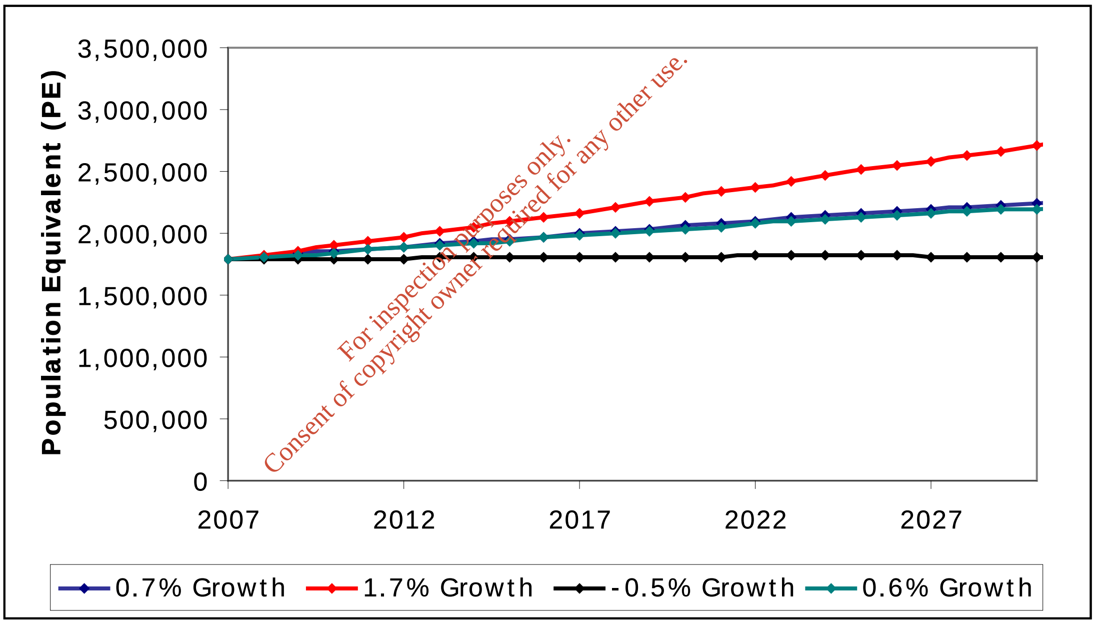





*Figure 3.4 Ringsend WwTW Future Growth Projections from 2008 data Load*

Using an annual growth rate of 1.7% the projected PE for the year 2025 is 2,505,720 PE and using the more conservative growth rate of 0.7% the 2025 PE is 2,162,600.

As discussed in Section 3.1.3, the Urban Wastewater Directive, Article 4.4, the (designated) load of a treatment plant expressed in PE "*shall be calculated as the basis of the maximum weekly average load entering the plant during the year, excluding unusual situations such as those due to heavy rain*".

Based on this definition the PE of the Ringsend Wastewater Treatment Plant for 2008 was 3,697,696 PE. If it is assumed that this unprecedented level of load does not return and take the average Peak week load from 2004 to 2007 (from Table 3.8) as 2,722,970 PE this gives a peak factor of 1.5 for the maximum week.

Figure 3.5 illustrates the projections in peak loading to the Ringsend WwTW using the four Growth rates discussed above. Using a Growth Rate of 1.7% the peak projected PE for the year 2025 is 3,758,580 PE and using the more conservative Growth Rate of 0.7% the 2025 PE is 3,243,901 (Figure 3.5). Consideration must be given for weekly peaks of this order in the design of the expansion.

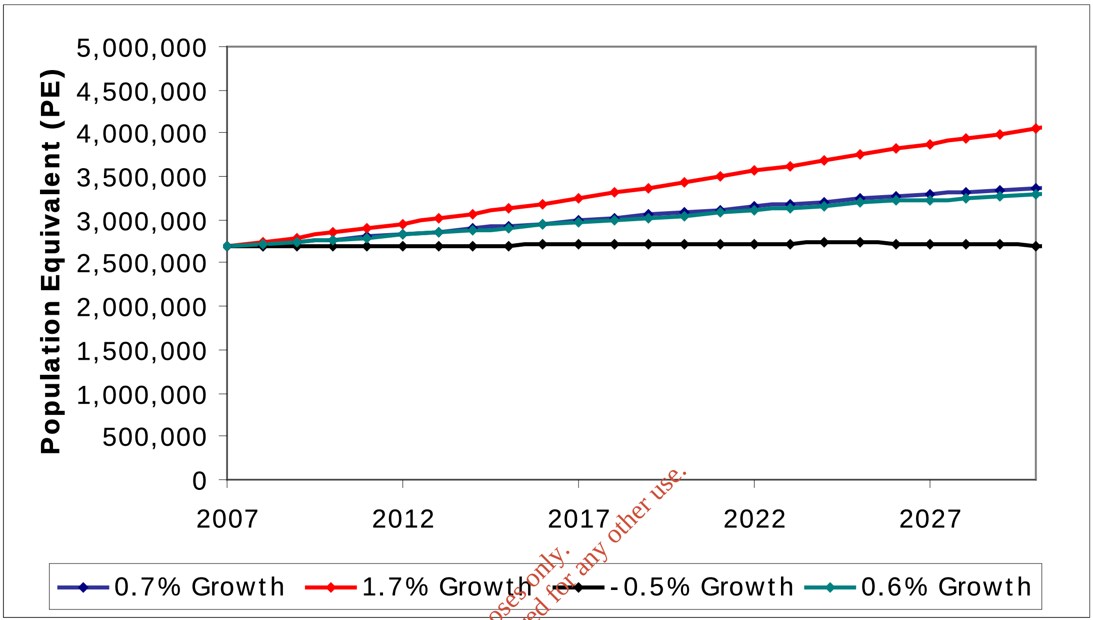



*Figure 3.5 Ringsend WwTW Future Growth Projections (peak) considering a Maximum weekly Load (Peak factor of 1.5)*



### 3.2.5 Proposed Design Load

Table 3.11 tabulates the Proposed Design Loads using the following design assumptions:

- Domestic and non-domestic (excluding industrial) growth rate of 0.7%;
- Projections forward from the 2008 measured influent WwTW load of 1,790,678 PE;
- An allocation of 400,000 PE for industrial load;
- A design year of 2025;
- A peak factor of 1.5; and
- The following unit loads*; 60 g BOD/c/d; 60 g TSS /c/d; 10 g TKN /c/d; 7 g AmmN /c/d; and TP 1.8 g/c/d.

*Table 3.11 Proposed Design Loads*

| | Average Design | Peak Design** |
|---|---|---|
| PE | 2.2 million | 3.3 million |
| Average Flow ($m^3$/d) | 504,000 | 756,000 |
| Full Flow to Treatment ($m^3$/s) | 13.8 | - |
| Peak Flow ($m^3$/s) | 23.5 | - |
| BOD (kg/d) | 132,000 | 198,000 |
| TSS (kg/d) | 132,000 | 198,000 |
| Ammonia (N) (kg/d) | 15,400 | 23,100 |
| Total Kjedahl Nitrogen (kg/d) | 22,000 | 33,000 |
| Total Phosphorus (kg/d) | 3,960 | 5,940 |

** Unit loads have been extrapolated from Ringsend WwTW Plant data 2003- 2008*
***Based on the maximum weekly load to the plant (or 1.5 times average)*

## 3.3 Conclusions

- The current load to the Ringsend WwTW is 107 t/d BOD or 1.79 million PE.
- The peak loading, defined as the maximum weekly load received at the plant, is also considered. The incoming load to Ringsend WwTW varies considerably and this is an extremely important issue to be factored into design of the plant expansion.
- Projections have been made on the WwTW influent data which is the best available measure of the current load. In relation to population growth, there are four official bases from which to choose, ranging from a negative growth rate of -0.5% per annum to 1.7% per annum. A growth rate of 0.7% has been chosen on the basis that the expanded works design should be robust, reliable, and provide adequate redundancy. A figure of 400,000 PE has been included for the contribution from the Industrial section.
- The 2025 design year proposed design is 2.2 million PE

# Section 4
# Wastewater Treatment Alternatives

This section examines alternatives to meet the new effluent requirements. It consists of introductory text about the history and operation of the SBRs, the basis of analysis (loadings, temperature, recycle loads, removal in primary clarifiers), analysis of the capacity of the existing SBRs, and review of treatment alternatives. A summary concludes the section.

## 4.1 Background

As noted previously, the Works were designed to produce effluent to meet standards for BOD, TSS, and ammonia nitrogen. After the contract was signed, the Liffey River Estuary was designated as Nutrient Sensitive Waters under the Urban Waste Water Treatment (UWWT) directive. Consequently, annual mean limits of 10 mg/L total nitrogen (TN) and 1 mg/L total phosphorus were set on effluent from the Works.



To achieve the more-stringent limits, larger pipes were installed to supply more air to the carbonaceous sequencing batch reactors (SBRs), and mixers and pumps for returning activated sludge were added to those SBRs. It was anticipated that MLSS concentrations in the carbonaceous reactors would be increased to 4,100 mg/L.

Design intent was to provide flexibility to operate the SBRs as true batch systems or with continuous flow. However, after a period of poor performance, operation was changed. With the new operation, feed is added during the decant cycle, to provide some de-nitrification where the influent mixes with the sludge layer. With this arrangement, the volume in the basin remains constant, and effluent from a basin is produced as influent enters the basin. This modification is called Constant Inflow, Constant Level (CICL) mode.

The Works has had several operating problems:

- The SBRs have not been able to support MLSS of 4,100 mg/L, and even at concentrations as low as 2,500 mg/L, effluent still sometimes fails to meet TSS limits.
- Because of wind impacts, the depth in the upper SBRs has been dropped to 5.9 m, from the 6.9-m depth available. This change decreases capacity of the Works by approximately 7 percent.
- Co-settling of surplus activated sludge in the primary clarifiers decreases their removal efficiency, passing on higher loadings to the SBRs.
- Occasional solids processing limitations cause solids inventories to exceed storage capacity, requiring retention of solids in the primary clarifiers and thereby hampering removal efficiency in that process.

- Individual SBR basins are frequently unavailable due to routine and unscheduled maintenance and periodic equipment replacement. While this is normal and expected, it was not accounted for in the design.

- When a single SBR basin is out of service, the three remaining basins in the "set of 4" are operated out of synchronisation, during which time effluent quality is degraded.

- The average daily flow to the Works is approaching the design average flow rate, resulting in a reduction of residence time in all wastewater processes as compared to the dry weather flow rate assumed in the mass balance.

## 4.2 Basis of Analysis

To analyse various alternatives, common bases have to be developed. This section develops and describes the common information.

### 4.2.1 Influent Concentrations

Averages of key parameters for influent concentrations are listed in Table 4.1.

*Table 4.1 Current Average Influent Concentrations*

| Constituent | Concentration (mg/L) |
|---|---|
| BOD[1] | 260.7 |
| COD | 520 |
| TSS | 255 |
| Total nitrogen | 40 |
| Total phosphorus | 5.0 |



*1 Urban Waste Water Treatment Directive 91/271/EEC specifies that BOD measurements be conducted with addition of a nitrification inhibitor. That practice is followed by laboratories conducting analyses on wastewater samples from Ringsend. In this report, the term "BOD" or "cBOD" refers to inhibited BOD.*

To determine an appropriate peaking factor for estimating plant capacity, monthly peaking factors were calculated for each month from January 2003 through December 2007. These peaking factors are the ratio of influent BOD load for a given month divided by the average BOD load for that year. The variation in BOD load is plotted in Figure 4.1. The highest monthly peaking factor of 1.23 is used by CDM in estimating capacity of the SBR basins.

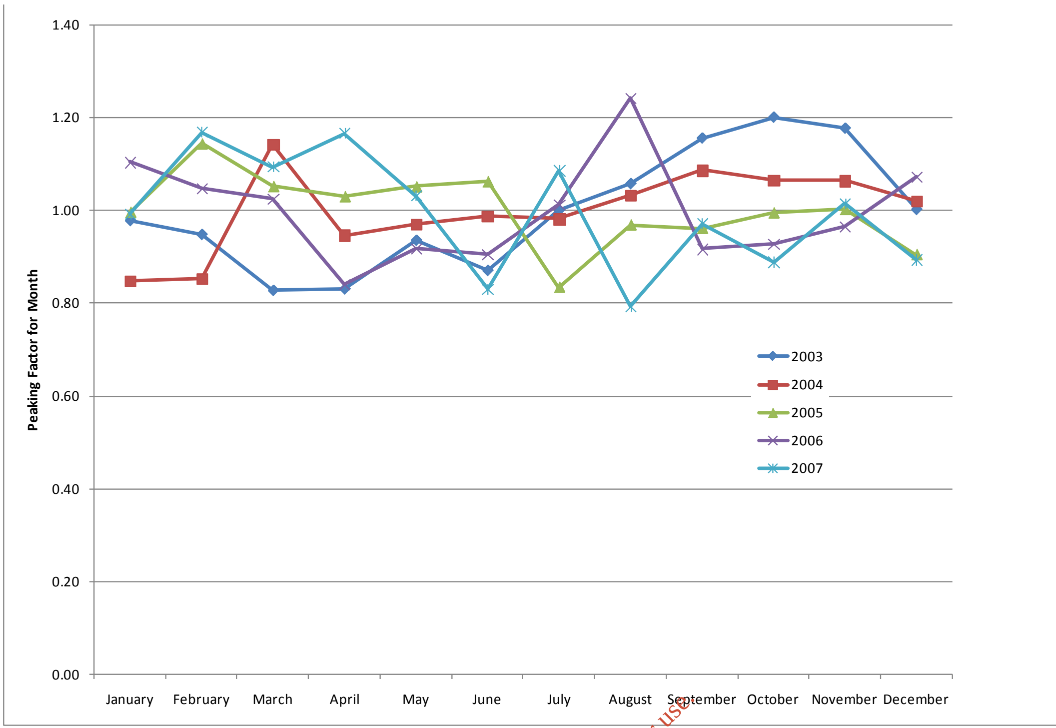



*Figure 4.1 Peaking Factors for BOD*



## 4.2.2 Wastewater Temperature

Wastewater temperature is an important factor in the rate of biological activity, especially as it relates to nitrification. Three years of influent wastewater temperatures were analysed and data in Table 4.2were extracted from that data base.

*Table 4.2 Wastewater Temperatures*

| Temperature, °C | Frequency Exceeded |
|---|---|
| 11 | 98.6% |
| 13 | 68.0% |
| 14.9 | 50.0% |
| 15 | 50.7% |
| 17 | 30.0% |

A wastewater temperature of 13°C has been selected for analysis. This temperature is exceeded 68% of the time. Design on this basis provides a moderate degree of conservatism, since compliance with the UWWT is based on the arithmetic mean.

## 4.2.3 Assumptions

In keeping with the original design, it is assumed that volatile suspended solids (VSS) constitute 82% of TSS and that the net growth yield coefficient remains 0.8 kg/kg cBOD5 applied. These numbers have been extracted from the original design basis of this plant.

The Works has recently begun collecting samples on the recycle streams from solids processing, but flow-proportioned samples are very limited at this point in time. Until a sufficient bank of flow-proportioned data has been collected, flows and loadings from recycle streams and efficiency of primary treatment can only be estimated from data collected elsewhere. Table 4.3 shows estimates prepared for Ringsend.

*Table 4.3 Estimated Recycle Stream and Removals in Primary Clarifiers*

| Parameter | Recycle Streams as Percent of Load in Influent | Percent Removals in Primary Clarifiers |
|---|---|---|
| Flow | 3 | |
| BOD | 3 | 30 |
| COD | 8 | 28 |
| TSS | 10 | 44 |
| TKN | 17 | 9 |
| Ammonia nitrogen | 3 | 0 |
| Total phosphorus | 7 | 33 |



As noted previously, effluent suspended solids do not consistently meet discharge requirement, even when MLSS is maintained at concentrations substantially less than the intended concentration of 4,100 mg/L. Wind effects are believed to be a major contributor to the failure to meet discharge requirements. For conservatism, calculations for this report are based on an MLSS of 3,100 mg/L. That concentration is now exceeded only 2% of the time.

If it is also assumed that all the SBRs will be operated at the design depth of 6.9 meters. To allow for operation at this depth, problems associated with wind will have to be resolved.

## 4.2.4 Flow and Mass Balance for Existing Facility

Figure 4.2 presents a block flow diagram for the existing works. It is a simplified version derived from the works' process and instrumentation diagrams. The circled numbers indicate streams for the associated material balance in Table 4.4. The mass balance reflects actual effluent concentrations of 19.1 mg/L TN and 5.0 mg/L total P over a three-year period. According to the mass balance, about 30% of the TKN in the primary effluent is removed via reduction to nitrogen gas.

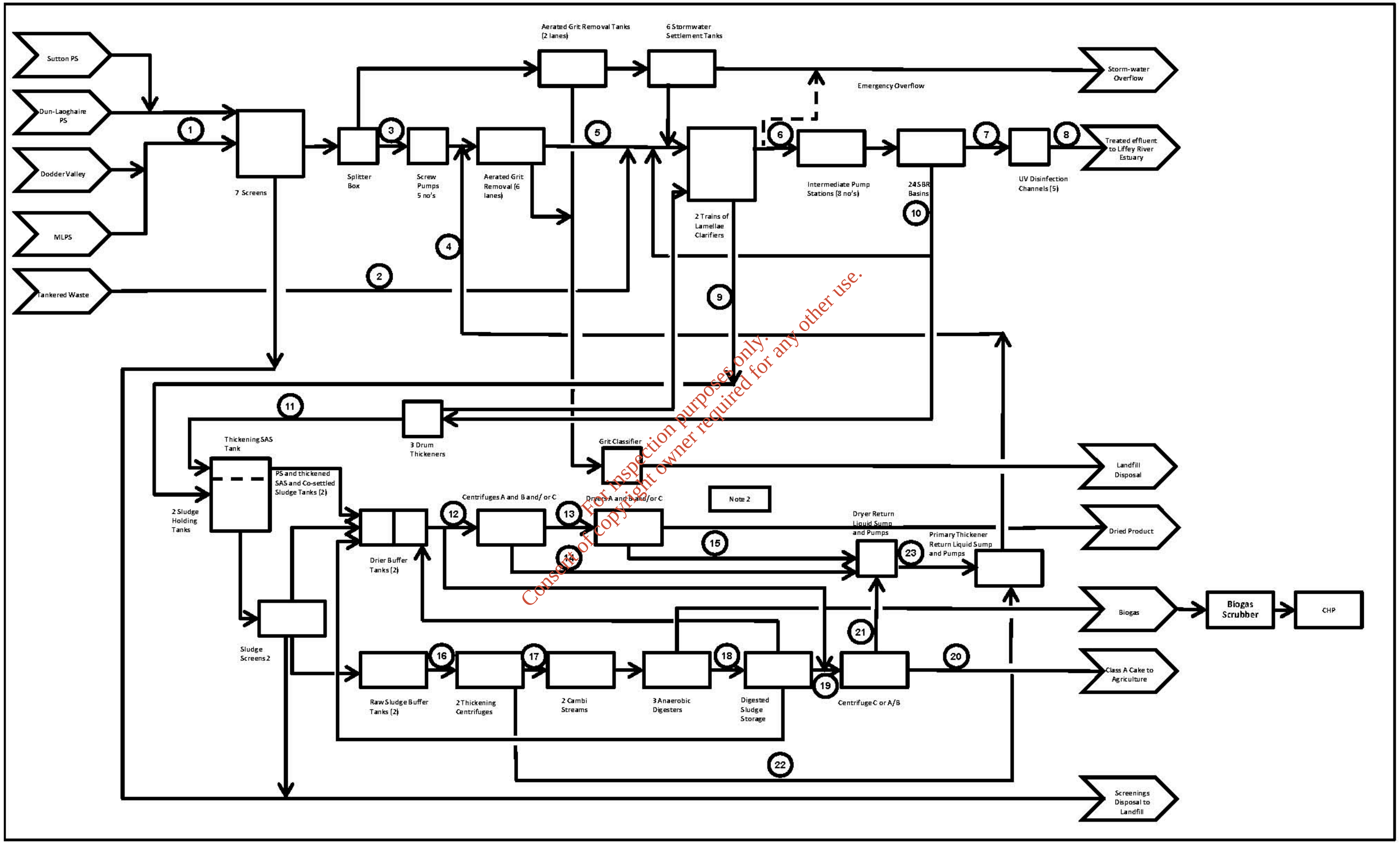





*Figure 4.2 Block Flow Diagram of Existing Works*

*Table 4.4 Existing Ringsend WwTW Mass Balance*

| Stream Number | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Parameters** | **Units** | **Combined Influent** | **Tankered Waste** | **Pumped Influent** | **Combined Recycle Streams** | **Degritter Effluent** | **Primary Treated Effluent** | **SBR Effluent** | **Final Effluent** | **Primary Solids** | **Surplus Activated Sludge** |
| Flow | $m^3$/day | 412,000 | 45 | 412,000 | 12,361 | 424,406 | 422,700 | 414,431 | 414,431 | 1,707 | 8,268 |
| Flow | $m^3$/s | 4.77 | 0.001 | 4.77 | 0.14 | 4.91 | 4.89 | 4.80 | 4.80 | 0.02 | 0.10 |
| COD | kg/day | 214,240 | 945 | 214,240 | 17,215 | 232,400 | 167,328 | 29,922 | 29,922 | - | - |
| COD | mg/L | 520 | 21,000 | 520 | 1,393 | 548 | 396 | 72 | 72 | - | - |
| cBOD | kg/day | 107,400 | 113 | 107,400 | 3,225 | 110,738 | 77,517 | 8,413 | 8,413 | - | - |
| cBOD | mg/L | 261 | 2,511 | 261 | 261 | 261 | 183 | 20.3 | 20.3 | - | - |
| TSS | kg/day | 105,060 | 716 | 105,060 | 10,578 | 116,354 | 65,158 | 14,505 | 14,505 | 51,196 | 62,013 |
| TSS | mg/L | 255 | 15,911 | 255 | 856 | 274 | 154 | 35 | 35 | 30,000 | 7,500 |
| VSS | kg/day | 86,149 | 587 | 86,149 | 8,674 | 95,410 | 53,429 | 11,604 | 11,604 | 41,980 | 49,611 |
| VSS | mg/L | 209 | 13,038 | 209 | 702 | 225 | 126 | 28 | 28 | 24,600 | 6,000 |
| Ammonia-N | kg/day | 10,300 | 4 | 10,300 | 309 | 10,613 | 10,613 | 1,451 | 1,451 | - | - |
| Ammonia-N | mg/L | 25.0 | 89.8 | 25.0 | 25.0 | 25.0 | 25.1 | 3.5 | 3.5 | - | - |
| TKN | kg/day | 16,480 | 4 | 16,480 | 2,802 | 19,286 | 17,551 | 3,440 | 3,440 | - | - |
| TKN | mg/L | 40.0 | 89.8 | 40.0 | 226.7 | 45.4 | 41.5 | 8.3 | 8.3 | - | - |
| Nitrate N + Nitrite N | kg/day | - | - | - | - | - | - | 4,476 | 5,576 | - | - |
| Nitrate N + Nitrite N | mg/L | - | - | - | - | - | - | 10.8 | 10.8 | - | - |
| Total N | kg/day | 16,480 | 4 | 16,480 | 2,802 | 19,286 | 17,551 | 7,916 | 7,916 | - | - |
| Total N | mg/L | 40.0 | 89.8 | 40.0 | 226.7 | 45.4 | 41.5 | 19.1 | 19.1 | - | - |
| Total P | kg/day | 2,060 | 1 | 2,060 | 144 | 2,205 | 1,477 | 2,072 | 2,072 | - | - |
| Total P | mg/L | 5.0 | 17.1 | 5.0 | 11.7 | 5.2 | 5.2 | 5.0 | 5.0 | - | - |

# 4.3 Capacity Analysis

This section examines the capacity of the existing facilities with respect to three options:

- Providing nitrification;
- Meeting the effluent requirement of 10 mg/L total nitrogen; and
- Providing treatment to meet BOD requirement without nitrification and denitrification.

Evaluation of the first two options is presented in one section.

## 4.3.1 Capacity of Existing Facilities to Nitrify and for Meeting Effluent Standards

This section estimates the capacity of the SBRs for nitrification and for nitrogen removal. Key assumptions are listed in Table 4.5.

*Table 4.5 Key Assumptions for Rating the SBR Basins*

| Parameter | Value |
|---|---|
| BOD peaking factor | 1.23 |
| Mixed liquor suspended solids | 3,100 mg/L |
| Design temperature | 13°C |
| Aerated solids retention time | 8.2 days |
| Total solids retention time[1] | 16.4 days |
| Decrease in capacity with nitrogen removal | 35% |

*1 The SBRs are in aeration mode 50% of the time.*



The mixed liquor suspended solids concentration (MLSS) of 3,100 mg/L is recognized as an upper bound given the history of the SBRs to meet the TSS standard. Effluent TSS polishing would be required. This could be accomplished by screens or filters specifically designed for fine solids removal, or by denitrification systems that use a granular media that will trap solids. Since denitrification is required, the denitrification filters would provide TSS polishing.

The aerobic solids retention time (SRT) is based on kinetic factors from the publication *Methods for Wastewater Characterization in Activated Sludge Modeling,* which was published by the Water Environment Research Foundation in 2003. For a temperature of 13°C, the SRT at washout is about 3.28 days. Applying a safety factor of 2.5 and accounting for aeration during half of a cycle provides a total SRT of 16.4 days. With SBRs operated for nitrogen removal, aeration time per cycle would be decreased to allow for nitrogen removal. It was estimated that the decrease in capacity with nitrogen removal would be about 35%.

Table 4.6 shows the logic for estimating the capacity of the SBRs. The table shows that the capacity for nitrification with all basins in operation is 1.49 million PE; the capacity for nitrogen removal is 0.97 million PE.

*Table 4.6 Calculations for Estimating Capacity of SBRs for Nitrification*

| Parameter | Value |
|---|---|
| Volume of each SBR ($m^3$) | 13,993 |
| Number in service | 24 |
| Total volume ($m^3$) | 335,837 |
| MLSS (mg/L) | 3,100 |
| Solids in SBRs (kg) | 1,041,094 |
| SRT (days) | 16.4 |
| Solids produced (kg/d) | 63,481 |
| Net yield (kg TSS produced/kg BOD applied) | 0.8 |
| BOD applied to SBRs (kg/d) | 79,352 |
| Removal of BOD in primary treatment | 30% |
| BOD to primary treatment (kg/d) | 113,360 |
| BOD in sidestreams, as fraction of influent load | 3% |
| BOD in influent at maximum month (kg/d) | 110,058 |
| Peaking factor | 1.23 |
| Average BOD in influent (kg/d) | 89,478 |
| BOD/PE (g/day) | 60 |
| Capacity for nitrification (million PE) | 1.49 |
| Decrease in capacity from nitrogen removal | 35% |
| Capacity for nitrogen removal (million PE) | 0.97 |



### 4.3.2 Capacity of Existing Facilities for BOD Removal

Preliminary analyses suggest that discharging effluent through an outfall extending into Dublin Bay might be economical compared to continued discharge into the existing outfall. The long outfall would require the plant to meet standards for BOD, while the existing outfall would require nitrogen removal. This report estimates the capacity of the existing plant to remove BOD. The capacity of the SBRs to meet standards for BOD exceeds capacity for nitrification, this analysis also reviews hydraulic capacity and aeration capacity.

#### 4.3.2.1 Hydraulics

The hydraulic analysis in this report is based on ABA's hydraulic profile (January 22, 2008) and on the Volumes 2 and 3 (both October 2004) of the Operation and Maintenance Manual. Current design for forward flow to treatment (FFT) is 11.1 $m^3/s$ and the ultimate design requirement is 13.8 $m^3/s$.

The inlet screw-pump station, the aerated grit channels, primary settling tanks, and intermediate pump station have been designed for a flow of 13.8 $m^3/s$. The grit channels and primary settling tanks would not require modification, but the inlet and the intermediate pump stations would require modification. The inlet station now has five screws (one standby) for 11.1 $m^3/s$ forward flow. A sixth screw would add enough capacity for future flows.

The intermediate pump station has four low-lift and four high-lift pumps, including a standby each for low lift and for high lift (i.e. 3 duty +1 standby), with a firm pumping capacity of 11.28 $m^3/s$. There is space for one additional pump of each type, potentially increasing the firm pumping capacity in excess of 13.8 $m^3/s$..

The SBRs are sized hydraulically for 11.26 $m^3/s$, including return flows. Return flows are only about 1% of total. According to notes on the hydraulic profile, one aeration tank is assumed to be out of service. The hydraulic capacity with all tanks in service is 13.5 $m^3/s$, close to the requirement of 13.8 $m^3/s$.

The UV plant is sized for 13.8 $m^3/s$.

#### 4.3.2.2 Capacity of SBR Basins for Treatment

The major factors affecting capacity of the SBR basins themselves in terms of loads are the fraction of each cycle that is aerated, the MLSS that can be sustained, and the SRT required to meet the effluent standard.

With the current operation of the SBRs, normal operation consists of four-hour cycles, of which one hour is for settling and one hour is for decanting. With this cycle structure, the SBRs could be aerated half the time. Other cycle times are also possible, but are used less frequently.



For plants whose effluent requirements are based on BOD and TSS, kinetic relationships suggest that soluble BOD can be decreased to the order of 5 mg/L or less with SRT of about 1 day. However, operation at that SRT produces high effluent suspended solids because of pin-point floc. Operation of conventional activated-sludge plant at SRT exceeding about 2 days has been found to meet effluent standards for both BOD and TSS. As will be shown later, the SBR basins themselves are not the bottleneck setting the capacity of the Ringsend plant, and a generous allowance of 4-day aerated SRT can be applied. Since the SBRs could be aerated half of the time (with the remaining time consisting of settling and decanting), the total SRT required would be eight days.[1]

Lowering the MLSS to 2,200 mg/L would improve effluent TSS quality and achieve the design capacity of 2.2 million PE with all six trains in operation. The ultimate volumetric capacity of the SBR basins, at a MLSS of 3,100 mg/L , is about 3 million PE with all six trains in operation and 2.5 million PE firm capacity with five of six trains in operation. As previously noted, operation at such a high MLSS would require effluent TSS polishing. If the Works were to be extended beyond the design year capacity and no additional tanksage is proviced, MLSS would need to be increased and effluent polishing alternatives would need to be considered.

---

[1] Page 15 of 25 of Section 4.1.1 of ABA's proposal (Table 2 in Section 4.1.1.8.7) states that total SRT is 8 days and aerobic SRT is 4 days, both for carbonaceous units.

### 4.3.2.3 Capacity of Aeration System

Depending on operation of an activated-sludge system, oxygen can be required to remove BOD only or to remove BOD plus provide nitrification. Denitrification, when provided, decreases oxygen required. Actual oxygen required (AOR) was calculated for BOD removal and nitrification, and a credit was taken for oxygen saved by denitrification.

Loads from the mass balance for current average conditions provide the basis for estimating oxygen requirement. Influent for the current mass balance was equal to a load of 1.79 million population equivalents (PE). This exercise was conducted to estimate oxygen requirement per population equivalent. Oxygen required is shown in Table 4.7.

*Table 4.7 Summary of Oxygen Required for Average Current Operation*

| Function | In kg/day | In kg/day/million PE |
|---|---|---|
| BOD removal | 99,231 | 55,436 |
| Nitrification | 37,999 | 21,229 |
| Denitrification (credit) | (20,830) | (11,637) |
| Total | 116,400 | 65,028 |



For the system envisioned, only BOD removal will be required. Still, though, at high temperatures, nitrification can be difficult to control, and it is wise to make some allowance for some nitrification. In the discussion below, three conditions are examined, with the oxygen consumption in Table 4.7 as the basis. The conditions are BOD removal only, BOD removal and nitrification, and BOD removal and nitrification with credit for nitrification. For design, the oxygen requirements are used to calculate related air-flow requirements. Two sets of assumptions to estimate air flow were used. One set applies the normal design criteria used by CDM; the other set applies criteria used by ABA during design. Table 4.8 shows the difference in assumptions.

In calculations for this report, as in calculations by ABA, standard conditions for air flow are 20°C and 36% relative humidity.

*Table 4.8 Comparison of Assumptions for Calculations Leading to Design Air Flow*

| **Factor** | In CDM Calculations | In ABA Calculations | Comment |
|---|---|---|---|
| Peaking factor from average to design | 1.5 | 1.28 | CDM's factor is for maximum day and is based on plant data |
| Dissolved-oxygen concentration (mg/L) | 2 | 1 | CDM's standard is to provide DO of 2 mg/L for maximum day. |
| Temperature (°C) | 19 | 20 | Lower temperature provides lower air flow requirement. Minimal effect, however. |
| α | 0.55 | 0.5 | Higher alpha provides lower air-flow requirement. |
| β | 0.95 | 0.95 | This value is commonly used. |
| θ | 1.024 | 1.024 | This is a standard value. |
| Water depth (m) | 6.9 | 6.2 | 6.9 m is level with SBRs full and operated in CICL mode. ABA used average depth of water through cycle at 2020. |



Equipment for aeration includes blowers, diffusers, and air piping. Limitations from each of these units are reviewed.

***Blowers***
The nine centrifugal blowers and have capacity of 19,000 $m^3$/hr each. For this analysis, as in plant design, it is assumed that one blower will serve as standby (i.e. 8 duty +1 standby).

***Diffusers***
The SBRs have a total of 107,712 diffusers, each with an effective area of 380 $cm^2$. ABA's calculations were based on a flow of 1.8 $Nm^3$/hr each. Diffuser effective area is 8.4% of the floor area.

***Air Piping***
Air piping was increased in size during construction to provide for a flow of 21,850 $m^3$/hr per blower, 15% more than in ABA's original proposal.

***Summary and Conclusions for Air System***
Table 4.9 summarises the results of calculations for oxygen transfer.

*Table 4.9 Summary of Estimated Capacities (in millions of PE)*

| Oxygen Use | Blowers | | Diffusers | | Air piping | |
|---|---|---|---|---|---|---|
| | CDM Criteria | ABA Criteria | CDM Criteria | ABA Criteria | CDM Criteria | ABA Criteria |
| For BOD removal only | 1.75 | 2.08 | 2.23 | 2.65 | 2.01 | 2.39 |
| With nitrification | 1.26 | 1.50 | 1.61 | 1.92 | 1.45 | 1.73 |
| Including denitrification credit | 1.49 | 1.77 | 1.90 | 2.26 | 1.71 | 2.04 |

For both sets of criteria, capacity is limited by the blowers, and, after the blowers, by air piping then by the diffusers. The estimates following CDM criteria are lower than estimates using ABA criteria. Both sets of criteria show that capacity of blowers falls short of future requirements (2.2 million PE) for all conditions. Indeed, aeration capacity has already been insufficient at times. Diffusers might have to be added, depending on the ability to avoid nitrification. Air piping might not have to be increased, if detailed analysis shows that the higher velocities required can be accommodated.

### 4.3.3 Summary of Capacity Analysis

Table 4.10 summarises the results of the capacity analysis. The SBRs are adequate to meet requirements for BOD removal. SBRs operated to provide effluent with less than 10 mg/L or to nitrify would have capacity inadequate to treat flow from 2.2 million PE. Additional capacity would be required.



*Table 4.10 Capacity of SBRs (in million PE) for Various Levels of Treatment*

| Level of Treatment | Existing Capacity | Additional Capacity Required |
|---|---|---|
| 10-mg/L total nitrogen | 0.97 | 1.13 |
| Nitrification | 1.49 | 0.71 |
| BOD removal | >2.2 | None |

## 4.4 Treatment Alternatives

Unless the long-outfall option, which would not require removal of nitrogen or phosphorus, is implemented, facilities will have to be added to the works. Some options include treatment to decrease loads to secondary treatment, treatment of storm flows to make most of the area at the storm storage tanks available for other purposes, and treatment to remove nitrogen and phosphorus.

### 4.4.1 Methods for Decreasing Loads to Secondary Treatment

The capacity of the SBRs could be increased by decreasing loads to secondary treatment. The means for doing so include improving efficiency of primary treatment and treating sidestreams. Efficiency of primary treatment can be improved by adding chemicals ahead of the primary settling tanks. Sidestreams can be treated by several methods. This section describes chemical enhancement for primary treatment and alternatives for treating sidestreams.

#### 4.4.1.1 Chemically Enhanced Primary Treatment (CEPT)

Many treatment plants around the world add chemicals ahead of primary treatment, in order to improve removal of suspended solids and affiliated contaminants. Frequently, ferric chloride is added, but, since iron interferes with disinfection by ultraviolet light, an aluminum salt would be preferable at Ringsend. Suitable aluminum salts include alum and polyaluminum chloride. Annual cost for alum would be €2 million.

CEPT removes more BOD and suspended solids than conventional primary treatment, CEPT also decreases required downstream facilities and decreases production of surplus activated sludge. The increased production of primary sludge must be accommodated, however. Table 4.11 shows the effect of CEPT on sludge production.

*Table 4.11 Summary of Sludge Production with and without CEPT*

| Type of primary treatment | Sludge production (kg/day) | | | |
|---|---|---|---|---|
| | Primary sludge | SAS | Total | Difference |
| With CEPT | 90,900 | 48,700 | 139,600 | 26,400 |
| Plain sedimentation | 51,200 | 62,000 | 113,200 | |



At a cost of €140/tonne of dry solids produced, processing and disposing the extra 26.4 tonnes per day of sludge would cost €1,350,000/year.

#### 4.4.1.2 Sidestream Treatment

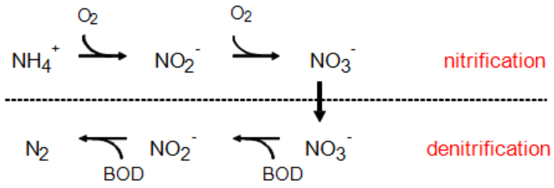
Figure 4.3 illustrates three alternatives for removing nitrogen from sidestreams.

Section a presents conventional steps, which consist of oxidation of ammonia nitrogen to nitrite and then to nitrate. Nitrate is then reduced to nitrite and then to nitrogen gas. Reduction of nitrogen requires biodegradable carbon. The carbon can already be BOD present in wastewater, or can be added.

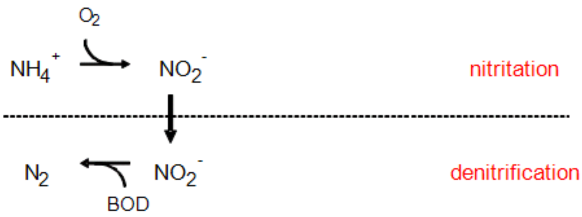
Section b illustrates the SHARON (Single High-Activity Ammonia Removal over Nitrite). In principle, this process takes advantage of the high temperature of the recycle streams which significantly enhances the rate of oxidation of ammonia over that of nitrite. This results in accumulation of nitrite, which can then be denitrified by adding methanol. By suppressing the oxidation of ammonia all the way to nitrate significant amounts of aeration air and methanol can be saved.

Section c illustrates the ANAMMOX (Anaerobic Ammonium Oxidation) and DEMON (De-Ammoniafication) processes. In these systems, ammonia rather than biodegradable carbon is used to biologically reduce nitrogen. The goal of operation is to oxidise half of the ammonia to nitrite and to use the remainder of the ammonia to reduce nitrite to nitrogen gas. These processes require less oxygen than the SHARON process and require no biodegradable carbon.

a. Conventional Nitrification/Denitrification




b. SHARON Process




c. ANAMMOX and DEMON Processes

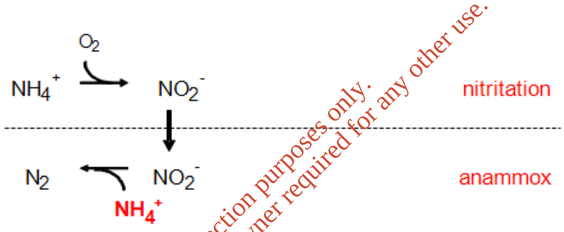



*Figure 4.3 Alternative Processes to Remove Nitrogen from Sidestreams*



Capital, operating, and present-worth costs for sidestream treatment are shown in Table 4.12

*Table 4.12 Costs for Sidestream Treatment*

| **Cost Item** | **Cost** |
|---|---|
| Capital cost | € 3,500,000 |
| Contingency | € 1,200,000 |
| Total capital cost | € 4,700,000 |
| Annual operating cost | € 150,000 |
| Total present worth | € 6,700,000 |

### 4.4.1.3 Decrease in Loads and Effect on Capacity of SBRs

Table 4.13 shows the assumptions regarding the effects of CEPT and sidestream treatment, singly and combined. The table shows that sidestream treatment has little effect on most constituents, but decreases total Kjeldahl nitrogen by 85% (from 17% of influent load to 2.6%). CEPT increases removal of all species. Applied individually or together, CEPT and sidestream treatment can substantially decrease concentrations in primary effluent.

*Table 4.13 Effects of CEPT and Sidestream Treatment on Concentrations in Primary Effluent*

| Parameter | BOD | COD | TSS | NH3-N | TKN | TP | Flow |
|---|---|---|---|---|---|---|---|
| Sidestream returns (% of influent) | | | | | | | |
| Without sidestream treatment | 3 | 8 | 10 | 3 | 17 | 7 | 3 |
| With sidestream treatment | 3 | 8 | 10 | 1.5 | 2.55 | 7 | 3 |
| Removals in primary treatment (%) | | | | | | | |
| Conventional primary treatment | 30 | 28 | 44 | 0 | 9 | 33 | |
| With CEPT | 45 | 40 | 70 | 0 | 11 | 60 | |
| Concentrations in primary effluent (mg/L) | | | | | | | |
| With conventional treatment | 183 | 393 | 153 | 25 | 41 | 3.5 | |
| With sidestream treatment | 183 | 393 | 153 | 25 | 36 | 3.5 | |
| With CEPT | 144 | 327 | 82 | 25 | 40 | 2.1 | |
| With sidestream treatment and CEPT | 144 | 327 | 82 | 25 | 35 | 2.1 | |

Due to changes in concentrations of primary effluent, application of CEPT and of sidestream treatment would also affect the capacity of the SBRs for nitrification and denitrification, and affect sizing requirements for additional facilities needed. Table 4.14 summarises the changes.

*Table 4.14 Effects of CEPT and Sidestream Treatment on Capacity of SBRs*

| Capacity Component | With Conventional Primary Treatment | With CEPT | With Sidestream Treatment | With Both |
|---|---|---|---|---|
| Capacity (million PE) | | | | |
| Of nitrifying SBRs | 1.49 | 1.90 | 1.49 | 1.90 |
| Of nitrogen-removal SBRs | 0.97 | 1.23 | 0.97 | 1.23 |
| Capacity ($m^3$/day) | | | | |
| Of nitrifying SBRs | 343,249 | 436,862 | 343,249 | 436,862 |
| Of nitrogen-removal SBRs | 223,112 | 283,960 | 223,112 | 283,960 |
| Extra capacity needed (million PE) | | | | |
| With nitrifying SBRs | 0.71 | 0.30 | 0.71 | 0.30 |
| With nitrogen-removal SBRs | 1.23 | 0.97 | 1.23 | 0.97 |
| Extra capacity needed ($m^3$/day) | | | | |
| With nitrifying SBRs | 163,120 | 69,507 | 163,120 | 69,507 |
| With nitrogen-removal SBRs | 283,257 | 222,409 | 283,257 | 222,409 |

### 4.4.2 Compact System to Treat Storm Flows (Ballasted Flocculation)

Ballasted flocculation could be used to treat storm flows that are now stored in the storm tanks across Pigeon House Road from the main plant. With ballasted flocculation, the storm tanks could eliminated, thus making a large area available for other uses. Ballasted flocculation would be used to treat flows exceeding 13.8 m3/s. Capacity of the process would be about 10 m3/s.

The area required can be decreased to a small fraction of the area for conventional primary treatment by employing ballasted flocculation. One ballasted-flocculation process adds a coagulant (usually ferric chloride, but sometimes alum) and "microsand" (grain size from 0.075 mm to 0.3 mm in diameter) to screened, degritted wastewater. The mixture is flocculated and then settled in plate settlers. The sludge is passed through a cyclone, where the microsand is recovered. See Figure 4.4.

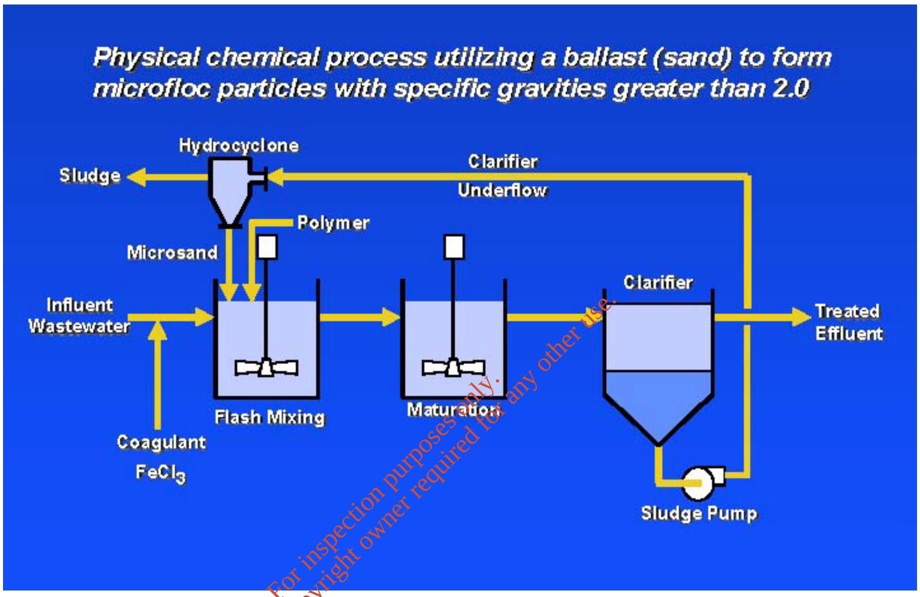





*Figure 4.4 Illustration of Ballasted Flocculation*

Chemical requirements are high (25 to 35 mg/L as ferric ion), but removals are outstanding. BOD removal is about 60 to 70% and TSS removal is about 85 to 90%. The area requirement for ballasted treatment is only about one tenth of the area required for conventional primary treatment.

Operating costs are very high because of the chemical dosage required. So, ballasted flocculation is appropriate only for special cases. One case is for treatment of high storm flows, where the units are used only occasionally. Another case is where the absolute minimum footprint has to be obtained. Ringsend meets both conditions.

The Achère wastewater treatment plant in Paris, France (23 $m^3/s$) uses ballasted flocculation. CDM has designed several ballasted-flocculation facilities. The largest of these is for Fort Worth, Texas, USA which has a capacity of 4.8 $m^3/s$.

Overall, the ballasted-flocculation facility would have a footprint about 30 m by 60 m. Space about 20 m by 50 m would be needed for chemical and sludge handling and for chemical storage.

The ballasted-flocculation facilities could be constructed at the west end of the storm tanks. See Figure 4.5, which illustrates the arrangement of the process units and the area require on site. These facilities would replace the storm tanks. However, during construction, arrangements would be required to keep the remaining storm tanks in operation.

Costs for ballasted flocculation are shown in Table 4.15. Annual operating costs are based on average flow of 16,000 $m^3$/day. That flow rate is based on operation from August 1, 2003 through September 15, 2008, when total flow to storm tanks was 30,000,000 $m^3$.

*Table 4.15 Costs for Ballasted Flocculation*

| Cost Item | Cost |
|---|---|
| Capital cost | € 30,500,000 |
| Contingency | € 10,700,000 |
| Total capital cost | € 41,200,000 |
| Annual operating cost | € 528,000 |
| Total present worth | € 48,100,000 |



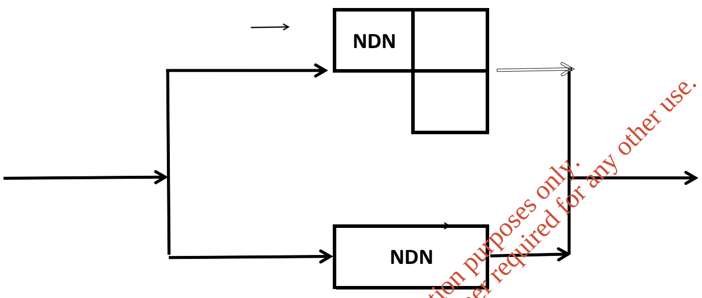

*Figure 4.5 Siting for Ballasted Flocculation*

### 4.4.3 Treatment Alternatives with Nitrogen Removal

This section describes and evaluates alternatives for treatment processes suitable for meeting effluent requirements for total nitrogen.

In all of the alternatives, SBRs will be used in some fashion. Figure 4.6 shows two arrangements. In the figure, NDN stands for nitrification and denitrification, N stands for nitrification, DN stands for denitrification, and C stands for carbon oxidation. Section a of the figure show SBRs operated to meet the 10 mg/L annual limit for total nitrogen. Flows in excess of the capacity of the SBRs would be treated in separate units. Section b of the figure shows SBRs operated to nitrify. Nitrate in the SBR effluent would be treated to biologically reduce the nitrogen to nitrogen gas. As in Section a, flow in excess of the capacity of the SBRs would be treated in separate units.

a. With SBRs Meeting Effluent TN Requirements




b. With SBRs Providing Nitrification

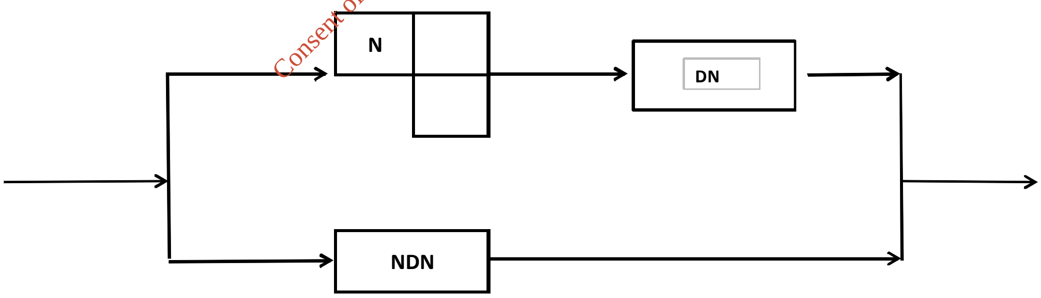



*Figure 4.6 Alternative Arrangements with SBRs*



#### 4.4.3.1 Denitrification Filters

SBRs operated in the nitrification mode could be followed by denitrification filters. Methanol, or another readily biodegradable carbon source, would be added upstream of the denitrification filters. In addition to denitrifying, the filters would capture effluent solids and return them to the primary clarifiers during the backwash cycle. The addition of a small amount of metal salts can precipitate phosphorus, which is then trapped in the media along with TSS.

For this analysis, filters were used with media 2.4 m deep. Loading limitation would be 1 kg nitrogen/day/$m^3$ and flow limitation would be 8 $m^3$/day/$m^2$. Number of filters includes allowance for 10% of the units to be out of service. Due to the fact that the requirement is an annual average, peaking factor was not included in calculations. Effluent target of 8 mg/L of total nitrogen was used (instead of the 10-mg/L requirement), to provide some conservatism.

Table 4.16 summarises the results of the evaluation of denitrification filters. The millions of population equivalents served by the filters are equal to the capacity of the SBRs for nitrification. The difference between the design population equivalents and the SBR capacity has to be treated elsewhere. With two stories of filters, the site can accommodate 40 filters. For this analysis, each filter has an area of 107 $m^2$. Two criteria were used for determining the area, loading and velocity. The loading rate was 1 kg/$m^3$/day, and the velocity was 8 m/hr.

*Table 4.16 Summary for Denitrification Filters*

| Parameter | With Conventional Primary Treatment | With CEPT | With Sidestream Treatment | With Both |
|---|---|---|---|---|
| Millions of PE served | 1.49 | 1.90 | 1.49 | 1.90 |
| Millions of PE to other treatment | 0.71 | 0.30 | 0.71 | 0.30 |
| Total nitrogen in primary effluent (mg/L) | 19.1 | 20.6 | 14.0 | 15.6 |
| Maximum-month load to be reduced (kg/day) | 3,924 | 5.651 | 2,121 | 3,401 |
| Number of filters | 18 | 26 | 10 | 16 |
| Cost item | | | | |
| Capital cost | €18,300,000 | €22,900,000 | €12,900,000 | €17,100,000 |
| Contingency | €6,400,000 | €8,000,000 | €4,500,000 | €6,000,000 |
| Total capital cost | €24,700,000 | €30,900,000 | €17,400,000 | €23,100,000 |
| Annual operating cost | €1,700,000 | €2,500,000 | €1,000,000 | €1,500,000 |
| Total present worth | €47,000,000 | €63,000,000 | €30,000,000 | €43,000,000 |



In Table 4.16, flows to the denitrification filters are equal to the capacity of the SBRs for the four options for primary and sidestream treatment. The concentrations of total nitrogen to be reduced depend on the concentration of nitrogen in the primary effluent. For the four options, nitrogen loading was the critical parameter in determining the number of filters.

### 4.4.3.2 Conventional Activated-Sludge

Conventional activated sludge, as distinct from SBRs, consists of aeration tanks and secondary clarifiers. At Ringsend, activated sludge could be implemented in two ways. One way would be to add secondary clarifiers and to convert the SBRs to aeration tanks. The other way would be as stand-alone activated sludge plant with new aeration tanks and secondary clarifiers.

The Modified Ludzack Ettinger process was examined for this purpose. The process is commonly used in the United States to meet total nitrogen requirements in the range of 6 to 10 mg/L. The MLE process incorporates an anoxic zone ahead of an aerobic zone, with nitrified mixed liquor recycled from the end of the aerobic zone to the beginning of the anoxic zone. Figure 4.7 is a schematic of the MLE process. Activated-sludge facilities could logically be constructed in two locations. One location is on the existing site in the open space south of the primary clarifiers and blowers. The other location is at the storm basins. The locations are shown in Figure 4.8.

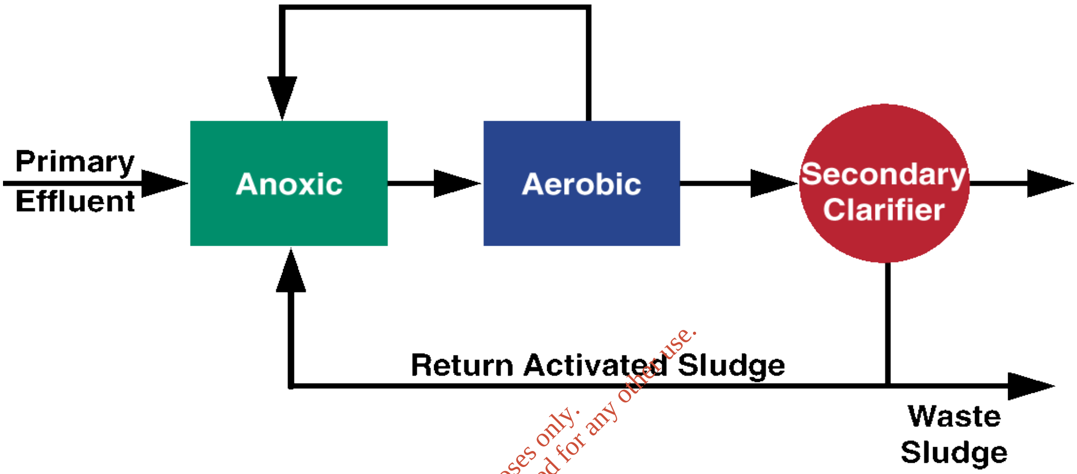



*Figure 4.7 Modified Ludzack Ettinger Process*



The area near the primary clarifiers and the blowers consists of about 7,934 m2, in two roughly rectangular areas of 6,524 m2 and 1,410 m2. This analysis examines the larger area (6,524 m2).

The area at the storm basins has 21,259 $m^2$ available. The area would only be available if storm flows in excess of the flow-to-works capacity are treated before discharge. Currently, these excess flows are stored in the storm basins and returned to the plant after flows recede. One option for treating storm flows is ballasted flocculation, which could be placed at the west end of the storm tanks.

Roughly a fifth of the area would be required for ballasted flocculation, leaving about 17,000 $m^2$ for activated sludge.

Estimated capacities for activated sludge are listed in Table 4.17. The capacity provided by stand-alone activated-sludge systems is too small, and stand-alone systems are not further examined.

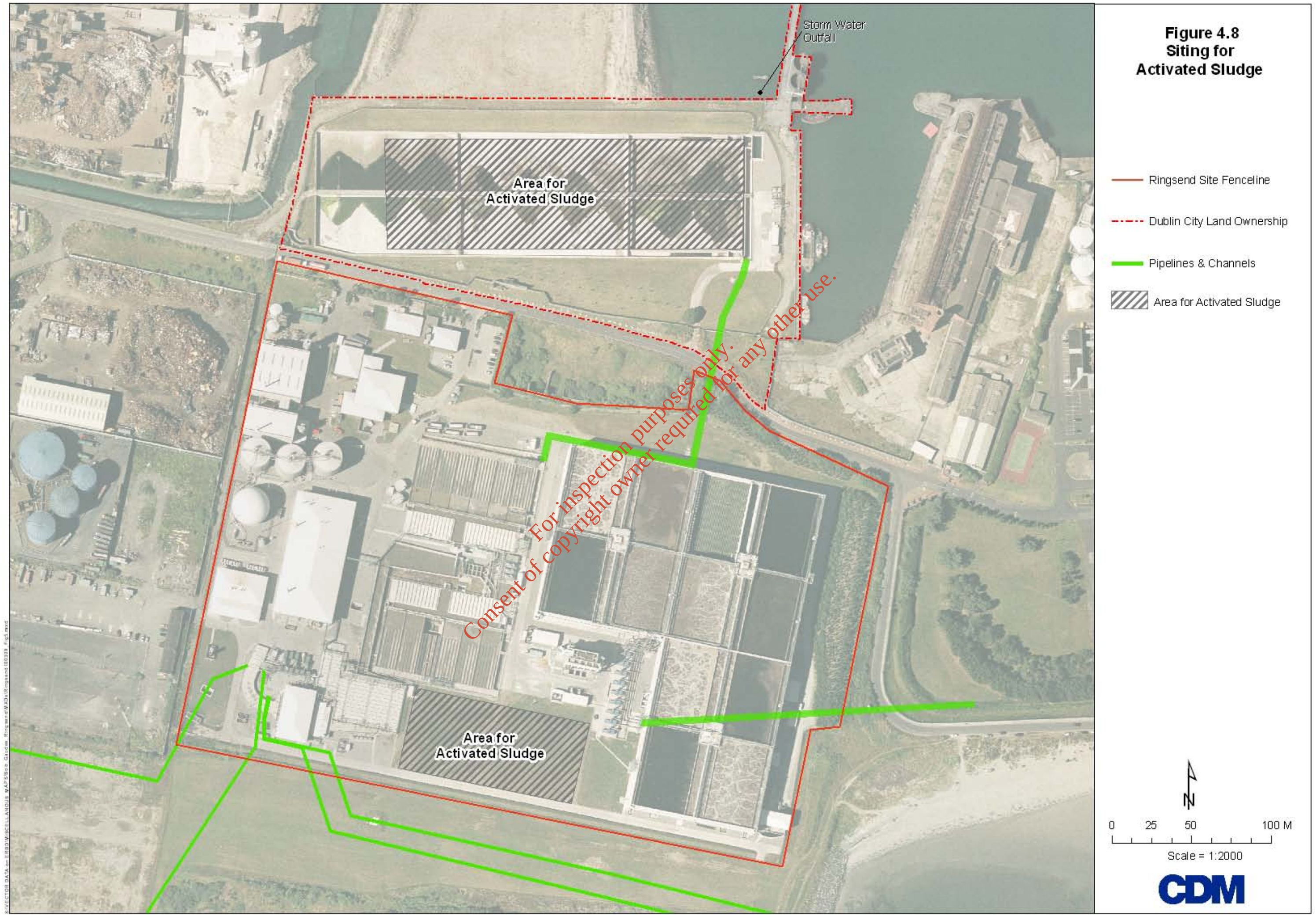





*Figure 4.8 Siting for Arctivated sludge Areas*

To convert the SBRs to the MLE process, requirements would include these items:

- A transverse wall to provide the volume needed for the anoxic zone;
- Mixers in the anoxic zone;
- Moving (perhaps replacing) diffusers and air piping; and
- Internal recycle pumps to transfer mixed liquor from the end of the aerated zone to the anoxic zone.

Yard piping to transfer mixed liquor, return activated sludge, and secondary effluent would have to be added. Effluent from the onsite secondary clarifiers would flow to the existing UV Disinfection Plant. Secondary effluent from the clarifiers in the storm basins would be disinfected in a new UV facility before discharge through the storm water outfall. If continuous discharge through the outfall is not allowed, effluent from clarifiers in the storm basin would have to be sent to the existing UV disinfection plant.

If this entire program is implemented, capacity would amount to about 1.85 million PE, as noted in Table 4.17. Even though this option occupies the entire site available for liquid treatment, its capacity is less than the 2.2-million-PE requirement. This complex option will not be further considered.

*Table 4.17 Estimated Capacity of Activated-Sludge Systems Providing TN of 10 mg/L*

| **Configuration** | **Million PE** |
|---|---|
| With SBRs as aeration tanks* | |
| Clarifiers on site | 1.01 |
| Clarifiers in storm tanks | 1.61 |
| Clarifiers on both sites | 1.85 |
| Stand-alone activated sludge | |
| On site | 0.16 |
| In storm tanks | 0.43 |
| Total | 0.59 |

*Note: Capacities of configurations with SBRs as aeration tanks are not additive. See text.*



### 4.4.3.3 Membrane Bioreactors

A membrane biological reactor (MBR) consists of a biological reactor with suspended biomass and solids separation by micro or ultra filtration membranes with nominal pore sizes ranging from 0.1 to 0.4 microns. The MBR process utilises activated sludge technology, but replaces conventional final settlement with a membrane that effectively filters the final effluent. MBR systems can operate at much higher MLSS concentrations (15,000 to 25,000 mg/L) than conventional activated sludge processes. However, MLSS concentrations in the range of 8,000 to 10,000 mg/L appear to be most cost effective when all factors are considered.

To provide treatment for 2.2 million PE, eight of the SBRs could be modified to serve as aeration tanks ahead of membranes, with the remaining SBRs operated to provide effluent total-nitrogen concentration of less than 10 mg/L. The membrane tanks would fit near the SBR tanks.

MBRs could be used in another fashion, as stand-alone units using their own aeration tanks rather than the SBRs. In this configuration, membrane tanks and their associated aeration tanks would be constructed in the open area near the SBRs. If aeration tanks and MBRs were built in the open space on site, the maximum capacity that would fit in the space would be 0.4 million PE. Table 4.16 shows that of the systems using the full capacity of the SBRs for nitrification, only the MBR options with CEPT will be capable of fitting onto the available area. Either the CEPT or CEPT plus sidestream treatment scheme would need additional treatment for about 0.3 million PE to achieve 2.2 million PE.

Table 4.18 presents estimated costs for the MBR options. Costs shown in the table include costs for converting the SBRs into aeration tank (including anoxic zones and internal recycle pumps) and for adding new aeration tanks for the stand-alone option.

*Table 4.18 Costs for MBR Options*

| **Parameter** | **With SBRs as Aeration Tanks** | **With Stand-Alone MBR System** |
|---|---|---|
| Millions of PE served | 2.2 | 0.30 |
| Cost Item | | |
| Capital cost | € 109,000,000 | € 34,000,000 |
| Contingency | € 38,000,000 | € 12,000,000 |
| Total capital cost | € 147,000,000 | € 46,000,000 |
| Annual operating cost | € 18,000,000 | € 3,000,000 |
| Total present worth | € 376,000,000 | € 85,000,000 |



#### 4.4.3.4 Deep-Shaft Process

The deep-shaft process is an activated-sludge process that uses an in-ground vertical shaft to provide biological treatment. The shaft can be up to 100 meters deep and 3 meters in diameter. The bioreactor consists of two tubes, one inside the other. Flow goes down the inner tube, and then up the space between the two tubes. As the depth of the aeration tank, its footprint is small and oxygen-transfer efficiency is high. At the large depths, solubility of nitrogen is very high, and flotation-type clarifiers are used rather than conventional clarifiers. Deep-shaft aeration has been applied only in special cases,because of limited availability of area, Ringsend could constitute a "special case."

The largest deep-shaft plant in operation for municipal wastewater is in Southport, UK. Its capacity is 122,000 PE, and there are industrial installations up to 397,000 PE. A deep-shaft installation at Ringsend would be six times larger than the largest deep-shaft installation at a municipal wastewater treatment plant.

Two suppliers were contacted for information relevant to this project. Aker Kvaerner Engineering Services, licensee for the ICI process, declined supplying information, believing that this is not the right technology for Ringsend. Noram Engineering did provide information for the VERTREAT system.

Noram Engineering provided estimates for a facility for 1.2 million PE. Total area required would be about 12,800 $m^2$. Allowing for access space, it was estimated that an installation with a capacity of 0.5 million PE could be built onsite adjacent to the SBRs, and an installation with a capacity of 0.2 million PE could be built in the triangular area between Pigeon House Road and the storm tanks. The total capacity is thus 0.7 million PE, sufficiently close to the 0.71 million PE additional capacity needed if the SBR units are operated in nitrifying mode and followed with denitrification filters. With CEPT and the SBRs operated in nitrifying mode, extra capacity needed is 0.3 million PE. Costs for deep-shaft treatment are presented in Table 4.19.

*Table 4.19 Costs for Deep-Shaft Treatment*

| Cost Item | Million Population Equivalents | | |
|---|---|---|---|
| | 0.2 | 0.5 | 0.3 |
| Capital cost | €19,700,000 | €34,100,000 | €25,100,000 |
| Contingency | €6,900,000 | €11,900,000 | €8,800,000 |
| Total capital cost | €26,600,000 | €46,000,000 | €33,900,000 |
| Annual operating cost | €900,000 | €2,200,000 | €1,300,000 |
| Total present worth | €38,300,000 | €74,600,000 | €50,800,000 |



The design criteria proposed by Noram are substantially less conservative than CDM's and further discussion would be needed before recommending their process.

### 4.4.3.5 Integrated Fixed-Film Activated Sludge

The capacity of the activated-sludge systems can be increased by adding to the aeration tanks material on which organisms grow as a film. This process is called "integrated fixed-film activated sludge" (IFAS). IFAS systems add materials such as ropes, sponges, and fixed or neutrally buoyant plastic material. With the added media, the concentration of biomass can be increased by about one-third compared with suspended-growth systems, resulting in decreased volume and surface area.

The Ringsend facility is much larger than any existing facility, and IFAS has not yet been attempted with SBRs. Two suppliers did not recommend the application at Ringsend. The option is dropped from further consideration.

### 4.4.3.6 Biological Filters

Biological filters are able to contain concentrations of biomass four or five times those of activated-sludge processes, thus decreasing the land area required as compared to conventional suspended-growth processes. The advantage of less land area must often be balanced against higher capital costs and greater operational complexity.

For application at Ringsend, the flow diagram would be as shown on Figure 4.9.

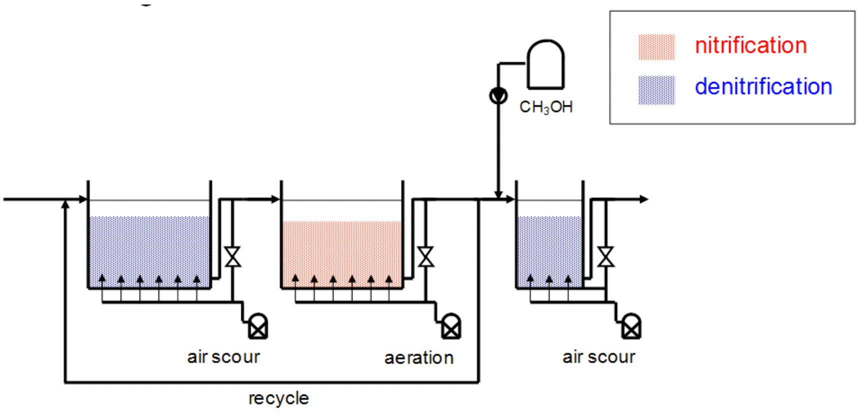



*Figure 4.9 Arrangement for Biological Filters*

The systems consists of denitrification cells followed by nitrification cells, with a recycle of nitrified effluent sent to the denitrification cells for reduction of nitrate to nitrogen gas. Polishing cells with methanol added would follow the nitrification cells.

From information provided by system suppliers, available area (including area near the SBRs and usable area between Pigeon House Road and the storage basins), would limit the capacity to 420,000 PE. This capacity would only be adequate with CEPT ahead of nitrifying SBRs. It was judged unwise to use all available area for a limited application. This option will not be examined further.



### 4.4.4 Phosphorus removal

Concentration of phosphorus in effluent from the SBRs averages about 5 mg/L. To meet effluent requirement of 1 mg/L, about 20 $m^3$/day of alum solution would be required. The cost for chemical and for processing the additional sludge would be €1,400,000/year.

## 4.5 Conclusions

Costs for deep-shaft and membrane treatment are summarised in three tables. Two tables are for systems with the SBRs operated to meet the effluent requirement for ammonia. The first table (4.20) is for deep-shaft treatment; the second table (4.21) is for membrane treatment. The third table is for SBRs operated to meet the effluent requirement for total nitrogen, with membrane treatment. There is no table for deep-shaft treatment with SBRs operated to meet total-nitrogen standards, as there is not enough space for those systems on site. In tables in this section, the costs and totals are rounded. Annual costs do not include current operating costs.

Table 4.23 summarises costs for the viable options for Ringsend. Costs include costs for process units and for chemical. Piping between units is not included.

Present-worth costs for options with deep-shaft treatment are much less than for than those for option with membrane treatment. However, membrane treatment is well established and considered reliable, and deep-shaft treatment has not been applied to any treatment plant close to the capacity required for Ringsend. Further, criteria used by the supplier of equipment for deep-shaft treatment were more aggressive than those used by CDM.

*Table 4.20 Costs for Systems with Deep-Shaft Treatment and SBRs Operated to Nitrify ahead of Denitrification Filters*

| **Option** | **With Conventional Primary Treatment** | **With CEPT** | **With Sidestream Treatment** | **With Both** |
|---|---|---|---|---|
| Primary treatment | | | | |
| Capital Cost | | | €4,700,000 | €4,700,000 |
| Annual O&M Cost | | €3,300,000 | €150,000 | €3,500,000 |
| Present-Worth | | €43,000,000 | €6,700,000 | €50,000,000 |
| Denitrification filters | | | | |
| Capital Cost | €24,700,000 | €30,900,000 | €17,400,000 | €23,100,000 |
| Annual O&M Cost | €1,700,000 | €2,500,000 | €1,000,000 | €1,500,000 |
| Present-Worth | €47,000,000 | €63,000,000 | €30,000,000 | €43,000,000 |
| Deep shaft | | | | |
| Capital Cost | €72,600,000 | €33,900,000 | €72,600,000 | €33,900,000 |
| Annual O&M Cost | €3,100,000 | €1,300,000 | €3,100,000 | €1,300,000 |
| Present-Worth | €113,000,000 | €51,000,000 | €113,000,000 | €51,000,000 |
| Phosphorus removal | | | | |
| Capital Cost | | | | |
| Annual O&M Cost | €1,400,000 | | €1,400,000 | |
| Present-Worth | €20,000,000 | | €20,000,000 | |
| Total | | | | |
| Capital Cost | €97,000,000 | €65,000,000 | €95,000,000 | €62,000,000 |
| Annual O&M Cost | €6,200,000 | €7,100,000 | €5,700,000 | €6,400,000 |
| Present-Worth | €177,000,000 | €157,000,000 | €169,000,000 | €145,000,000 |

*Table 4.21 Costs for Systems with Membrane Treatment and SBRs Operated to Nitrify ahead of Denitrification Filters*

| **Option** | **With Conventional Primary Treatment** | **With CEPT** | **With Sidestream Treatment** | **With Both** |
|---|---|---|---|---|
| Primary treatment | | | | |
| Capital Cost | | | | €4,700,000 |
| Annual O&M Cost | | €3,300,000 | | €3,500,000 |
| Present-Worth | | €43,000,000 | | €50,000,000 |
| Denitrification filters | | | | |
| Capital Cost | | €30,900,000 | | €23,100,000 |
| Annual O&M Cost | | €2,500,000 | | €1,500,000 |
| Present-Worth | | €63,000,000 | | €43,000,000 |
| Aeration tanks and membrane tanks | | | | |
| Capital Cost | | €46,000,000 | | €46,000,000 |
| Annual O&M Cost | | €3,000,000 | | €3,000,000 |
| Present-Worth | | €85,000,000 | | €85,000,000 |
| Phosphorus removal | | | | |
| Capital Cost | | | | |
| Annual O&M Cost | | | | |
| Present-Worth | | | | |
| Total | | | | |
| Capital Cost | | €77,000,000 | | €74,000,000 |
| Annual O&M Cost | | €8,800,000 | | €8,000,000 |
| Present-Worth | | €191,000,000 | | €178,000,000 |

*Table 4.22 Costs for Systems with Membrane Treatment and SBRs Operated to Meet Total-Nitrogen Requirement*

| **Option** | **With Conventional Primary Treatment** | **With CEPT** | **With Sidestream Treatment** | **With Both** |
|---|---|---|---|---|
| Primary treatment | | | | |
| Capital Cost | | | €4,700,000 | €4,700,000 |
| Annual O&M Cost | | €3,300,000 | | €3,500,000 |
| Present Worth | | €43,000,000 | €26,700,000 | €50,000,000 |
| MBRs | | | | |
| Capital Cost | €147,000,000 | €147,000,000 | €147,000,000 | €147,000,000 |
| Annual O&M Cost | €18,000,000 | €18,000,000 | €18,000,000 | €18,000,000 |
| Present Worth | €376,000,000 | €376,000,000 | €376,000,000 | €376,000,000 |
| Phosphorus removal | | | | |
| Capital Cost | | | | |
| Annual O&M Cost | €1,400,000 | | €1,400,000 | |
| Present Worth | €20,000,000 | | €20,000,000 | |
| Total | | | | |
| Capital Cost | €147,000,000 | €147,000,000 | €152,000,000 | €152,000,000 |
| Annual O&M Cost | €19,000,000 | €21,000,000 | €20,000,000 | €21,000,000 |
| Present Worth | €394,000,000 | €421,000,000 | €413,000,000 | €426,000,000 |



*Table 4.23 Summary of Costs for Viable Options for Nitrogen Removal*

| **Option** | **With Conventional Primary Treatment** | **With CEPT** | **With Sidestream Treatment** | **With Both** |
|---|---|---|---|---|
| *With SBRS Operated to Operated to Nitrify ahead of Denitrification Filters* | | | | |
| Deep-Shaft Option | | | | |
| Capital Cost | €97,000,000 | €65,000,000 | €95,000,000 | €62,000,000 |
| Annual O&M Cost | €6,200,000 | €7,100,000 | €5,700,000 | €6,400,000 |
| Present-Worth | €177,000,000 | €157,000,000 | €169,000,000 | €145,000,000 |
| MBR Option | | | | |
| Capital Cost | | €77,000,000 | | €77,000,000 |
| Annual O&M Cost | | €8,800,000 | | €8,000,000 |
| Present-Worth | | €191,000,000 | | €178,000,000 |
| *With SBRs Operated to Meet Effluent Requirements for Total Nitrogen* | | | | |
| Deep-Shaft Option | | | | |
| *Not enough space on site.* | | | | |
| MBR Option | | | | |
| Capital Cost | €147,000,000 | €147,000,000 | €152,000,000 | €152,000,000 |
| Annual O&M Cost | €19,000,000 | €21,000,000 | €20,000,000 | €18,000,000 |
| Present-Worth | €394,000,000 | €421,000,000 | €413,000,000 | €426,000,000 |

# Section 5
# Long Sea Outfalls

As previously noted the existing outfall discharges into the Liffey River Estuary north of the ESB Ringsend Power Station. In order to continue discharging to the estuary, the Works must meet the UWWT directive for nitrogen and phosphorus. As shown in Section 4, the costs to construct and operate facilities that provide nutrient removal in addition to secondary treatment at Ringsend for the 2.2 million PE design load are very high. Further, due to site restrictions the processes required to do so are unproven at the scale required for Ringsend.

In this section, alternative outfall terminus locations are considered that would achieve designated water quality criteria with the discharge treated to secondary standards, but without nutrient removal.

## 5.1 Assumptions

### 5.1.1 Discharge Flow and Loadings

The Works' ultimate forward flow (FFT) to treatment is 13.8 $m^3/s$. Accordingly, this flow is taken as the design basis for a long sea outfall. The Contract No. 2 hydraulic profile shows the effluent conduit of the UV plant to have a water surface elevation of 5.6 m (above OD Malin Head) at a flow of 13.8 $m^3/s$. This would be the controlling water surface elevation if disinfection is to remain in operation. If disinfection is not required with the long sea outfall, the controlling water surface elevation would become the effluent conduit of the lower SBRs, which is approximately 8.2 m OD (Malin).

The highest recorded tide in Dublin Port was 2.95 m OD (Malin). Given the 100+ year life expectancy of the pipeline, an allowance of 1 m for sea level increases is considered prudent. Thus, there would be a minimum available head of 1.65 m (5.6 m-3.95 m) to compensate for entrance and exit losses and pipeline friction loss. Conservatively assuming the entrance and exit losses to be 0.65 m, one meter of head remains as a driving force. If disinfection can be eliminated, the additional 2.6 m of available head on the outfall would permit a smaller pipeline diameter.

As a worst-case screening tool, maximum effluent loadings were assumed for modelling purposes. These values were derived by applying historical average effluent concentrations to the ultimate FFT. These worst-case mass and bacterial loadings should not be considered as typical. Further studies would be required to evaluate the impacts of a more typical range of flows and effluent loadings.

Table 5.1 summarises the worst-case discharge loading input into the dispersion model.

*Table 5.1 Assumed Worst-Case Discharge Loadings for Long Sea Outfalls*

| **Parameter** | **Value** |
|---|---|
| Flow | 13.8 m3/s |
| Ammonia Nitrogen ($NH_3$-N) | 5 mg/L |
| Dissolved Inorganic Nitrogen (DIN) | 22 mg/L |
| Molybdate Reactive Phosphorus (MRP) | 3.6 mg/L |
| Faecal Coliform[1,2] | 140,000 MPN/100 ml |

*1. E. Coli is accepted as a surrogate for Faecal Coliform*
*2. Value is chosen as arithmetic mean of bacterial counts prior to UV disinfection during the bathing season.*

## 5.1.2 Discharge Standards

Several sources of water quality standards apply to the Ringsend discharge. Environmental Quality Standards drafted in the European Communities Environmental Objectives (Surface Waters) Regulations 2009; Physiochemical Standards supporting Biological Elements (S.I. No. 272 of 2009) for compliance with the Water Framework Directive (2000/60/EC) and Bathing Water Quality Regulations 1992 Standards are used. In addition the more stringent bathing water quality standards required by the Blue Flag Beaches Programme have been assessed.

### 5.1.2.1 Water Quality Standards

The Water Framework Directive (2000/60/EC) was enacted in Ireland in 2003. Draft Regulations were proposed in September 2008 establishing Environmental Objectives and Environmental Quality Standards for the classification and management of Surface Waters and requiring the implementation of measures to reduce water pollution and protect and restore surface waters. These standards for physiochemical parameters affecting transitional and coastal waters were enacted in 2009 and are shown in Table 5.2.



*Table 5.2 European Communities Environmental Objectives (Surface Waters) Regulations 2009, Physiochemical Standards supporting Biological Elements (S.I. No. 272 of 2009)*

| **Parameter** | **Transitional Water Body** | **Coastal Water Body** |
|---|---|---|
| Temperature | <1.5ºC rise in ambient temperature downstream of a point of discharge | |
| BOD (mg $O_2$/L) | ≤4.0mg/L ($95^{th}$ percentile) | N/A |
| DIN (mg N/L) | N/A | Good status<br>(0 psu[1]) ≤2.6 mg N/L<br>(34.5 psu) ≤ 0.25 mg N/L<br>High statuL<br>(34.5 psu ) ≤0.17mg N/L |
| MRP (mg P/L) | (0-17 psu) ≤ 0.060 (median)<br>(35 psu) ≤ 0.040 (median) | |

*1. psu is the practical salinity unit - a unit of measurement of salinity similar to part per thousand (ppt)*

The Bathing Water Quality Regulations 2008 (S.I. 79 of 2008) will repeal and replace the Quality of Bathing Waters Regulations, 1992 (S.I. No. 155 of 1992) with effect from 31December 2014. Until the first monitoring calendar as specified in the new Bathing Water Regulations, 2008, is established for each Bathing Water on the 24 March 2011, the Bathing Water Standards as set in Schedule 2 Part I of the Quality of Bathing Waters Regulations, 1992 remain relevant and have therefore been used for comparison to model results. The standards are shown in Table 5.3.

*Table 5.3 Quality of Bathing Waters Regulations, 1992 (S.I. No. 155 of 1992)*

| Parameters | Guide | Mandatory |
|---|---|---|
| Total coliforms (Number/100ml) | ≤ 5,000 [1] | ≤ 10,000 [2] |
| Faecal coliforms (Number/100ml) | ≤ 1,000 [1] | ≤ 2,000 [2] |

1. *80% of the samples*
2. *95% of the samples*

In addition, a number of Bathing Water Beaches in Dublin seek to obtain Blue Flag Status. The more stringent Blue Flag standards are shown in Table 5.4.



*Table 5.4 Blue Flag Programme for Beaches – Water Quality Standards*

| Parameter | Unit | Standard | | Accepted % of test results higher than standard | |
|---|---|---|---|---|---|
| | | Guideline | Mandatory | Guideline | Mandatory |
| Total Coliforms | No./100ml | <500 | <10,000 | 20 | 5 |
| Faecal Coliforms[1] | No./100ml | <100 | < 2,000 | 20 | 5 |
| Faecal Streptococci | No./100ml | <100 | N/A | 10 | N/A |

1 *that E.Coli is accepted as a surrogate for Faecal Coliform*

### 5.1.3 Locations of Outfall Termini

Five potential outfall termini were chosen for evaluation. Stations 1 and 2 were chosen because they had been evaluated earlier in "*Modelling the Impact of Ringsend Wastewater Treatment Works and Storm Overflow Discharge in the Liffey and Tolka Estuaries and Dublin Bay*", CDM/DHI, April 2009. This document was prepared in support of the Ringsend WwTW Existing Discharge Licence Application. The other sites (Stations 3 through 5) were chosen by inspection to provide broad distribution of geographies beyond the transitional waters boundary, reasonably deep discharges, and to avoid shipping lanes. It was hoped that by initially examining a broad array of termini, that the more favourable (i.e. lesser impact) areas would be identified for further examination.

Table 5.5 displays the coordinates of the termini and Figure 5.1 displays them on a bathymetric map of the bay.

*Table 5.5 Outfall locations in Dublin Bay (UTM30)*

| X Coordinate | Y Coordinate | Station Number |
|---|---|---|
| 291071.3 | 5913249.5 | Bay 1 |
| 293191.1 | 5912568.7 | Bay 2 |
| 296067.2 | 5910494.6 | Bay 3 |
| 299988.6 | 5912494.4 | Bay 4 |
| 296922.2 | 5915317.0 | Bay 5 |

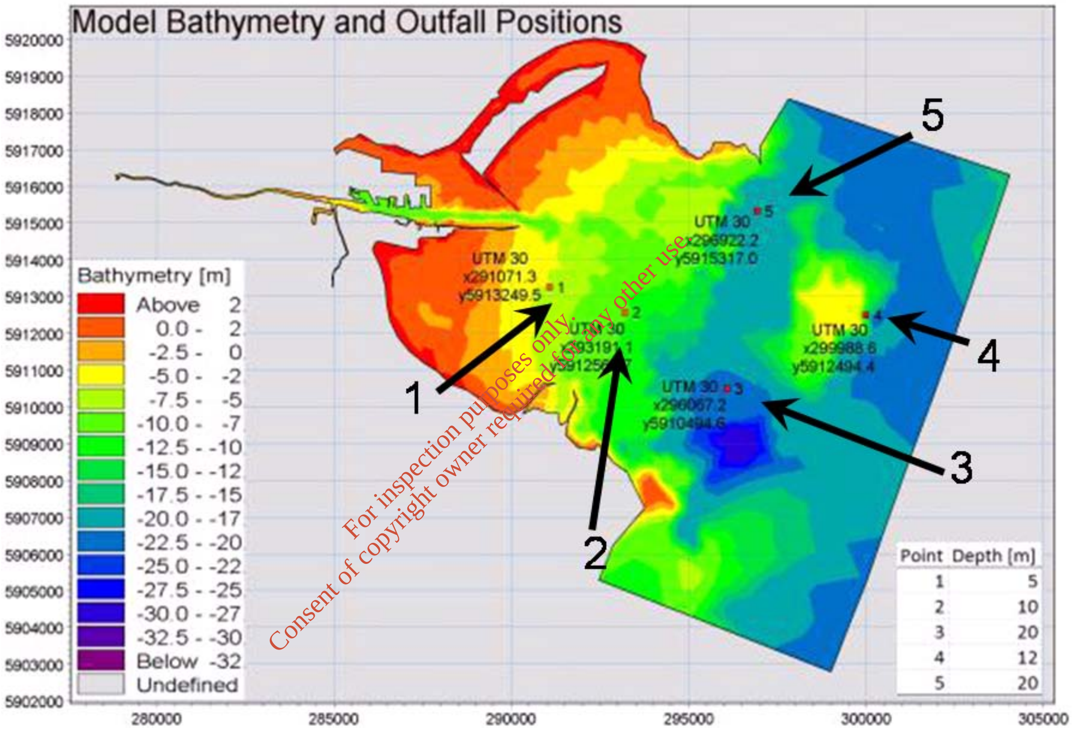



*Figure 5.1 Model Bathymetry and Outfall Positions*

## 5.2 Water Quality Impacts

The 3-dimensional model and its inputs and outputs are described in detail in *"Modelling the impact of Ringsend Discharges in the Liffey and Tolka Estuaries and Possible Long Sea Outfalls in Dublin Bay"*, CDM/DHI, October 2009. In that report, it was determined that Stations 3 and 4 show the least impacts. Neither location will adversely affect beaches or preservation areas. The report proves the initial feasibililty of ocean outfalls from a water quality perspective.

The environmental impacts of these two termini are examined in further detail in *"Preliminary Assessment of Long Sea Outfall Locations"*, CDM/JBB, January 2010, provided to EPA under a separate cover.

## 5.3 Geology of Dublin Bay

Initial investigations were undertaken to collect and examine existing data on the geology of Dublin Bay. The studies are presented in Appendix B "Overview of Geology in Dublin Bay" and are summarised herein.

### 5.3.1 Geology

The predominant bedrock in the inner Bay is likely to be Calp Limestone. This is the more easily solubilised, less resilient limestone that has eroded gradually, leaving a well-defined bay.

However, it cannot be determined if there are changes in the bedrock type, as there is little available information on the structural geology of the Bay at present. The Leinster Granite formation to the south of the Bay, from Dun Laoghaire to Dalkey, that may lie in the path of a tunnel to Station 3 terminus.

The Rathcoole Fault has been inferred fault from onshore geology to runs diagonally across the mouth of the Bay from the Rathcoole Fault in Dun Laoghaire to the Dalkey Fault. This fault is likely to be encountered with either of the outfall alignments.



While there have been a number of subsurface investigations conducted within the Bay, no boreholes have met bedrock with the deepest being 25 m below the surface.

A significant subsurface exploration program will be required if the outfall option is to proceed.

### 5.3.2 Bathymetry

The Bay, as defined by a straight line from the Baily Lighthoue to Dalkey Island, is shallow with depths typically less than 20 meters. The seabed deepens to the east until it reaches the Burford Bank, which sits centrally across the mouth of the Bay and is approximately 5 km in length. The Bank rises to within 5 m of the surface and the seabed deepens to 22 – 25 m again to its east.

INtegrated Mapping FOr the Sustainable Development of Ireland's Marine Resource (INFOMAR) is Ireland's near shore seabed mapping project. It is currently engaged in updating geophysical information on Dublin Bay, to include: hydrographic maps; seabed classification maps showing sediments and types; and habitat maps. Data from 2009 surveys are expected to be available in 2010.

## 5.4 Cost Estimates

Pipeline hydraulics were evaluated based on 1.0 m of available head and assumed lengths of 7.5 km and 10 km. The associated minimum finished diameters were 4.72 m and 5.01 m, respectively. A 5 m diameter was assumed for cost estimating purposes. Given the large diameter and the intent to minimise impacts to the marine environment from construction, it was further assumed that the outfall pipeline would be constructed as a tunnel.

The tunnel inlet would be located at the southeast corner of the ESB Ringsend Power Station site, below the site or in the foreshore immediately to the east of it. The existing outfall pipeline, constructed as a box culvert, passes nearby, providing ready access and reducing the length of the tunnel.

The tunnel would likely be constructed using a Tunnel Boring Machine (TBM). In order to have a finished diameter of 5.0 m, it is estimated to have a bored diameter of approx 5.90m. One 20 – 25 m diameter access shaft would be constructed for an entry point for the TBM. Bedrock below the Poolbeg Peninsula is in the range of 30 – 50 m deep, so it is likely that the access shaft would be at least 50 m deep.

The tunnel would be constructed in the bedrock of Dublin Bay. Further investigation is required to determine the location of the long sea outfall terminus but it is expected to be either in an easterly direction from Ringsend WwTW and terminate beyond Burford Bank or be in an east south-east direction. The map in Figure 5.2 display's the approximate locations for the outfall.

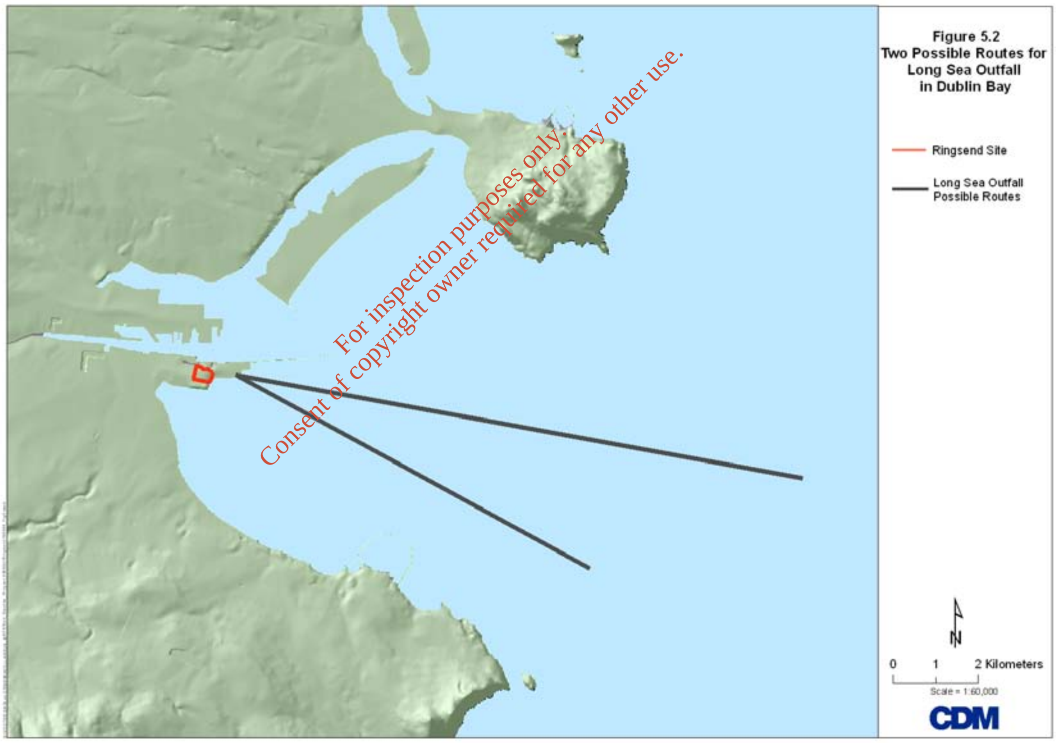




*Figure 5.2.Dublin Bay and two possible routes for the Long Sea Outfall*

In order to assess the likely range of costs to be incurred, historical data on other large diameter tunnels was undertaken. The aim of this analysis was to determine a typical or average unit outfall cost, expressed as Euro per millimeter diameter per meter of length (€/mm DIA/m), which could then be applied to the outfall lengths applicable to Ringsend. Sources of information included:

- The British Tunnelling Society;
- CDM employees (design and construction of tunnels);

- Australia database of tunnels;
- American Society of Civil Engineers, Marine Outfall Construction; and
- Other sources available from public files, including consultants and construction companies.

### 5.4.1 Data Analysis

Using the sources mentioned previously a list of approximately 60 tunnels was compiled. Information collated included the service for which it was constructed (e.g. water, sewage, stormwater, etc), internal and bored diameter, year constructed, length, type of rock, cost, etc. Unit costs were calculated using the total length, internal diameter and cost for construction. The data tended to fall evenly into two groupings. One grouping was considered to have sufficient information and used to calculate the average cost for constructing the tunnel. The second grouping, while having some valuable information, was not considered detailed enough to be included in the cost data base.

The entire data base is included in Appendix C, "Ringsend WwTW Proposed Tunnelling Outfall Cost".

Tunnels which were completed prior to 2009 are adjusted using the Engineering News-Record Construction Cost Index (CCI) or the UK Resource Cost Index for Infrastructure. Cost was calculated from the mid-point of construction of the completed works.

For UK tunnels constructed prior to 1997, the US CCI data was used. For tunnels constructed in Australia, Iraq, Egypt etc. the US CCI data was used.

The unit cost was calculated using the following steps:

1. Construction Cost Index multiplier: =

*Construction Cost Index 2009 / Construction Cost Index for mid-point of Construction*

2. Cost €/mm DIA/m =

*CCI multiplier x (Cost / Diameter / Length)*

Cost = €, Diameter = ID in mm, Length = m.

The resultant unit tunnel costs are presented in Table 5.6.

*Table 5.6 Unit Tunnel Costs*

| | **Average (€/mmDIA/m)** | **Median (€/mmDIA/m)** |
|---|---|---|
| British Tunnels | 2.24 | 2.00 |
| European Tunnels | 2.38 | 2.24 |
| Non-European Tunnels | 2.38 | 2.50 |
| All Tunnels | 2.38 | 2.34 |

While the range of tunnel costs is quite variable due to local labour costs, construction method, types of rock or soils encountered and other factors, a central tendency of unit costs in the range of 2.00 to 2.50 €/mmDIA/m emerges.

Again, to be conservative, the higher value of 2.50 €/mmDIA/m shall be used for cost comparison purposes. Thus, a 5000 mm diameter tunnel would cost €12,500 per meter of length prior to any contingencies.

## 5.4.2 Capital Cost Estimate

As with previous cost estimates, a contingency factor of 35% is applied to the tunnel capital cost estimates. Costs are rounded to the closest million Euro.

*Table 5.7 Costs for Tunnel Options*

| | **7.5 km tunnel** | **10.0 km tunnel** |
|---|---|---|
| Capital Cost | €94,000,000 | €125,000,000 |
| Contingency | €33,000,000 | €44,000,000 |
| Total Capital Cost | €127,000,000 | €169,000,000 |

There are no annual operating costs associated with the outfall tunnels. Therefore, Total Capital Cost is equivalent to Total Present Worth Cost.

# Section 6
# Alternatives Analysis

This section compares alternatives that would provide nutrient removal, compatible with discharge into the Liffey River Estuary, versus an alternative that provides secondary treatment without nutrient removal or disinfection, compatible with an ocean discharge. Cost and non-cost factors are considered.

In all cases, it is assumed that co-settling in the primary clarifiers will cease because newly installed SAS thickening capacity is sufficient to thicken all the SAS generated.

The Works currently generates approximately 98 tonnes per day (tpd) of sludge (dry weight) at an influent loading of 1.79 million PE. With no changes in processing, sludge production would be expected to increase in approximate proportion to influent loading to 120 tpd at the design year loading of 2.2 million PE. The current sludge stream expansion project will bring the digestion capacity to 120 tpd. Thus, any net increase to sludge production as a result of chemical addition will necessitate further expansion of the sludge stream capacity.

In all cases the storm tanks would continue to treat flows in excess of the FFT, but the FFT would be increased to 13.8 $m^3/s$.



## 6.1 Description of Alternatives Compatible with Discharge into the Liffey River Estuary

As presented in Section 4, there are several alternative wastewater treatment scenarios that can achieve UWWT discharge standards of 10 mg/L Total Nitrogen and 1 mg/L Total Phosphorus at average daily loadings of 2.2 million PE. Due to the extreme site constraints, only MBR and deep shaft technologies are able to achieve the design goals and then only in concert with other wastewater treatment processes. The MBR alternatives that would require full nitrification and denitrification in the SBRs are far more expensive than the other alternatives, especially in the area of operating costs, and have been removed from further consideration.

CDM has some concerns that the design criteria provided by the deep shaft technology vendor may not be equivalent to CDM's more conservative design criteria, but the criteria will be accepted for the purposes of cost and non-cost comparisons. If a deep shaft alternative is selected for implementation, further evaluation of system capacity would be required.

### 6.1.1 Deep Shaft Aeration with Conventional Primary Treatment

Under this alternative the SBRs would be operated to achieve full nitrification for an influent loading of 1.49 million PE. Denitrification filters would be provided. Alum would be added to the denitrification filters for phosphorus control.

Deep shaft aeration systems would be constructed in the 0.8 hectare open space on site (0.5 million PE) and in the triangular open space to the southeast of the storm tanks (0.2 million PE).

SAS production rates would be very similar to current rates and would increase in general proportion to loading increases. Alum sludge from phosphorus control would increase total sludge production by an additional 9 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €8 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced.

### 6.1.2 Deep Shaft Aeration with CEPT

Under this alternative the primary tanks would be dosed with alum to enhance TSS removal as well as to control phosphorus. The SBRs would be operated to achieve full nitrification for an influent loading of 1.90 million PE. Denitrification filters would be provided.

A deep shaft aeration system, sized for 300,000 PE, would be constructed in the 0.8 hectare open space on site.

Net sludge would increase by approximately 26 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €23 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced.



### 6.1.3 Deep Shaft Aeration with Sidestream Treatment

Under this alternative the SBRs would e operated to achieve full nitrification for an influent loading of 1.49 million PE. Denitrification filters would be provided. Sidestream treatment (either SHARON or ANAMMOX) would be provided to reduce the nitrogen load returning to the SBRs. While this does not increase the SBRs' capacity to remove BOD, it would reduce power consumption within the SBRs and also reduce the mass of nitrate to be denitrified in the denitrification filters. Alum would be added to the denitrification filters for phosphorus control.

Deep shaft aeration systems would be constructed in the 0.8 hectare open space on site (0.5 million PE) and in the triangular open space to the southeast of the storm tanks (0.2 million PE).

SAS production rates would be very similar to current rates, and would increase in general proportion to loading increases. Alum sludge from phosphorus control would increase total sludge production by an additional 9 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €8 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced.

### 6.1.4 Deep Shaft Aeration with CEPT and Sidestream Treatment

Under this alternative the primary tanks would be dosed with alum to enhance TSS removal as well as to control phosphorus. Sidestream treatment would remove a portion of the nitrogen from recycle streams. The SBRs would be operated to achieve full nitrification for an influent loading of 1.90 million PE. Denitrification filters would be provided.

A deep shaft aeration system, sized for 300,000 PE, would be constructed in the 0.8 hectare open space on site.

Net sludge would increase by approximately 26 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €23 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced.

### 6.1.5 Membrane Bioreactors with CEPT

Under this alternative the primary tanks would be dosed with alum to enhance TSS removal as well as to control phosphorus. The SBRs would be operated to achieve full nitrification for an influent loading of 1.90 million PE. Solids would be removed by membranes. Denitrification filters would be provided.



A stand-alone MBR system, sized for 300,000 PE, would be constructed in the 0.8 hectare open space on site.

Net sludge would increase by approximately 26 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €23 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced because the membranes would only treat about 14% of the total flow.

### 6.1.6 Membrane Bioreactors with CEPT and Sidestream Treatment

Under this alternative the primary tanks would be dosed with alum to enhance TSS removal as well as to control phosphorus. Sidestream treatment would remove a portion of the nitrogen from recycle streams. The SBRs would be operated to achieve full nitrification for an influent loading of 1.90 million PE. Solids would be removed by membranes. Denitrification filters would be provided.

A stand-alone MBR system, sized for 300,000 PE, would be constructed in the 0.8 hectare open space on site.

Net sludge would increase by approximately 26 tpd in the design year. Based on the costs of the current sludge stream expansion project, the capital cost to provide capacity for alum sludge is about €23 million, inclusive of contingencies.

Seasonal UV disinfection would continue to be practiced because the membranes would only treat about 14% of the total flow.

## 6.2 Alternatives Compatible with Ocean Discharge

It is assumed that only BOD and TSS will need to be removed at secondary treatment standards if there is an ocean discharge. Under this scenario, the existing SBRs would be operated in BOD removal mode to treat an influent loading of 2.2 million PE. Two blowers, with the same unit capacity of the existing blowers would be added. Sludge production rates are expected to be similar to current rates, but would increase in proportion to the loading increase.

TSS compliance is expected to increase with the elimination of co-settling and the installation of wave attenuation devices in the upper level SBRs. As a contingency, €9 million, inclusive of contingencies, is assumed for rigid GRP covers in the event that the combination of reduced loadings and wave attenuation does not improve effluent compliance sufficiently. Note that the cost of SBR covers has not been applied to the alternatives that include denitrification filters because the filters would remove TSS that would carry over from the SBRs.

It is assumed UV disinfection would no longer be practiced. The potential power savings would approximately 800 megawatt-hrs per year.

No alum or methanol addition is required because there will be no requirements for phosphorus or nitrogen control.



## 6.3 Cost Analysis

Table 6.1 presents the capital, annual, and present worth costs of the seven alternatives presented in Section 6.1 and 6.2.

The alternative providing secondary treatment with an ocean outfall is the least costly on a present worth basis. The cost is sensitive to the length of the outfall, ranging from €176 million for a 7.5 km outfall to €218 million for a 10 km outfall. It is possible that a longer outfall may be necessary, but studies conducted to date indicate that the shorter (7.5 km) outfall length would meet all water quality standards.

The present worth costs of deep shaft aeration alternatives are comparable to that of a 10 km outfall, but 20% to 30% more costly than a7.5 km outfall.

Membrane bioreactors alternatives are much more expensive than any of the other alternatives.

*Table 6.1 Cost Comparison of Wastewater Treatment Alternatives*

| | **Nutrient Removal Alternatives** | | | | | | **Secondary Treatment** | |
|---|---|---|---|---|---|---|---|---|
| | **Deep Shaft Aeration** | | | | **MBRs** | | **SBRs** | **SBRs** |
| | **Conventional Primary** | **with CEPT** | **with Sidestream** | **with Both** | **with CEPT** | **with Both** | **with 7.5 km Outfall** | **with 10 km Outfall** |
| Capital Cost | | | | | | | | |
| Base Alternative | € 97,000,000 | € 65,000,000 | € 95,000,000 | € 62,000,000 | € 77,000,000 | € 74,000,000 | € 9,000,000 | € 9,000,000 |
| Additional Sludge Facilities | € 8,000,000 | € 23,000,000 | € 8,000,000 | € 23,000,000 | € 23,000,000 | € 23,000,000 | € - | € - |
| Outfall | € - | € - | € - | € - | € - | € - | € 127,000,000 | € 169,000,000 |
| Total | € 105,000,000 | € 88,000,000 | € 103,000,000 | € 85,000,000 | € 100,000,000 | € 97,000,000 | € 136,000,000 | € 178,000,000 |
| | | | | | | | | |
| Annual Operating Cost | | | | | | | | |
| Base Alternative | € 6,200,000 | € 7,100,000 | € 5,700,000 | € 6,400,000 | € 8,800,000 | € 8,000,000 | | |
| Existing SBRs - Power | € 2,500,000 | € 3,100,000 | € 2,500,000 | € 3,100,000 | € 3,100,000 | € 3,100,000 | € 3,100,000 | € 3,100,000 |
| UV Disinfection | € 200,000 | € 200,000 | € 200,000 | € 200,000 | € 200,000 | € 200,000 | | |
| Total Annual | € 8,900,000 | € 10,400,000 | € 8,400,000 | € 9,700,000 | € 12,100,000 | € 11,300,000 | € 3,100,000 | € 3,100,000 |
| | | | | | | | | |
| Present Worth | € 220,000,000 | € 223,000,000 | € 211,000,000 | € 211,000,000 | € 257,000,000 | € 244,000,000 | € 176,000,000 | € 218,000,000 |

## 6.4 Non-Cost Factors

Non-cost factors may be divided into categories that are objective and those that are subjective.

Objective factors are those that are measurable. They are related power consumption and chemical consumption as well as the quantities of sludge that are generated and must be treated and disposed of. Increased sludge production adds levels of indirect power and chemical consumption, as well as additional solids to be disposed.

Subjective factors are more associated with the ability of the proposed alternatives to be operated and maintained efficiently. The less complex the alternative system, the easier it is to understand, to operate and to maintain.

### 6.4.1 Direct Power Consumption from Wastewater Treatment

Direct power consumption for each of the alternatives is that derived from treating the wastewater to the appropriate discharge standards. The categories are summarized as follows:

- Aeration and pumping in SBRs
- Aeration, pumping and solids separation in deep shaft systems
- Membrane system power
- UV disinfection
- Denitrification system pumping



### 6.4.2 Indirect Power Consumption from Sludge Processing

The THP/digestion system has a net power consumption of approximately 0.26 MWh per tonne of dry solids processed. There is potential to improve upon this consumption rate by limiting the energy losses resulting from flaring digester gas and by operating the steam generators downstream of the CHP engines more frequently. By beneficially utilizing virtually all of the digester gas unit consumption would be reduced to 0.19 MWh/tonne dry solids. A slight increase in steam generator operation should be achievable, further reducing unit power consumption to 0.15 MWh/tonne dry solids.

The volatile suspended solids (VSS) destruction rate in the digesters is approximately 55% and the ratio of VSS to TSS is approximately 80%. Therefore, the overall mass destruction rate in the digesters is approximately 44% of its input. For every tonne of sludge fed to the THP/digestion system, 0.56 tonnes will be fed to the dryer system.

Sludge fed to the dryers is mechanically dewatered to approximately 22% dry matter. The dryers further reduce water content to 92% dry matter. For every tonne of sludge fed, approximately 760 kg of water is evaporated. At a water evaporation rate of 0.978 kWh/kg (per O&M Manual) approximately 0.75 MWh of natural gas is consumed per wet tonne of sludge fed to the dryers or 3.4 MWh/tonne dry solids at the current feed solids concentration. Since only 56% of the solids fed to the THP/digestion system are fed to the dryers, the net effect is to reduce the dryer energy consumption, as applied to sludge produced, to 1.90 MWh/tonne dry solids.

The total indirect power consumption is the sum of that devoted to the THP/digestion system plus the dryers, or 2.05 MWh/tonne dry solids. This rate is applied to the additional solids produce from chemical addition. It is noted that unit power consumption may actually be higher because much of the sludge generated from chemical addition will be inorganic and destruction rates in the digesters may not continue to be as high as they currently are.

Table 6.2 summarizes energy consumption from each of the alternatives and compares each of the alternatives to the secondary treatment alternative, which consumes the least amount of electricity.

Generation of electricity produces greenhouse gases, the rates of which vary according to the source of the electric power from renewable sources on the low end of the scale to coal and peat at the upper end. While the rates vary according to the sources, total emissions will be linear with consumption for the same mix of sources.

## 6.4.3 Chemical Consumption

Table 6.2 also presents a summary of the chemicals that would be used for each of the alternatives. The secondary treatment options would require no additional chemicals for wastewater treatment. The others would consume between 10,000 $m^3$/yr and 20,000 $m^3$/yr of alum, methanol, polymer, sodium hypochlorite and citric acid. These chemicals produce secondary greenhouse gas emissions from the production and transport and the additional sludge they produced further add to greenhouse emissions from sludge processing and disposal.



## 6.4.4 Other Non-Cost Factors

### 6.4.4.1 Water Quality

All alternatives discharging to the Liffey River Estuary, while compliant with UWWT standards, will add pollutant loading to the estuary and the inner bay, which contain Special Protected Areas (SPAs), Special Areas of Conservation (SACs) and Bathing Waters. The ocean outfall alternative would improve water quality in the estuary and the bay without impacting any of the SPAs, SACs, or Bathing Waters as demonstrated in *"Preliminary Assessment of Long Sea Outfall Locations", CDM, Jan 2010.*

*Table 6.2 Electrical Power and Chemical Consumption of Wastewater Treatment Alternatives*

| | Nutrient Removal Alternatives | | | | | | Secondary Treatment |
|---|---|---|---|---|---|---|---|
| | Deep Shaft Aeration | | | | MBRs | | |
| | Conventional Primary | with CEPT | with Sidestream [1] | with Both [1] | with CEPT | with Both [1] | with Outfall |
| Power Consumption (MWh/yr) | | | | | | | |
| Direct Power Consumption | 39,400 | 33,800 | 40,600 | 35,000 | 49,700 | 50,900 | 25,000 |
| Indirect Power Consumption | 23,400 | 36,200 | 23,400 | 36,200 | 36,200 | 36,200 | 16,500 |
| Total | 62,800 | 70,000 | 64,000 | 71,200 | 85,900 | 87,100 | 41,500 |
| | | | | | | | |
| Ratio to Secondary Treatment with Outfall | 151% | 169% | 154% | 172% | 207% | 210% | 100% |
| | | | | | | | |
| Chemical Consumption | | | | | | | |
| Alum ($m^3$/yr) | 7,300 | 15,400 | 7,300 | 15,400 | 15,400 | 15,400 | - |
| Methanol ($m^3$/yr) | 3,500 | 5,100 | 2,000 | 3,100 | 5,100 | 3,100 | - |
| Polymer [2] ($m^3$/yr) | 200 | 100 | 200 | 100 | | | |
| Sodium Hypochlorite ($m^3$/yr) | | | | | 65 | 65 | |
| Citric Acid ($m^3$/yr) | | | | | 45 | 45 | |
| Total | 11,000 | 20,600 | 9,500 | 18,600 | 20,610 | 18,610 | - |

1. *Assumes ANAMMOX process*
2. *Assumes 25% active liquid polymer*

### 6.4.4.2 Reliability

The Secondary Treatment/Ocean Outfall alternatives would continue to use the SBRs for biological treatment. The SBRs themselves would not be modified other than to possibly add covers to the upper level basins and to add some blower capacity. The SBR process is well established at Ringsend and has proven reliable under stressed conditions of extended peak biological loadings. In addition, the process is well known to the operations and maintenance staff.

All alternatives compatible with discharge to the Liffey River Estuary would require the addition of denitrification facilities downstream of the SBRs. While this is considered to be a reliable process, any additional unit processes reduce overall system reliability.

Deep shaft aeration has not been practiced at a scale comparable to that at Ringsend and hence there is some risk of scale up. In addition, the double-decked flotation clarifier arrangement would be unique. As previously noted, the design criteria used by the vendor are not as conservative as those CDM would normally recommend. There is, therefore, some risk that the facilities may not achieve the design year loadings.



Membrane bioreactors are more complex than the other systems considered. Potential fouling of the membranes is a particular concern.

### 6.4.4.3 Ease of Operations

This criterion considers the relative ease of operation for each of the alternatives. A lesser number of unit processes is desirable. Also, the degree of operability is generally considered to be inversely related to complexity.

Deep Shaft Aeration with Conventional Primary Treatment would add two unit processes (denitrification and deep shaft aeration). In addition, deep shaft aeration facilities would be located on the existing site and adjacent to the storm tanks, making operational logistics more difficult.

Deep Shaft Aeration with CEPT would add three unit processes. Treatment of the much increased sludge production would place further demands on operations staff.

Deep Shaft Aeration with Sidestream Treatment would add three unit processes. The deep shaft systems would be located on the existing site and adjacent to the storm tanks.

Deep Shaft Aeration with CEPT and Sidestream Treatment would add four unit processes. Treatment of the much increased sludge production would place additional demands on operations staff.

Membrane Bioreactors with CEPT would add three unit processes. As previously noted, membranes are considered to be more complex than the other alternatives. In addition, treatment of the much increased sludge production would place additional demands on operations staff.

Membrane Bioreactors with CEPT and Sidestream Treatment would add four unit processes. Again, the additional sludge produced would place additional demands on operations staff.

Secondary Treatment with an Ocean Outfall would add no new unit processes, and seasonal disinfection would no longer be practiced.

#### 6.4.4.4 Maintenance

This criterion considers the relative ease of maintenance for each of the alternatives. A lesser number of unit processes is desirable, because there would be a lesser number subsystems and different equipment items to understand, maintain, and support. The existing spare parts storage area is limited and would need to be expanded to support any significant increase in equipment items. Maintenance requirements will generally increase as control systems become more complex due to the number of field instruments that must be calibrated and maintained. It is often difficult to train and retain electricians and instrumentation and control systems specialists at wastewater treatment works because they are in high demand elsewhere.

Denitrification requires chemical storage and feed systems, which are considered to be easy to maintain. Backwash pumps and controls are fairly simple and require only routine maintenance. However, there will be a large number of equipment items due to the numbers of individual filters to be added.



CEPT requires chemical storage and feed systems. Neither of which are very difficult to maintain.

Sidestream treatment systems would be proprietary designs and their control systems may not be totally compatible with other control systems at the Works. The SHARON process requires the addition of a soluble carbon source, such a methanol, and the ANAMMOX process uses ammonia.

Deep Shaft Aeration systems would be proprietary designs and their control systems may not be totally compatible with other control systems at the Works. These systems would add several types of pumps, as well as blowers, compressors and flotation clarifiers. Polymers would be used to assist in TSS capture.

Membrane Bioreactors systems would proprietary designs and their control systems may not be totally compatible with other control systems at the Works. Alternatives including membrane bioreactors would be the most intensely instrumented. Preventative maintenance on the membranes requires periodic treatment with different chemicals to remove fouling. At some point in time – perhaps 7 to 10 years – the membranes will have to be replaced at a very great cost.

Secondary Treatment with an Ocean Outfall would not require any new equipment. The additional aeration blowers would be sourced from the same vendor as the existing blowers in order to reduce spare parts inventories.

Elimination of UV disinfection would eliminate yearly programmes to prepare the system for operation, including time consuming and expensive re-lamping.

# Section 7 Conclusions and Recommendations

## 7.1 Cost Comparison

In order to achieve required levels of nutrient reduction at the current discharge location, treatment alternatives including deep shaft aeration have present worth costs ranging between €211 million and €223 million. The least cost alternatives would require pretreatment with either CEPT or CEPT plus sidestream treatment.

Alternatives including MBRs had present worth costs of €244 million and €257 million, depending upon the level of pretreatment.

Two outfall scenarios, at lengths of 7.5 km and 10 km, were considered for discharge of secondary treated effluent. Based on initial dispersion modelling and environmental assessments, there is confidence that an outfall terminus falling within this range of lengths is likely. Present worth costs are heavily influenced by outfall length, with preliminary cost estimates ranging from €176 million and €218 million, or between 83% and 103% of the next lowest cost alternatives.

The secondary treatment alternative has a distinct cost advantage over those providing nutrient removal.



## 7.2 Energy Consumption

The secondary treatment alternative would be operated to avoid nitrification and reduce the associated power for oxidation of nitrogen species. No chemicals would be applied and, therefore, no chemical sludge would be produced. Direct power consumption for the SBRs plus indirect power consumption for sludge treatment is estimated to be approximately 41,500 MWh/yr in the design year.

In addition to higher power demand for nitrification, all nutrient removal alternatives require chemical addition for CEPT and/or phosphorus control. Additional chemical sludge quantities add substantial indirect power demands. The least energy intensive of these alternatives would consume 50% more power than the secondary treatment alternative. The most intensive would consume more than 110% more power.

The secondary treatment alterative has a distinct energy advantage over those providing nutrient removal.

## 7.3 Chemical Consumption

The secondary treatment alternative would require no chemicals other than those currently used for solids processing.

Nutrient removal alternatives would require between 10,000 $m^3$/yr and 20,000 $m^3$/yr of alum, methanol or other soluble carbon source, polymer, sodium hypochlorite, and citric acid. Deliveries are estimated at two to four tankers per day.

The secondary treatment alterative has a distinct advantage in chemical consumption over those providing nutrient removal.

## 7.4 Sludge Production

The alternatives including CEPT will generate approximately 9,600 tonnes per year more sludge than the secondary treatment alternative. Those that do not include CEPT will generate approximately 3,400 tonnes per year more sludge from phosphorus removal in the denitrification filters.

The secondary treatment alterative has a distinct advantage in sludge production over those providing nutrient removal.

## 7.5 Greenhouse Gases

Power consumption has the most direct relationship with greenhouse gas production. However, the ratio of carbon emissions to energy consumption at the Ringsend WwTW is complex because it varies greatly with the source of energy used in power production (by the utility), the degree to which the CHPs are operated with natural gas, the ratio of sludge dried (with natural gas) or dewatered as biocake, and the degree to which energy is recovered from sludge processing. With all these variables it is difficult to establish a greenhouse gas generation rate that can be applied to consumption. It is much safer to say that greenhouse gases will rise in general proportion to net energy consumption.



Chemical production and transportation generate greenhouse gases as does sludge treatment, transportation to the distribution centre, and incorporation into the soil.

Since the secondary treatment alternative has distinct advantages in power consumption, chemical consumption and sludge production, it follows that it has a distinct advantage in greenhouse gas emissions.

## 7.6 Reliability

The secondary treatment process does not add any new unit operations. The operation and maintenance of the SBR process is well established at Ringsend and elsewhere. The reduction of MLSS concentration will improve effluent TSS. There is little risk in continuing to use this process.

Deep shaft aeration is an established technology, but has not yet been scaled up to a facility with the capacity required for Ringsend. There is some concern that the design factors used by the vendors are aggressive and, hence, present a degree of risk.

MBRs are very stable but require constant chemical treatment to ensure that the membranes do not foul. Membrane replacement will be required at least once in the planning period.

The secondary treatment alterative has a reliability advantage over those providing nutrient removal.

## 7.7 Ease of Operations and Maintenance

As previously noted, SBRs process is well established at Ringsend and is considered the baseline for comparisons. Secondary treatment adds no unit processes. With the elimination of seasonal disinfection, there will actually be a decrease in demand for operations and maintenance activities.

The nutrient removal alternatives require the continued operation of the SBRs in addition to another biological process – either deep shaft aeration or MBRs – and between one and three additional unit processes. Operators and maintenance personnel must be trained on several new systems and even more sub-systems

The level of sophistication of the high rate biological processes is higher than that of the existing SBRs. Specialists required to maintain these systems are in high demand and may not be readily available.

The secondary treatment alterative has a very distinct advantage in ease of operations and maintenance over those providing nutrient removal.



## 7.8 Water Quality

Modelling of the two (long sea) outfall options demonstrated that secondary treated discharges would not have any impact on Coastal or Transitional water bodies. Appropriate Assessments of these outfall discharges conclude that there would be changes in water quality in the immediate vicinities of the discharges, but that no significant impacts were predicted for any Natura 2000 site. Further, by moving the discharge terminus outside of Dublin Bay, water quality within the estuary and the inner bay, where Natura 2000 sites and bathing waters are located, will improve.

## 7.9 Recommendation

Secondary treatment with an ocean outfall discharge is the low cost alternative. It also consumes less energy and chemicals and produces less sludge and greenhouse gases. It requires no new unit process and is, therefore, much simpler to operate and maintain. It is the low-risk alternative. The discharge, while not treated to the same levels as the other alternatives, would meet water quality standards and be more protective of existing Natura 2000 sites and bathing waters.

Pending an environmental impact assessment, it appears that providing secondary treatment with an ocean outfall discharge would be the most beneficial option for the Ringsend WwTW Extension.

# Appendix A

# Ringsend Wastewater Treatment Works Extension Baseline Report

# RINGSEND WASTEWATER TREATMENT WORKS EXTENSION

# BASELINE REPORT



CDM

December 2008

# Document Control Sheet

| | | | | |
|---|---|---|---|---|
| Client | Dublin City Council | | | |
| Project | Ringsend | | | |
| Report | Ringsend Baseline Report | | | |
| Date | 11/DEC/08 | | | |
| Project No: 67 511 | | Document Reference: | | |
| **Version** | **Author** | **Reviewed** | **Checked** | **Date** |
| Draft 01 | Bob Gaudes | C. O'Reilly | Bob Gaudes | 10/11/08 |
| Draft 02 | Bob Gaudes | P. Fullam, C. O'Reilly, E. Schweizer, P. Loomis | Bob Gaudes | 13/11/08 |
| Draft 03 | Bob Gaudes | D. Molphy | Bob Gaudes | 17/11/08 |
| Final | Bob Gaudes | | | 11/12/08 |

# Table of Contents

# 1 Initial Observations

The works appears to be in reasonably good condition, vastly improved since CDM's first visit three years ago. Many of the improvements, in particular the screening and odour control, have had significant impacts on works operations, works cleanliness, and reduction of odours since 2005. DCC and the CAW staff should be proud of the accomplishments.

Even though the works has experienced pollutant loadings in excess of its design year loadings, it has performed consistently well with regard to biological treatment and effluent disinfection. Effluent total suspended solids are not in compliance with standards, however, and the sludge handling system does not achieve its goals for producing dried biosolids.

The main concerns noted during the visit were housekeeping and redundancy issues. Housekeeping includes keeping doors closed in addition to keeping the works neat, clean and picked up. True redundancy of equipment and processes is extremely limited. Many of the works facilities do not have sufficient redundancy. This results in reduced capacity of those unit processes during scheduled and unscheduled maintenance activities.

Other significant areas of concern are:

- The operating level on the top deck sequencing batch reactors (SBRs) has been reduced by approximately 1 meter to minimize turbulence due to wind effects
- There is insufficient thickening capacity for the surplus activated sludge (SAS)
- Co-thickening of SAS and primary sludge has reduced removal efficiencies in the primary clarifiers
- There are no back-up centrifuges for dewatering prior to the thermal hydrolysis process (THP)
- Due to safety and odour control issues, an entire THP stream must be removed from service to work on a single reactor
- Return liquor load is quite significant particularly from the dewatering of hydrolysed digested sludge.
- Rated capacity of each digester has been reduced from 33 dT/day to 30 dT/day based on operational experience
- There are no back-up centrifuges for dewatering prior to sludge drying
- Dryers have not achieved their design intent or capacity, requiring continuous production of Class A Biocake from hydrolysed digested sludge and frequent production of Class A limed biosolids.

# 2 Background Information

The Ringsend facilities, including modifications made since the commissioning of the Works in March 2005 are presented in this Ringsend Wastewater Treatment Works Extension Baseline Report.

This report is organised to include the following subjects:

- Basis of Design
- Influent Analysis
- Effluent Analysis
- Solids Process Analysis
- Functional Process Areas, including:
    - Headworks
    - Primary Settling
    - Intermediate Pump Station
    - Flow Splitting Boxes (flow distribution to SBRs)
    - Sequencing Batch Reactors
    - Ultraviolet (UV) Disinfection
    - Storm Tanks
    - Flow Metering
    - Solids Processing
    - Sludge Dryers
    - Odour Control





CDM

22825-67511-RT. BASE

# 3 Basis of Design

The parameters listed below constitute the Basis of Design for the existing facilities as put forth in Volume 1 of Dublin Bay Project Contract 2:

| **Description** | |
|---|---|
| **Average Daily Flow (ADF)** | 5.7 $m^3$/sec |
| **Flow to Full Treatment** | 11.1 $m^3$/sec |
| **Peak Instantaneous Flow** | 23.0 $m^3$/sec |
| **Influent BOD load** | |
| Average | 98,400 kg/day (200 mg/L)[1] |
| 95 Percentile | 156,700 kg/day |
| **Effluent BOD** | |
| 95 Percentile | 25 mg/L |
| Not to be Exceeded | 50 mg/L |
| **Influent COD load** | |
| Average | 225,100 kg/day (445 mg/L)[1] |
| 95 Percentile | 383,300 kg/day |
| **Effluent COD** | |
| 95 Percentile | 125 mg/L |
| Not to be Exceeded | 250 mg/L |
| **Influent TSS load** | |
| Average | 101,100 kg/day (205 mg/L)[1] |
| 95 Percentile | 194,300 kg/day |
| **Effluent TSS** | |
| 95% Percentile | 35 mg/L |
| Not to be Exceeded | 87.5 mg/L |
| **Influent Nitrogen load** | |
| Total N - Average | 15,600 kg/day (31.7 mg/L)[1] |
| Total N - 95 Percentile | 21,400 kg/day |
| Ammonia N – Average | 9,500 kg/day (19.3 mg/L)[1] |
| Ammonia N – 95 Percentile | 12,800 kg/day |
| **Effluent Ammonia Nitrogen** | |
| 95% Percentile | 18.75 mg/L |
| Not to be Exceeded | 37 mg/L |
| **Influent Total Phosphorus** [2] | |
| Average | 3,700 kg/day (7.5mg/L)[1] |
| 95 Percentile | 5,600 kg/day |

*1. As computed from ADF*
*2. No limits were placed on effluent Total Phosphorus in Contract 2.*

# 4 Influent Analysis

The Ringsend WwTW is operating under stressed conditions. As may be seen from Figure 1, the influent BOD load has always exceeded the design loading. Please note that one population equivalent (PE) is defined as 60 g/day BOD. Thus the design loading of 98,400 kg/day is equivalent to 1.64 million PE. There was a fairly steep drop off in loading at the end of 2006 has not yet been explained and has only partially returned. This phenomenon warrants further investigation as part of the determination of design flows and loadings for the extended works.

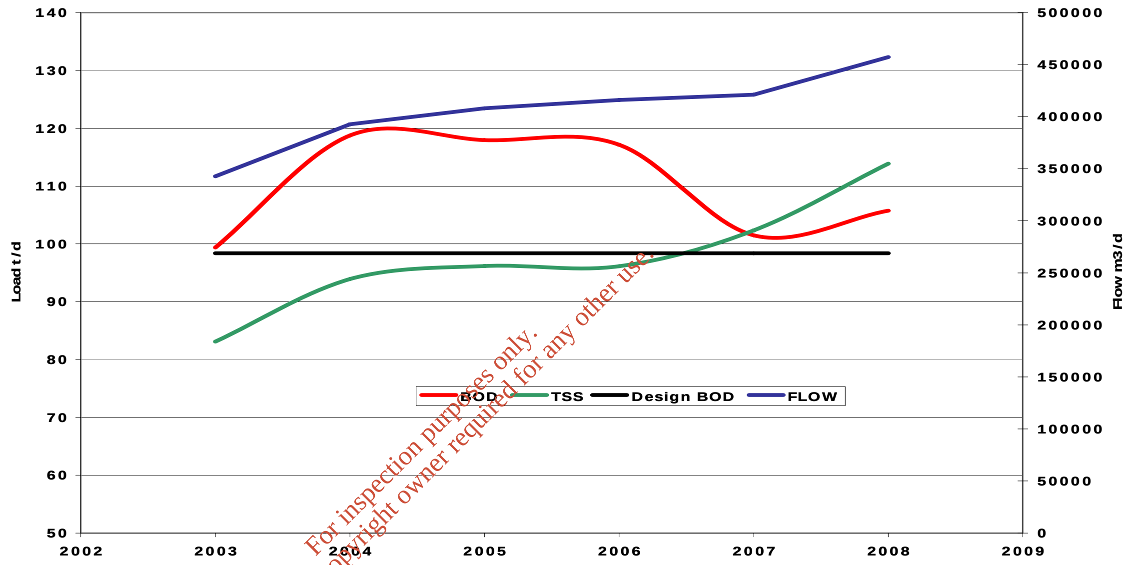





***Figure 1.*** *Influent Flows and Pollutant Loadings 2003 to Present*

Influent TSS loadings have increased continuously over this period with a steeper increase in loadings beginning in 2006. The average daily TSS loading exceeded the design year loading in early 2007 and is currently about 15% higher than the design. The steepness of the increase in TSS loadings is of concern.

Influent flows have also increased continuously, but at a lower rate than TSS. The current average daily flow rate is approaching 95% of the design ADF.

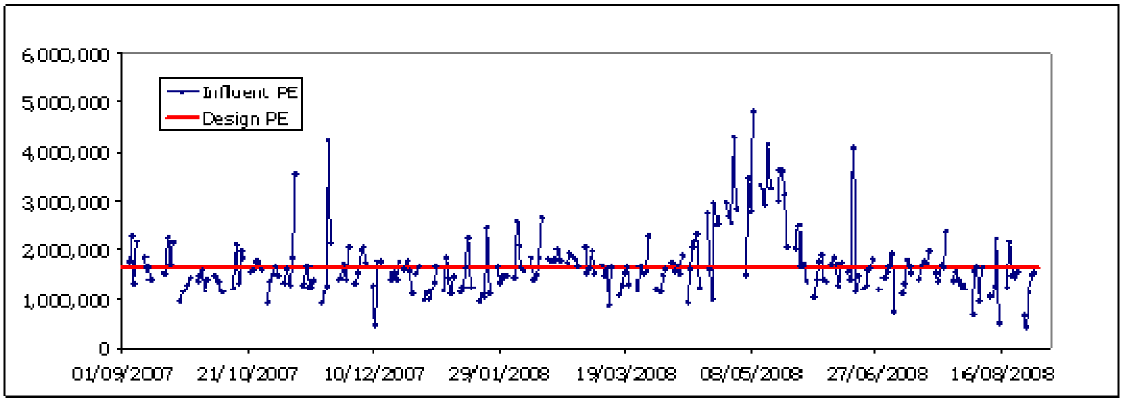



***Figure 2.*** *Influent BOD, expressed as PE, September 2007 through August 2008*

Figure 2 displays the more recent trend regarding influent BOD loading, covering the period of September 2007 through August 2008. During this period the average PE was 1.7 million as compared to the design PE of 1.64 million, or 104% of the design value. The maximum day, week, and month values were 4.83 million, 3.39 million, and 3.14 million, respectively.

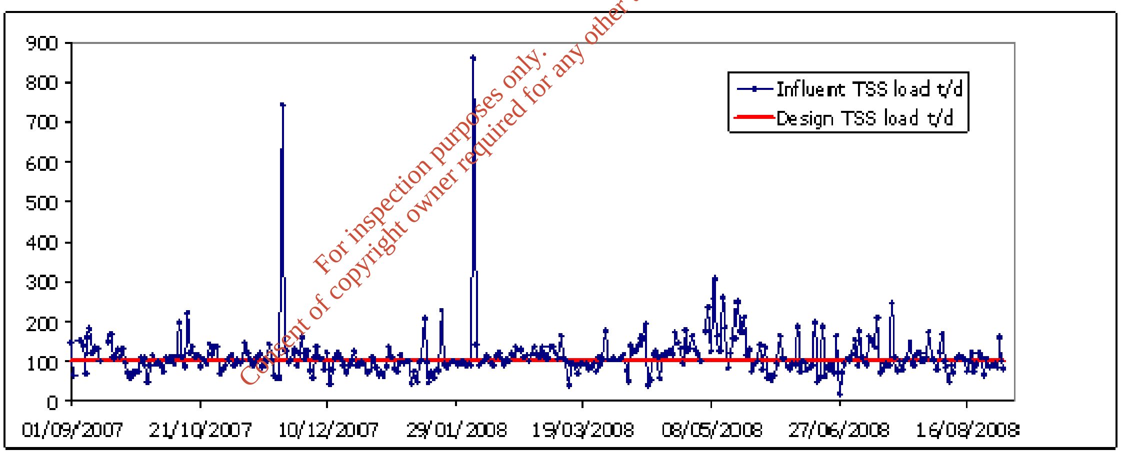





***Figure 3****.* *Influent TSS Load t/d, September 2007 through August 2008*

Figure 3 displays the influent TSS loading over the same time period as Figure 2. These loads are even more variable. During this period, the average influent loading was 113 tonnes per day (tpd) as compare to the design loading of 101.1tpd, or 112% of the design value. The maximum day, week, and month values were 860 tpd, 216 tpd and 161 tpd, respectively.

Year 2007 data, taken from the EPA returns shows that influent nitrogen species exceed the design basis. Total Kjeldahl Nitrogen (TKN) averaged 40.9 mg/L and Ammonia Nitrogen ($NH_3$-N) averaged 26.8 mg/L. Since there is essentially no inorganic nitrogen in the influent, TKN is equivalent to Total N. Thus, influent concentrations of Total N and $NH_3$-N were 129% and 139% of design average values, respectively.

Year 2007 data, also taken from EPA returns, show that Total Phosphorus (TP) averaged 4.6 mg/L as compared to a design average concentration of 7.5 mg/L.

# 5 Effluent Analysis

The works has demonstrated the ability to adequately remove BOD and has often performed exceptionally well under stressed conditions.

Over the period of 1 July 2007 through 31 August 2008, the average effluent BOD concentration was 15 mg/L as compared to the 95%-ile standard of 25 mg/L. The average BOD removal rate was 93.2% and 99.2% of the flow received full secondary treatment. The effluent achieved compliance with the effluent standard of 25 mg/L 90.8% of the time. While not achieving the required 95%-ile compliance rate, 69% (18 of 26) of the exceedances occurred during days on which the influent loadings exceeded the design basis. There were seven days when the effluent exceeded the not-to-exceed limit of 50 mg/L.



There was one 30-day period between 24 April 2008 through 23 May 2008 in which the average BOD loading to the works averaged 188.3 tpd (3.14 million PE), or 192% of the design basis. During this stressed period, the biological treatment system performed exceptionally well, achieving an average effluent concentration of 14.4 mg/L with no days in excess of 25 mg/L. The overall BOD removal rate was 97.0% and all of the influent flow received full secondary treatment.

Another high stress month was August 2008, during which time the works saw very high flows, averaging 6.7 $m^3/s$, or 17% higher than the design ADF. The maximum day flow was 13.2 $m^3/s$ and the peak instantaneous flow was 21.0 $m^3/s$. During this period, 99.6% of the flow received secondary treatment. The average effluent BOD was 7.0 mg/L and there were no days in excess of 25 mg/L. The overall BOD removal rate was 97.0%

TSS removal is not on par with BOD removal. Over the period of 1 July 2007 through 31 August 2008, the average effluent TSS concentration was 30.1 mg/L as compared to the 95%-ile standard of 35 mg/L. The average TSS removal rate was 87.8%. The effluent achieved compliance with the effluent standard of 35 mg/L 81.4% of the time. There were seventeen days when the effluent exceeded the not-to-exceed limit of 87.5 mg/L. These exceedances correlate better with high influent loading than with high inflow, with fifteen days exceeding the design average influent TSS loading and eight days in which the inflow exceeded the design ADF. There were seven days in which both the design influent flow and TSS loading parameters were exceeded.

During the high load period of April-May 2008, the effluent averaged 35.6 mg/L, with 13 exceedances of the 35 mg/L standard. During the high flow period in August 2008, the effluent averaged 18.6 mg/L and there was only one exceedance of the 95%-ile standard. The average removal rate was 90.0%. During the stressed months it again appears that influent loading has a greater influence on effluent quality that influent flow.

Another correlation that is apparent is the sludge volume index (SVI) and effluent TSS. SVI is not measured every day, so direct day-for-day correlations are difficult to obtain. However, over the last 12 month period, there were 77 exceedances of 35 mg/L and almost 60% of them occurred when the SVI exceeded 150 ml/g.

In 2007, effluent Total-N and $NH_3$-N averaged 22.1 mg/L and 4.6 mg/L, respectively. The works is reliably achieving its ammonia limit of 18.75 mg/L as required by the Contract between DCC and CAW. The works is not currently meeting the 10 mg/L Total-N Urban Waste Water Treatment (UWWT) limit for Nutrient Sensitive Waters.

In 2007, the effluent contained an average of 3.6 mg/L TP. The UWWT limit is 1.0 mg/L. There is no requirement in the Contract between DCC and CAW to remove P.

Disinfection is required from 1 May through 31 August, annually. During this period, the standard is 100,000 faecal coliforms per 100 ml (FC/100 ml) and 80% compliance must be achieved over an 8-week rolling average. Both laboratories performing bacteriological analyses for the works (i.e. Central Labs for DCC and City Analysts Ltd. for CAW) had difficulty providing reliable and reproducible faecal coliform results. Discussions centred on the appropriate bacteriological standard took place in 2006 between senior microbiologists from both labs, DCC and CAW. Water-quality studies indicating excellent correlation between Escherichi coliform (E. coli) and faecal coliform were cited. It was also noted that in 1986, the U.S. Environmental Protection Agency (USEPA) recommended that E. coli be used in place of faecal coliform bacteria in State recreational water-quality standards as an indicator of faecal contamination. As are result of these discussions, DCC and CAW agreed to monitor E.Coli instead of faecal coliforms from 1 May 2006 forward. Since that time, the works has always been in compliance with the revised standard.



# 6 Solids Process Analysis

The solids handling system has been in a state of flux since the works were accepted by DCC in 2005. Over this period there have been several modifications to the solids processing system that have impacted availability and reliability of system components. Rarely have all three dryers been available for operation. DCC's original intent was to have all of its sludge dried to Class A standards with the product, labeled Biofert, used agriculturally. For several reasons, this goal has not been achieved and the works has produced Class A biosolids (Biocake) from dewatered, thermally hydrolyzed, digested biosolids and Class A limed biosolids by liming dewatered raw sludge.

Figure 4 displays the monthly production of Biofert, Biocake and Class A limed cake from May 2005 through August 2008 in tpd of dry solids. As may be seen, the dryers have never been able to handle all of the sludge produced and Class A limed cake has had to be produced to make up for the shortfall in Biofert and Biocake production.

The last of dryer improvements have recently been completed on Dryers A and B. The final safety (ATEX) upgrades are being installed on Dryer C. Once completed with two operating units will be operated full-time with one standby unit. Long-term dryer performance can begin to be assessed.

The THP/digestion system has been very successful in destroying volatile solids and producing biogas.

DCC has recently awarded a contract to expand the sludge stream. This work entails:

- Installation of a third THP stream
- Installation of a fourth digester
- Installation of three SAS thickeners (bringing the total to six)
- Installation of a dual gas fuelled steam boiler

Once this work is completed, the maximum capacity of the THP/digestion system is expected to be 120 tpd. Actual loadings will be lower, but since the dryers cannot accept 100% digested product and dryer throughput will be no more than 60 dT/day, some Class A cake will be produced. Ultimately, DCC's goal is to eliminate Class A cake production.



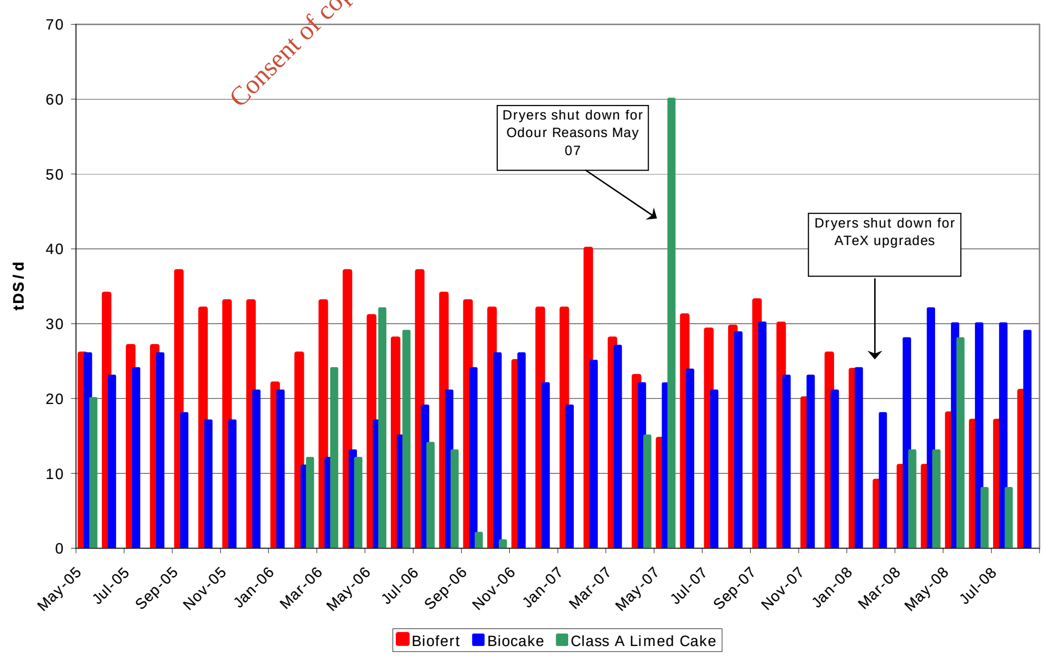



***Figure 4**. Monthly Sludge Production by Type*

# 7 Functional Process Areas

## 7.1 Headworks

The headworks facilities collect and consolidate the flow from four main sources: the Sutton Pumping Station; the West Pier Dun Laoghaire Pumping Station; the Main Lift Pumping Station (MLPS); and the Dodder Valley Gravity Sewer. After consolidation the flow is directed to the screening area.

The 6-mm perforated screens originally installed at the works were replaced with 6-mm wedge wire bar screens. These screens are raked on the basis of wastewater level control. The replacement was made because the perforated screens would blind with the combination of grease and fibrous material and cause the system to back up, creating occasional overflows. Due to the hydraulic problems, one of the perforated screens was removed. This resulted in significant solids being conveyed to the downstream facilities. The modifications to install the wedge wire screens in all of the screen bays were completed at the end of 2007 and the improvements have resulted in no bypasses of the screens since that time. The screening facilities are designed to handle a peak wet weather flow of 23 $m^3/s$.



Screenings are dropped into individual channels for each screen and flushed through the channels loaded directly into to compactors. The originally installed macerators were removed in 2007 and replaced by Wash Pactors due to significant problems with blockages and reliability during high flows. The compacted screenings are placed in enclosed skips for disposal.

Screened flows are pumped by screw pumps into the grit removal tank feed channel.

The FOGG (Fats, Oils, Greases and Grit) tanks when first commissioned did not effectively remove FOG (Fats, Oils and Greases). The first modification was to install a baffle within each tank to assist in FOG removal. The baffles were unsuccessful in aiding in FOG removal and were subsequently removed. Since then the FOGG tanks have been operated as aerated grit removal tanks. The original submersible grit pumps have been replaced with recessed impeller pumps located exterior to the tanks. The tanks now achieve improved grit removal.

The removed grit is then sent to cyclones and classifiers. The dewatered grit is dropped into open skips for disposal.

## 7.2 Primary Settling

The primary settling tanks, which have lamella packs to facilitate sludge settling, can meet the existing flow requirements. However, the actual capacity is largely an unexplored issue and it is unclear whether they can meet future flows. The intent is to expand the primary settling with additional lamella packs to the tanks. During commissioning, the grit tanks, channels and primary settling tanks were tested at the (then anticipated) ultimate design flow of 13.6 $m^3/s$, but that was without co-settling.

A portion of the Surplus Activated Sludge (SAS) is returned to the lamella clarifiers for co-thickening. This is done primarily due to limitations on SAS thickening (i.e. insufficient thickeners). The sludge stream expansion project will double the number of rotary drum thickeners from three to six. This will result in a significant reduction in the amount of sludge that is co-thickened. At this time, approximately 1/3 of the SAS is processed through thickening and 2/3 is processed through co-thickening in the primary settling tanks. With the increase in thickening capacities, the hope is co-thickening will be a maximum of 1/3 of the SAS. It is clear that further upgrading of SAS thickening will be required for existing as well as future flows.

Prior to the initiation of co-thickening removal efficiencies in the primary clarifiers were reportedly approximately 50% of TSS removed and 35% of BOD removed. The removal efficiencies have been reduced since co-thickening has been incorporated. Currently primary influent quality is not measured and so the actual removal rates cannot be accurately assessed. In addition to the raw influent, recycles from solids processing and tankered wastes increase the loadings to the primary tanks. Removal rates are, therefore, higher than the "apparent" removals computed by comparing the raw influent to settled effluent. The "apparent" removal rates for TSS and BOD in 2007 were 27% and 16%, respectively. If one assumed additional loads would account for a 20% increase in primary influent loadings, the removal rates would increase to 39% and 30% for TSS and BOD, respectively. It has been noted that co-settling has some beneficial effects. The lamella plates remain cleaner and the bottom scraper is less stressed as compared to conventional primary treatment. It may be advantage to retain a small degree of co-settling. However, if the primary clarifiers are to achieve their highest potential efficiency, co-thickening must be curtailed.



A sampling program primary influent and effluent quality should be undertaken to determine what the actual loadings and removal rates are across the primary clarifiers. This data is needed to better understand the works' current mass balance as well as to determine what the likely removal capacity will be under future flows and loadings.

The recently completed upgrade project improves scum removal, as well as odour capture and control. Prior to this project, there was one scum removal bridge for each bank of six clarifiers. The single bridge had difficulties in tracking and transferring from one tank to the other and this resulted in the scum accumulating in the tanks with frequent necessity to remove it by vacuum tanker. The new system has one bridge for each settling tank, discharging scum to a scum trough at the end of each clarifier. The scum is discharged to both sludge holding tanks.

It was found that the scum caused difficulties with the belt filter press (BFP) dewatering as it blinded the belts. Since the implementation of the centrifuges, the FOG has reportedly not caused problems.

Examination of monthly reports indicates that primary sludge pumps require frequent maintenance.

## 7.3 Intermediate Pump Station

The intermediate pump station (IPS) is comprised of four high lift and four low lift submersible pumps. The original installation had difficulties with motors overheating. As a result, the independent cooling jackets were removed from the pumps and modifications made to the motors. This has resolved the overheating issues with these pumps. It was noted that there have been intermittent problems with the electrical gear overheating as well, although these problems seem to have been resolved.

## 7.4 Flow Splitting Boxes

The Intermediate pump station lifts the primary effluent into distribution boxes at two levels for splitting in three SBR units on each level. The accuracy of flow splitting is thought to be adequate. There is an additional chamber in each distribution box to split to a fourth SBR bank on each level.

## 7.5 Sequencing Batch Reactors (SBRs)

The SBRs were originally installed with the intent to operate as typical SBRs. During the initial operations there was an issue with filamentous growth, which could not be controlled by the operators. This growth resulted in a problem with decanting and sludge being in the decant stream. During investigations into these issues, the operations team attempted a pre-react mode (anoxic selector) and dosing of disinfectant (chlorine) to kill the filamentous bacteria. Neither attempt was successful. The operations staff implemented a continuous inflow, constant level (CICL) mode of operation of the SBRs with increased aeration to overcome the filamentous problems.



SBR operations have also changed in terms of the operating level on the top deck SBRs. The original operating level resulted in significant turbulence and unbalanced effluent weir loading due to wave action in the basins caused by the wind. As a result, the basin operating level has been reduced by approximately one meter. This has reduced the overall active volume of the SBRs by about 10% and the lesser depth probably contributes to solids carry over, especially during high flow or influent loading conditions. Even with the lowering of the water level wind on the top deck still causes problems with settling and certain non-compliant effluent results can be attributed to severe wind chop particularly in the winter months and in the SBR basins 3B and 3C, which are most exposed.

The performance of the SBRs in CICL mode has been compliant with effluent quality with regard to BOD, but not TSS. There are likely a number of factors that contribute to effluent TSS quality, such as co-thickened primary effluent, high recycle rates from solids processing, and occasionally high SVIs, in addition to the lowered water surface on the upper level. All of theses issues need to be addressed and remedies sought.

It has been reported that high the air demand of SBRs places significant pressure on SBR blowers and the air diffusion grids in the basins.

If the SBRs were to revert to a traditional operating mode, penstocks would have to be reactivated and programming would have to be reinstalled.

## 7.6 Ultraviolet (UV) Disinfection

As previously noted the UV disinfection system is operated on a seasonal basis and has performed well in terms of bacteriological kill. There are maintenance issues of concern, however.

Flow strainers prior to the banks of lamps that also function as screens protecting UV lamp modules from small screenings discharged with final effluent. As the screens on individual UV channels block with screenings at different rates, the flow between channels varies leading to different hydraulic loads to the individual channels, different retention times and, therefore, reduced received UV dose on channels with higher hydraulic load. Blocked screens can also increase the level in upper effluent channel and this causes occasional discharge of the final effluent through emergency overflow, bypassing UV plant. There is no equipment installed to facilitate the lifting and cleaning of the screens on UV plant . Cleaning currently takes several hours of manual operation by the operators.

As part of this project, CDM will consider installing additional UV capacity with the objective of further reducing E. coli thereby providing even greater safeguards to bathing areas than are required.



## 7.7 Storm Tanks

During periods when the perforated screens were in service, there were several problems reported at the storm tanks. There was debris on the mixers, significant odor problems, and debris catching on any equipment. Since the headworks improvements these problems have been eliminated.

The existing storm flow retention basins comprise a volume of 62,100 $m^3$ and overflow directly to the Liffey estuary portion of Dublin Bay. The overflow discharge point is separate from the works outfall. In addition to the overflow from the storm tanks, there is a default overflow from the primary settling tanks that is available should there be a system fault or power failure affecting th IPS or SBRs. There is a record of all overflows from the lamella tanks. At the capacity to hold up to 90 minutes of flow in excess of the maximum flow rate into the treatment system, the maximum nominal flow to the storm tanks is, theoretically, 11.5 $m^3$/sec.

The allowable storm tank overflow is limited to 3,000,000 faecal coliform/100 ml. No halogenated compounds (chlorine or fluorine) are allowed to be used as a disinfectant on the storm water overflows. (see Table 2.3.2 of the original RFT).

Storm water collected during storm events must be returned into treatment in shortest possible time to prevent odour release and restore storm water treatment capacity before next storm event starts or contaminated influent requires diversion into storm tanks.

The only facility for cleaning of these storm tanks is mixers in the bottom of each tank and operational experience shows that installed mixers are prone to blocking and entanglement with debris. There is no other facility for cleaning of storm tanks other than these mixers.

## 7.8 Flow Metering

Influent flow is measured on the flow from the Main Lift Pumping Station, the Sutton Pumping Station, and remotely from the Dun Laoghaire pumping station. There is also a flow meter on the Dodder Valley gravity sewer; however, the accuracy of this meter is questionable. The flow to and returned from the storm tanks is measured.

Flow is measured on each of the main channels from the grit removal tanks into the lamellas. There are also flow meters on the pumped flow pipelines to each SBR at the intermediate pump station.

The final effluent channel also has a flow meter consisting of a level sensor upstream of a flume.

## 7.9 Solids Processing



Some of the Surplus Activated Sludge (SAS) is returned to the lamella clarifiers for co-thickening. This is done primarily due to limitations on SAS thickening (i.e. insufficient thickeners). An upgrade is being initiated to increase the number of drum thickeners available. This will result in a significant reduction in the quantity of SAS that must be co-thickened.

As part of the upgrade/expansion of SAS thickening, a dry polymer bulk system is being installed to replace the existing unreliable liquid polymer system.

The sludge holding tanks associated with the primary settling system receive primary co-thickened sludge and scum from the primary settling tanks.

The SAS holding tanks receive sludge from the SBRs. Thickened SAS from the drum thickeners is held in a partitioned portion of one of the sludge holding tanks.

All of the co-thickened sludge is screened through sludge screens and discharges to the buffer tanks. The buffer tanks discharge to the centrifuges for dewatering. The centrifuges were installed as a temporary system to reduce reliance on the BFPs. The two Westfalia CA755 centrifuges are located outside adjacent to the THP building. The location of these centrifuges results in some uncaptured odour release, but this is a minor problem as compared to the odours from the BFPs. The centrifuges dewater the sludge to approximately 20% solids. The un-dewatered sludge is then blended with the dewatered sludge to produce 15% total solids sludge prior to the sludge thermal hydrolysis (THP) system.

The BFPs have been relegated to a standby mode and only operate when one of the centrifuges is not operating and process demands dictate a higher quantity of sludge dewatering than a single centrifuge can maintain. There are five 3-meter

BFPs available; however, they have not been recently operated. During the period these BFPs were operated, problems were noted with blinding of the belts from rags and grease. Odour was also a significant issue when operational the BFPs were in operation.

The THP system consists of two streams. Each stream has two pulping tanks, four reactor tanks and one flash tank. The THP system is operated on a batch basis, which means as sludge is withdrawn from the first pulper tank it is filled to its highest level. Each stream is theoretically capable of up to 50 cycles per day, or 100 cycles per day, total. One cycle consists of flow through each of the pulping tanks, one reactor tank, and the flash tank.

Operation of the THP system includes increasing sludge temperature in the first stage pulper using low-pressure waste steam from the flash tank. Sludge in the second stage pulper is heated using high-pressure waste steam from the four reactors. Sludge in the reactors is heated to 165ºC by steam provided by steam generators. The pressure in the reactors is increased to 6-bar for 30 minutes, after which the pressure is reduced to approximately 3-bar and the sludge is ejected to the flash tank. Off-gas vented from the reactors is condensed and sent to the digesters. The pressure in the flash tank is reduced to approximately 1.2 bar and sludge is sent to the heat exchangers.



The hydrolysed sludge is mixed with sludge from the digesters to reduce the solids concentration from approximately 14% to approximately 12%. The reduction in solids concentration is necessary to prevent plugging of the heat exchangers. The hydrolysed sludge is mixed with recycled digested sludge at the ratio of about 3 parts digested sludge (at approximately 5.5% TSS) to 1 part feed sludge. The sludges are combined to prevent plugging of the heat exchanger tubes through increased velocity, dilution, and a pH more favourable to dissolving fat deposits on tube walls. The combined sludge is cooled to approximately 42ºC before being fed to the digesters.

In the past year, the THP system has averaged 78 cycles per day with a variance of +/- 3 cycles on a monthly basis with each cycle conditioning approximately one tonne of sludge. The reduced number of cycles compared to design maximum are attributed to several factors, including poor mixing in the first stage pulper, shutdowns for maintenance, and sludge availability. An average target of 75 cycles per day was accepted in the Taking-Over Certificate to take into account down time and other factors. The current Sludge Stream Expansion project will increase maximum output to approximately 120 per day, which will match up with the expanded digestion capacity of 120 DT/day.

Modifications to the THP sludge processing system have included installation of wear (316SS) plates in the flash tanks, bursting discs have been installed on de-pressurization lines, and off-gas is now compressed and conveyed to the sludge discharge line to the digesters. Sludge transfer pumps were also modified with a more heat resistant stator material and then subsequently upgraded. The initial problems experienced with odors have been largely resolved and the THP system is generally operating as designed.

According to CAW operators that there have been issues with the THP related to operations and required maintenance in the reactors. The reactors build up a film on the walls, which will slough off, become lodged in and block the fill and draw piping. This results in having to shut down the reactors to fix. Since all reactors must be shut down, this eliminates half of the sludge processing capability during these periods. Isolation of individual vessels is being addressed as part of the Sludge Stream Expansion project. There is currently no mixing in the first stage pulper, which can result in minimally heated sludge being transferred to the second stage pulper. Solids from the THP system are sent to the digesters using progressive cavity pumps.

There have been no significant modifications to the digesters. The three 4,000 $m^3$ digester tanks operate in parallel as complete mix mesophillic digesters. Mixing is accomplished using single top entry agitators. Each digester is fed sequentially on a timed cycle. During initial operations there was "burping" caused by overfeeding from the THP system. The original design of the digesters was for a 15 day SRT and each digester had a rated capacity of 33 DT/d; however, operational experience has resulted in de-rating each digester to 30 DT/d.

It was noted that gas production and solids destruction have been successes at the works. Volatile solids destruction efficiency in digesters averaged 56% in over the last year. The ability of the THP/digester system to produce a Class A sludge has been invaluable in compensating for difficulties experienced with the sludge driers. Additionally, the energy generated from the THP and digester off-gassing has provided up to half of the electricity needed to operate the works. Biogas-generated power has averaged 40 MW-hr/day over the last year.



There are three buffer tanks available following digestion. One of the tanks processes to the dewatering centrifuges, one is to the dryers and the third is a blending system installed in 2006/2007.

The intent for subsequent processing is to dry as much as possible. If the feed to the dryers (digested and TSAS) exceeds capacity at any time, digested sludge is directed to the available dryer centrifuge to produce Class A Biocake.

As determined during the visits, the current conditions of sludge operation are:

- Approximately 1/3 of the SAS is processed through rotary drum thickening, and 2/3 is processed through co-thickening in the primary settling tanks. With the increase in thickening capacities, the hope is co-thickening will be a maximum of 1/3 of the SAS, with a goal of eliminating co-thickening. Most of the co-thickened sludge is processed through the THP/digester combination. Some of the THP/digested sludge is dewatered and directly transported for agricultural uses. The remainder of the THP/digested sludge along with the TSAS and undigested portions of the co-thickened sludge is combined in the Buffer Tanks ahead of the centrifuge dewatering units preceding the dryer prior to agricultural use.

- The current target feed makeup to the dryers is comprised of about 40% TSAS at 2 to 3% TS content and 60% THP/digested sludge at 5% to 6% TS. The sludges are mixed in the Buffer Tanks ahead of the centrifuges that are integral to each of the drying system streams.

## 7.10 Sludge Dryers

The sludge dryers have proven to be very difficult to operate properly and have not met their rated capacity. The dryers have had significant problems with dust production and the inability to granulate when provided with the full stream of THP/digested sludge. The target ratio between TSAS and THP/digested sludge is not fixed. The operator continues to optimise it. However, it is clear that there is no scenario in which 100% digested sludge or 100% TSAS can be fed to the dryers.

The sludge dryers are operating at approximately 75% of their rated capacity or 30 DT/d per dryer. This limits sludge production to 60 DT/d when two units are in service, which is the desired mode of operation.

The dryers have frequently been out of service due to the need to clean filters amongst other maintenance issues. The filters used to control dust on these dryers (Swiss Combi) are bag houses. The bag houses require frequent shutdowns to remove the dust. Further to ATEX upgrading, the automatic "hot" shutdown procedure entails emptying the drum, conveyors, etc. of hot product either through the system proper or through an emergency discharge conveyor. Recovery from hot shutdowns reportedly requires approximately 6 hours. Odours are emitted until such time as the product is removed from the building.



While the dryers have been out of service or at reduced capacity, the facility has had to haul approximately 3,000 to 3,500 tons per month of centrifuge dewatered solids. The expectation is that once all dryers are back on-line (two in service one standby) this hauling requirement will be significantly reduced. Cake tonnage was halved in October, which is very promising.

## 7.11 Odour Control

DCC and CAW have aggressively attacked the odour problem from both capital and operational fronts and should be commended for their efforts.

With regard to capital projects, the following capital projects have been undertaken since the 2005 odour survey.

- Increased headworks odour control capacity
- New channel covers and odour control units (OCUs)
- New IPS covers and OCU
- New off-gas compressors for THP system
- New primary clarifier covers and OCUs
- Upgraded dryer combustion chambers

- New biogas scrubbers upstream of the CHP plant

Operational improvements have stemmed from enhanced vigilance on the part of CAW staff and frequent maintenance. A full-time odour control technician was appointed in early 2007.

Odour complaints in 2008 are averaging less than one half the 2007 level, with only nine complaints registered over the last three months. Most complaints in recent months have been attributed to the dryers. Decomposing algae is another source of odour, but it is sometimes difficult to get the public to acknowledge this. Now that the dryer combustion chamber upgrades have been made, odour complaints are expected to decrease further. However, fugitive odours will continue to escape the building when the dryers are being vented during hot shutdown and certain other maintenance activities. Odour capture and control for the dryer buildings must be addressed as part of the Works Extension, or preferably, beforehand.

# Appendix B

# Overview of Geology in Dublin Bay

# RINGSEND WASTEWATER TREATMENT WORKS EXTENSION
# OVERVIEW OF GEOLOGY IN DUBLIN BAY



CDM

November 2009

# Document Control Sheet

| | | | | |
|---|---|---|---|---|
| Client | Dublin City Council | | | |
| Project | Ringsend Wastewater Treatment Works Extension | | | |
| Report | Overview of Geology in Dublin Bay | | | |
| Date | November 2009 | | | |
| Project No: 67511 | | Document Reference: DG20 | | |
| Version | Author | Checked | Reviewed | Date |
| Draft 01 | Lorraine Gaston | Bob Gaudes | Dawn Keating | 17 Nov 2009 |
| | | | | |
| | | | | |
| | | | | |

# Table of Contents

# 1 Introduction

## 1.1 Introduction

The Ringsend Wastewater Treatment Works Extension Project involves the planning of further works to maximise its capacity in order to meet future needs and to comply with the Urban Waste Water Treatment Regulations.

One option being considered is the construction of a long sea outfall, which would bring the treated discharge from Ringsend 7 to 10 kilometres out into Dublin Bay. It is therefore important to understand the geology of Dublin Bay in terms of the type of rock and sediments present, and their respective depths and thicknesses.

## 1.2 Dublin Bay

Dublin Bay is a small, shallow sandy embayment on the east coast of Ireland. It is enclosed by two the headlands Howth to the north and Dalkey to the south. It is approximately 10 kilometres across the mouth of the bay and narrows to the mouth of the River Liffey which enters the Irish Sea in Dublin Bay.

A large portion of the inner bay is affected by the rise and fall of the tides, with large areas of sand and mudflats exposed at low tide.

The North Bull Island is a prominent physical feature in the Bay which developed due to sedimentation accumulation after the construction of the North Bull wall in 1821.



## 1.3 Report Outline

This report was prepared following a desk study of the geology of Dublin Bay. Detailed surveys are intended to be carried out prior to the detailed design and the construction phases.

The relevant data and documents that were utilised include:

- GSI 1:100,000 scale Bedrock Geology map, Sheet 16 (Kildare-Wicklow);
- Teagasc soil and subsoil maps;
- Depth to bedrock data and other quaternary information obtained from the Geological Survey of Ireland (GSI) Geotechnical Mapviewer from previous ground investigations;
- INFOMAR - INtegrated Mapping FOr the Sustainable Development of Ireland's MArine Resource;
- Existing geotechnical reports prepared for sites on Poolbeg Peninsula; and
- Papers relating to the geology of the Kish Bank Basin.

# 2 Bedrock Geology

Most of Dublin city is underlain by Carboniferous limestones and Dublin Bay itself is confined by the granite headland of Dalkey to the south and the peninsula of Howth to the north which is comprised of Cambrian quartzites and slates. The more easily solubilised, less resilient limestone has eroded gradually, leaving a well-defined bay. The changes in the bedrock geology are fault controlled to the south of the Bay. A large fault, known as the Rathcoole Fault forms the southern margin of the basin.

Figure 1 shows the onshore bedrock geology of the Dublin Bay area. Descriptions of the dominant bedrock formations taken from the 1:100,000 scale geological map of Kildare and Wicklow (McConnell and Philcox, 1994) are contained in the following section.

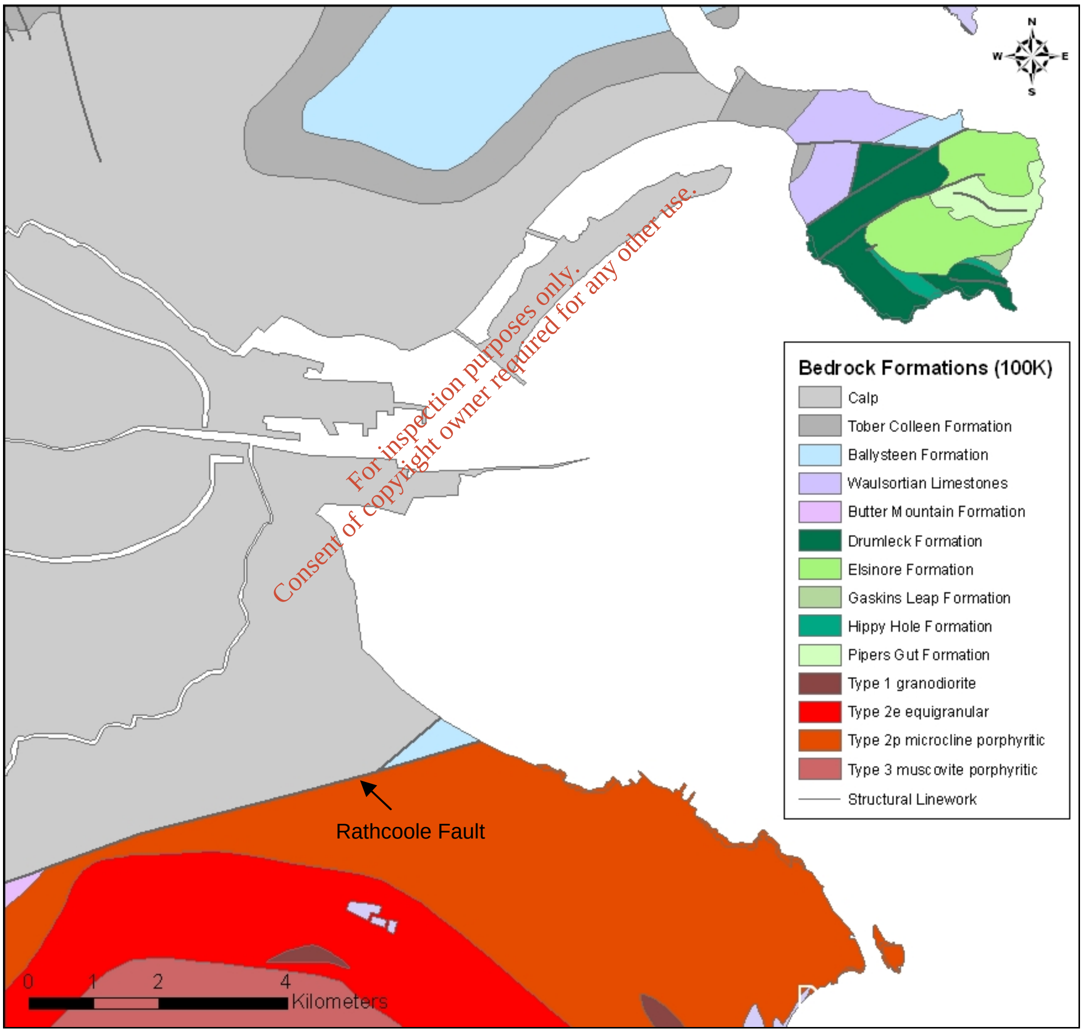




*Figure 1: Onshore Bedrock Geology of the Dublin Bay Area*

## 2.1 Bedrock Formations

***Bray Group – Howth***

Howth Head is part of the Bray Group, which are Cambrian in age, the oldest rocks in the area. The Bray Group consists of sedimentary rock where some metamorphism has occurred. These rocks include greywacke sandstones, shales and quartzites. On Howth Head the Bray Group is divided into five formations as displayed in Figure 1. These formations consist mainly of a quartzite mudstone melange and a polymict melange of quartzite and greywacke. As the rocks are hard and quartzite is not susceptible to chemical weathering, Howth Head is more resistant to erosion.

***Calp Limestone***

Much of Dublin is dominated by rocks of Carboniferous age. During the early Carboniferous period, the eastern part of Ireland underwent uplift and erosion. Following this, there was a period of general subsidence in the area. This subsidence permitted the sea to invade the lower ground from the south during the Carboniferous age. Continued subsidence resulted in shallow and then deeper marine sediments accumulating across most of Dublin City and County.

The Calp limestone (Dinantian Upper Impure Limestones), which covers most of Dublin are thick sequences of muds and muddy limestones accumulated in the basins, sometimes showing graded bedding deposited in the basins that formed. The Calp Limestone itself comprises dark grey, fine-grained, graded limestone with interbedded black, poorly fossilised shales. While the top 1m or so layer of rock is weathered, the overall mechanical strength is described as strong to very strong (Mott MacDonald Pettit, 2008).



***Ballysteen Formation***

An unconformity exists between the Calp Limestone and the Rathcoole Fault. This is a small wedged shaped section of bedrock that is part of the Ballysteen Formation, which comprise dark grey muddy limestones.

***Leinster Granite***

The Leinster Granite is a large igneous intrusion stretches from Blackrock to New Ross in County Wexford, consisting of five plutons. The Northern Pluton is present in the south of Dublin Bay. The Northern Pluton was intruded as a mobile mass piercing and rising through the crust under buoyancy called a diaper. It is a rounded body with a broadly concentric internal zonation of granite types. The granite is Type 2e, which is microcline phenocrysts (large mineral crystals). The northern limit of the granite at the surface is a fault contact with Carboniferous limestone, the Rathcoole Fault.

## 2.2 Near-shore Geology

Near-shore geology can be determined by a number of means:

- Rock outcrops along the shoreline and extrapolation of the adjacent land geology;
- Boreholes; and
- Geophysical surveys.

Most of the known information about the near shore of Dublin is from studies carried out on the Kish Bank Basin. It is located approximately 20km offshore from Dublin, in water depths of up to 100m. It comprises sandstone sealed by the overlying mudstone. Many studies have been carried out on the Kish Bank for gas exploration, sand and gravel resources and most recently for carbon dioxide storage (CSA Group, 2000).

A summary of the geology of Kish Bank Basin is presented by Dobson and Whittington (1979). It discusses the results of a seismic survey that covers the whole Kish Bank area and includes part of the mouth of Dublin Bay. Figure 2 presents their interpretation of the geology, which is further discussed below.

The granite of Dalkey Headland has been mapped (Whittington, 1977) and the outcrop of granite to the south of Dublin Bay is limited in extent as shown in Figure 2.

The heavy dashed line in Figure 2 extending from the Dalkey Fault to the mainland is an inferred fracture and the extrapolation from the adjacent geology proved difficult. Dobson and Whittington (1979) concluded that the basement (rocks below sedimentary cover which are usually not of interest as they rarely contain petroleum or natural gas) north-west of the Lambay Fault is likely to be Lower or Upper Palaeozoic in age, such as Cambrian metaphorohic rocks of Howth Head or Carboniferous limestones.

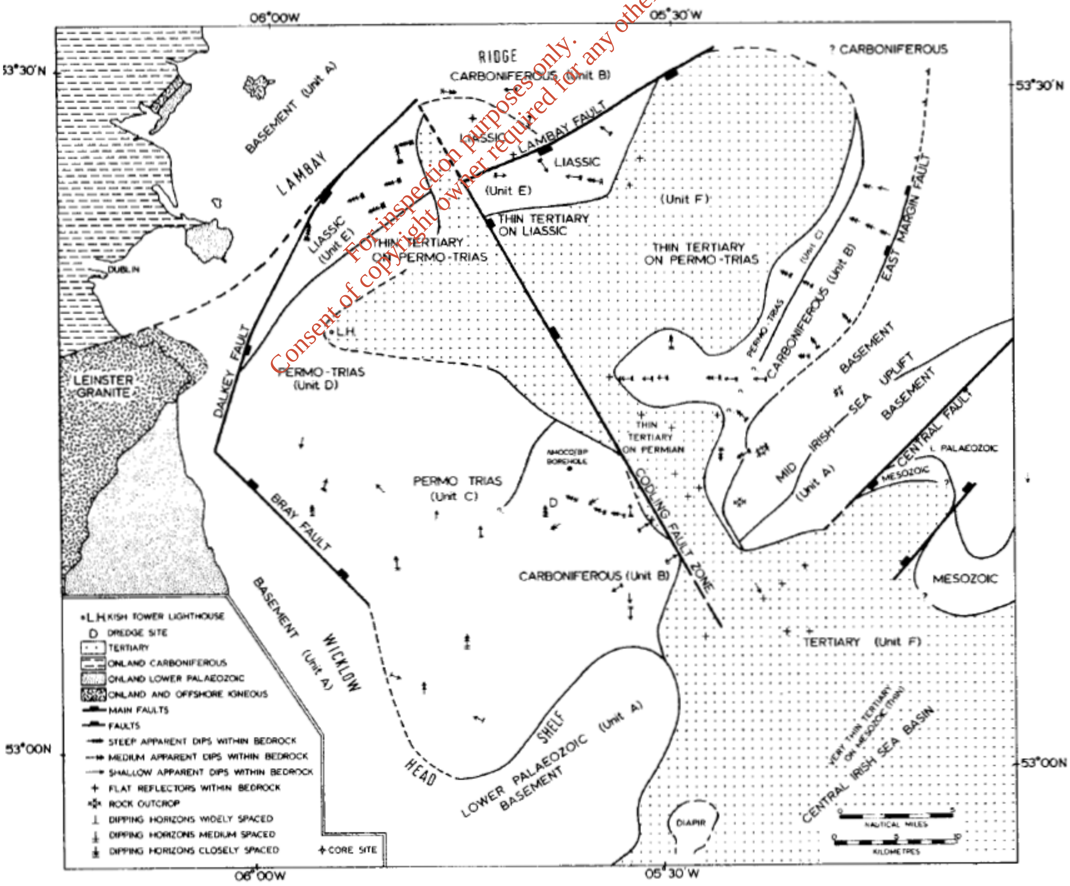



*Figure 2: Geological Map of the Kish Bank Basin (Dobson and Whittington, 1979)*



The dominant bedrock in the inner bay is likely to be the Calp limestone judging by the adjacent onshore geology. This is the more easily solubilised, less resilient limestone that has eroded gradually, leaving a well-defined bay. However it cannot be determined if there are changes in the bedrock type, as there is little available information on the structural geology of the inner bay at present and none of the existing borings within the Bay reached bedrock (from National Geotechnical Borehole Database - see Section 2.3).

## 2.3 Depth to Bedrock

The Geological Survey of Ireland (GSI) holds the National Geotechnical Borehole Database. Over 12,000 boreholes and trial pits have been georeferenced, their locations digitised for the cities in Ireland - Dublin, Cork, Waterford, Limerick and Galway. The majority of the georeferenced boreholes are in the Greater Dublin City Area, and have been used to generate a Depth to Bedrock Contour Map and a 3D Model of the bedrock topography.

Figure 3 shows the 3D graphic produced by the GSI showing the surface between overlying unconsolidated material and solid bedrock i.e. the bedrock topography.

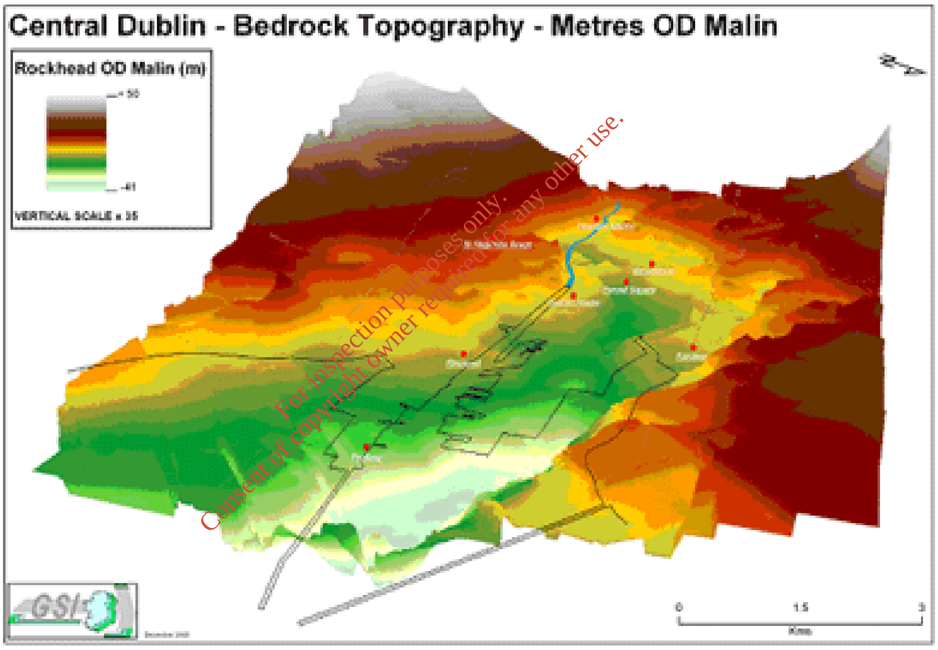





*Figure 3: Central Dublin Bedrock Topography (source GSI)*

The GSI have also developed a depth to bedrock map using the depth to bedrock values from the database in the borings; this is reproduced in Figure 4. Bedrock on the Peninsula lies between 30m and 50m below ground level. The deepest rock is in the central area with slightly shallower rock at the tip of the Peninsula. Bedrock on the Peninsula is generally not an issue for the construction of buildings as it is too deep to require excavation and also too deep for either piles or traditional foundations to bear on it.

Figure 4 also displays borings within the Bay itself. None of the existing borings have reached bedrock, so the depth to bedrock is largely unknown. The boreholes range in depth from 2m to 25m.

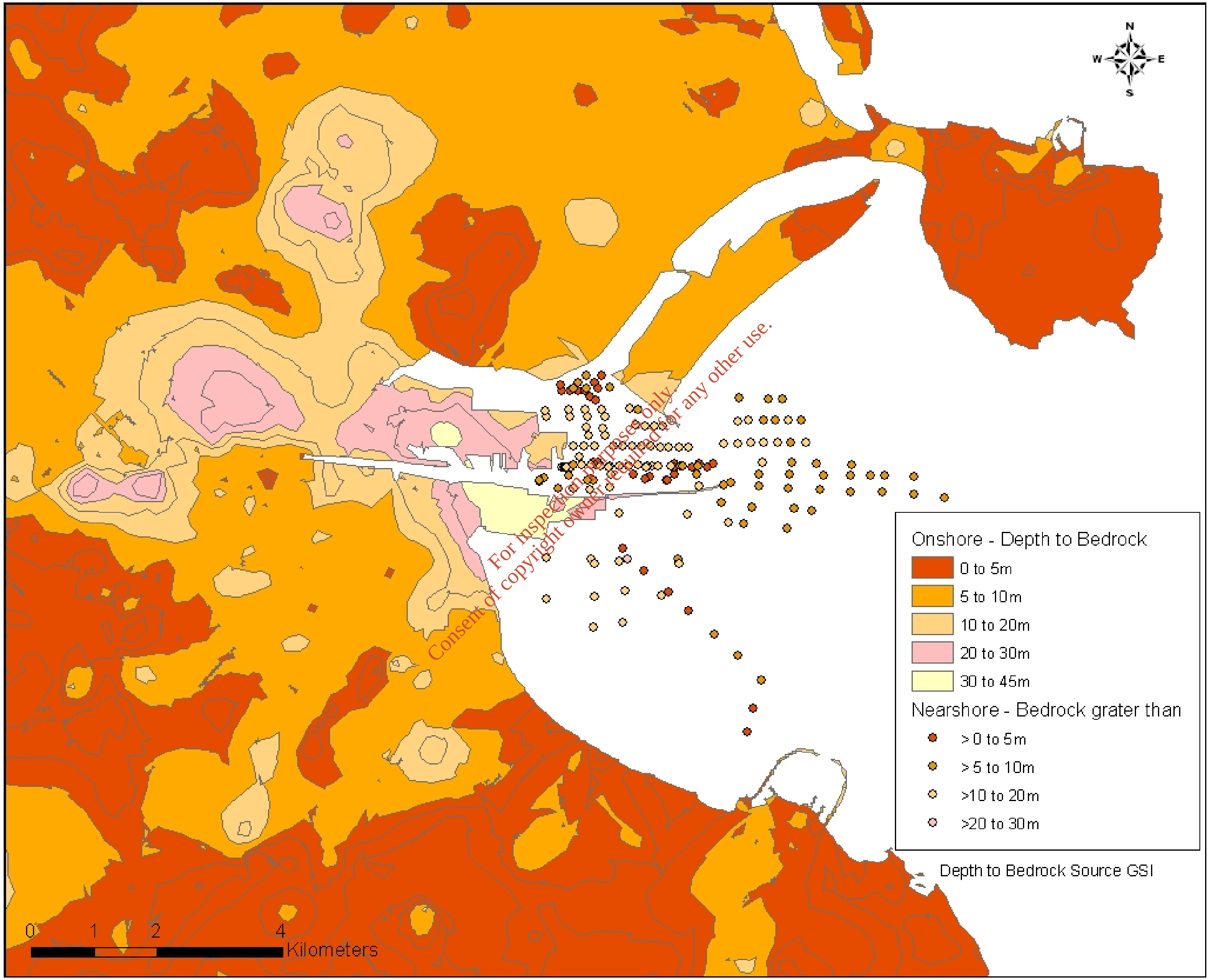





*Figure 4: Onshore depth to bedrock and nearshore bedrock is greater than value stated (source GSI)*

# 3 Sediment Cover

## 3.1 Onshore

The general stratigraphy for Poolbeg Peninsula in Ringsend has been documented as part of a geotechnical assessment carried out by Arup Consulting Engineers (2006) for the Dublin Waste to Energy Project and by Mott MacDonald Pettit (2008) for the Poolbeg Planning Scheme. Table 1 taken from the report gives an overview of the stratigraphic layers overlaying the Calp limestone at the site, which are described in more detail in the following sections.

*Table 1: Overview of General Stratigraphy of Poolbeg Peninsula (Arup, 2006)*

| Stratigraphic Divisions | | Lithostratigraphy and Genetic Classification | Principal Materials |
|---|---|---|---|
| Quaternary | Recent | Made ground (fill) | Natural earth and man made waste / made ground. |
| | | Marine (beach, estuarine and seabed) deposits | Generally mixed silts/clays and fine sands with shell fragments |
| | Pleistocene-Recent | Glacial and Fluvioglacial deposits | Generally well sorted sand and gravels, typically with some cobbles, and boulders in places. Some boulder clay layers reported in places |
| | Pleistocene | Outwash/ glacio-marine clay deposit | Slightly sandy clays with some silt and sand layers. Thicker sandy silt/clay at top in places |
| | | Lodgement till/ weathered rock | Boulders, cobbles, gravel, clay, silt |
| Lower Carboniferous | | Calp Formation | Dark grey, fine grained limestone with interbedded black shale, and locally common chert |



### 3.1.1 Quaternary

The Quaternary deposits in Dublin area are quite uniform in composition. They consist of tills derived and gravels deposited by the ice sheet from the Irish Sea Basin. The description below is taken from Mott MacDonald Pettit (2008).

The Calp bedrock is overlain outwash, glacio-marine clay which consists of over 20m of material that is stiff dark grey or black slightly sandy clay with layers and laminations of silt and silty sand overlain by silt with sand laminations.

Above this is a glacial, fluvioglacial layer which is over 10m deep of sands and gravels with occasional cobbles and boulders. This layer is occasionally silty in nature.

There is evidence that materials in this area have been modified by the typical marine processes of erosion and deposition prior to the recent period of reclamation. Overlying the drift geology, the next layer consists of marine or seabed deposits up to 2.5m thick. There is also evidence of riverine deposits from the Liffey and Dodder. This layer generally includes soft or loose to medium dense sandy silt and slightly clayey/ silty fine sand including shell fragments and some fine gravel.

### 3.1.2 Made Ground/ Fill

Made ground comprising a variety of material has been used as fill, including a mixture of gravels, sands, silts and clays, and also rubble, bricks, concrete, glass, timber and cinders. It is also reported that hydraulic fill (dredged material from the seabed) material was used to reclaim Dublin Port land (Farrell and Wall, 1990).

Site investigations in the Peninsula have previously logged made ground as being between 1.6m and 5.6m in thickness. The presence of made ground and the frequent industrial usage of land in the Peninsula means that hotspots of soil contamination are quite likely to be encountered.

The composition of Construction and Demolition (C&D) waste varies greatly but commonly consist of a mixture of gravels, sands, silts, clays, rubble, bricks, concrete, glass, timber, concrete slabs, cabling, piping, rags, metal household containers and cinders.

In addition to areas being filled with rubble, large parts of the Peninsula have previously been used as a domestic landfill. Exact records of areas that were landfilled do not exist but it is known that the western part of the Peninsula was used and that the landfill may have extended as far as the Poolbeg Powerstation.



## 3.2 Near shore Bathymetry and Sediments

### 3.2.1 Bathymetry

Dublin Bay is a shallow sandy embayment on the east coast of Ireland. The bathymetry of Dublin Bay is presented on Admiralty Chart No. 1415. The intertidal zone of the Bay occupies the inner third of the bay. The Bay slopes gently reaching depths of 20m at the mouth of the Bay. The navigational channel of Dublin Port is maintained at 7.5m.

The Burford Bank sits centrally across the mouth of Dublin Bay. The Burford Bank is a linear sand ridge about 5km in length, which rises to within 5m of the surface. Bathymetric comparisons suggest that the offshore banks are quasi-stable over time probably maintaining their position due to the interaction between wave and current regimes (Wheeler et al., 2000).

INFOMAR is Ireland's near shore seabed mapping project. It is managed jointly by the Geological Survey of Ireland and the Marine Institute and is overseen by the INFOMAR Programme Board chaired by the Department of Communications, Marine and Natural Resources. INFOMAR will deliver: hydrographic maps illustrating everything from sandbars to underwater canyons and cliffs; seabed classification maps showing the type of sediment on the seabed; and habitat maps showing areas which provide homes to a wide range of marine flora and fauna. (INFOMAR, 2009)

INFOMAR surveys include both Dublin Bay and the approaches in the Irish Sea. Work to survey these areas began 2003 with the Celtic Voyager. Since late July this year, the GSI's new survey vessel, RV Keary has been making progress in completing the shallow water areas of inner Dublin Bay that were not covered by previous surveys by the Celtic Voyager. The RV Keary coverage will be added to by the Celtic Voyager which will be surveying in the Dublin Bay/North of Howth area in late November/early December 2009. A detailed chart of the bathymetry of Dublin Bay will be available following the compilation of this data.

### 3.2.2 Sediments

A seabed classification map showing the type of sediment on the seabed was produced by INFOMAR (2009) and is shown in Figure 5. There is a five class classification divided into two types of rock, reflecting the different textures observed from rock outcrops in the bay. Three more classes divide the sediments into gravels and coarse sand, coarse to medium sand and fine sand to mud.

Near shore, the bedrock is overlain by layers of varying depths of mud, silts, sands, gravels and clay. Predominantly the sediment within the Bay largely consists of upper layers of sand and silt, which overlie boulder clay. From the GSI's National Geotechnical Borehole Database the ranges of depths of each of the layers were determined. These ranges, however, are not conclusive as none of the boreholes met bedrock and the sequence of the layers were inconsistent. For example the boulder clay layer is likely to be a lot thicker. Generally the thicknesses of the sediment layers were:



- Mud: 1m to 12m;
- Sand: 1m to 15m;
- Silt: 1m to 8m;
- Gravel: 1m to 6m; and
- Clay: 1m to 5m.

The upper sedimentary unit of Burford Bank consists largely of sands, and some gravel but also includes clay layers; this unit was found to range in thickness from zero around the Muglins Rocks to about 30m at the bank crests. An average thickness of about 15m was found over flatter intervening seabed areas (Wheeler et al., 2000).

Whittington (1977) reports a channel that is a continuation of the Liffey channel across the Bay southwards. The shallow seismic work shows that this channel is also cut into the boulder clay.

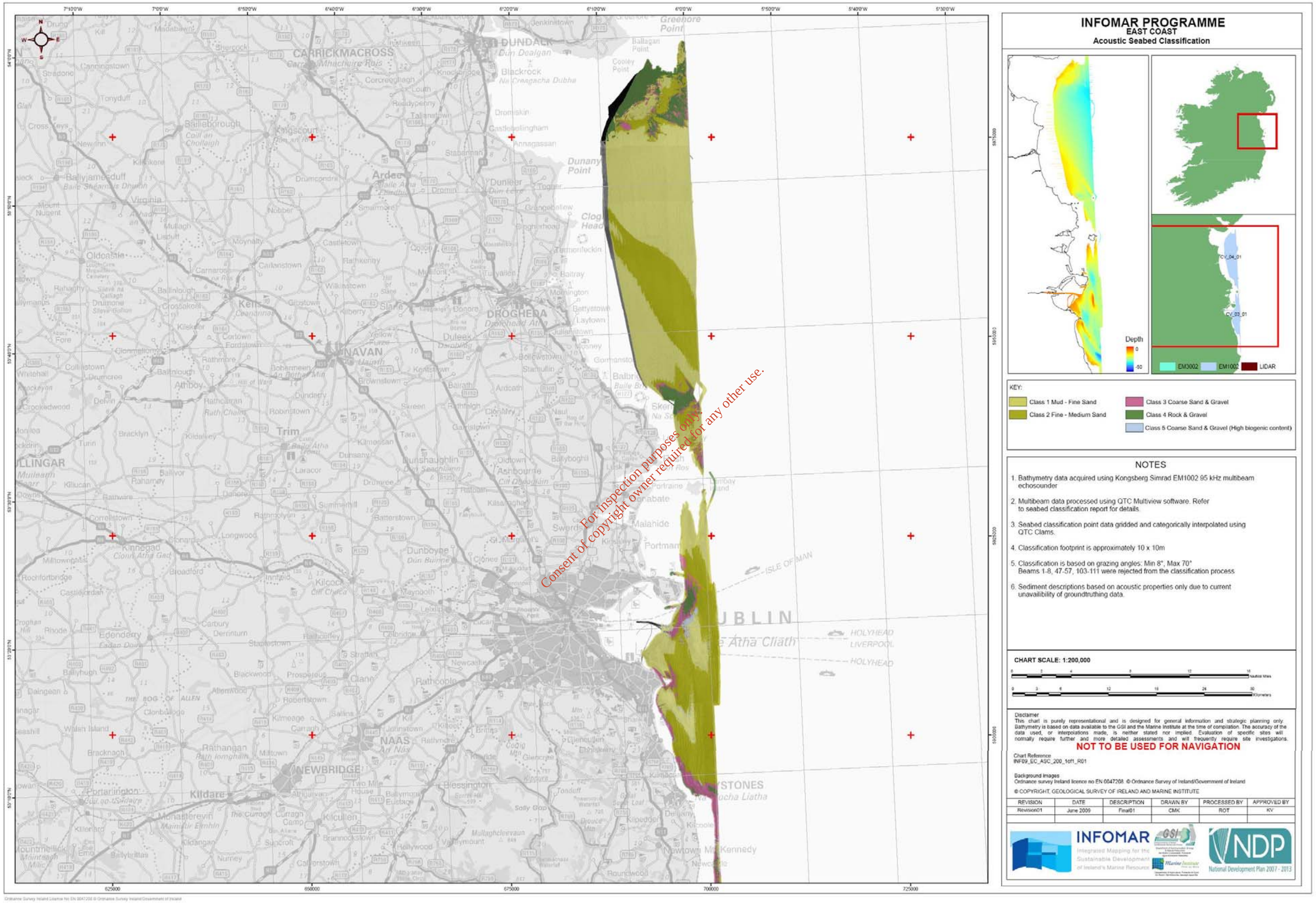



*Figure 5: INFOMAR Seabed Characterisation of Dublin Bay*

# 4 Conclusions and Recommendations

## 4.1 Conclusions

Dublin Bay is a shallow sandy embayment on the east coast of Ireland. The intertidal zone of the Bay occupies the inner third of the bay. The Bay slopes gently reaching depths of 20m at the mouth of the Bay. The Burford Bank sits centrally across the mouth of the Bay. Its a linear sand ridge about 5km in length, which rises to within 5m of the surface.

The dominant bedrock in the Bay is likely to be the Calp limestone judging by the adjacent onshore geology. This is the more easily solubilised, less resilient limestone that has eroded gradually, leaving a well-defined bay. However it cannot be determined if there are changes in the bedrock type, as there is little available information on the structural geology of the Bay at present and none of the existing borings within the Bay reached bedrock.

Bedrock on the Poolbeg Peninsula lies between 30m and 50m below ground level. The deepest rock is in the central area with slightly shallower rock at the tip of the Peninsula. No boreholes have met bedrock in the Bay with the deepest being 25m. The sediment overlying the bedrock within the Bay largely consists of upper layers of sand and silt, which overlies boulder clay.

## 4.2 Recommendations

This section suggests further steps for data gathering prior to the detailed design and construction phase of the proposed pipeline.



### 4.2.1 INFOMAR

It is recommended that the progress of the INFOMAR project is tracked for Dublin Bay, as the principle aims of INFOMAR as a marine mapping project is to collect a range of geophysical datasets that determine the bathymetry of the survey area and the nature of the sediments on and below the seabed.

Note: The seabed characterisation results and bathymetric data for Inner Dublin Bay are due to be completed in the near future.

INFOMAR uses instruments such as (INFOMAR, 2009):

- Multibeam Echosounder (MBES): The hull-mounted MBES transducers emit sound that travels down through the water column. When the high frequency sound wave reaches the seabed most is reflected back towards the surface where sensors record the returning sound wave. Multibeam Systems can also collect additional information, including the strength of the acoustic signal (or return) from the seafloor. This is known as Backscatter. Differing seafloor types, such as mud, sand, gravel and rock will have different Backscatter values depending on the amount of energy they return to the sonar head. Output data from the MBES is used in the production of shaded relief, bathymetric contour, backscatter and seabed classification charts. See Figure 5 the seabed characterisation of outer Dublin Bay.

- Side Scan Sonar (SSS): This allows images of the seabed to be generated. The INFOMAR project uses SSS to acquire good images of wrecks that have been identified on the MBES.

- Light Detection and Ranging (LiDAR): Some shallow areas within the bays are not safe to survey using boats so another method of airborne LiDAR is carried out. The basic principle behind this method is to use laser pulses from the airplane to determine the distance from the sea surface and seabed. The difference between the two beams allows the water depth to be calculated. In Ireland the typical depth penetration is 15 metres and this may vary if sediment or biological material is present in the water.

### 4.2.2 Further Surveys

INFOMAR is carrying out geophysical surveys of the entire Bay and this is not intended to include any geological borings. Further information specific to the route of the proposed pipeline would need to be gathered in advance of the detailed design and construction phase. This may include:

- Hydrographic and marine physical survey: The objective of the hydrographic and marine physical survey will be to collect information supporting the evaluation of seabed and as well as sub-bottom (bedrock) conditions within the area.

- Geophysical Surveys: Offshore borings should be conducted to obtain soil and rock samples, to characterise the general area of the proposed route of the pipeline.

- Side Scan Sonar: To image the seabed conditions and to identify hazards along the proposed route.

# 5 References

Arup Consulting Engineers (2006). Dublin Waste to Energy Project, Ringsend, Dublin. Geo-Environmental Engineering Assessment.

CSA Group (2000). Assessment of the Potential for Geological Storage of Carbon Dioxide for the Island of Ireland. SEI and EPA.

Dobson M. R. and Whittington R. J. (1979).The geology of the Kish Bank Basin. *Journal of the Geological Society*, Vol. 136, Pages 243-249.

Farrell, E. R., and Wall D., (1990). The Soils of Dublin, *Institution of Engineers of Ireland*, Vol. 115, Pages 78-97.

Geological Survey of Ireland (2006). Bedrock Geology Map 1:100,000.

GSI 2009. Geotechnical MapViewer. Online:
<http://www.gsi.ie/Programmes/Quaternary+Geotechnical/Databases/Home.htm>

INFOMAR (2009). Dublin Bay and Approaches. Online:
<http://www.infomar.ie/surveying/Bays/Dublinv1.php>

McConnell, B. and Philcox, M. (1994). Geology of Kildare - Wickow. A Geological Description to Accompany the Bedrock Geology 1:100,000 scale Map Series, Sheet 16, Kildare - Wicklow. GSI.

Mott MacDonald Pettit (2008). Geotechnical Report for Poolbeg Peninsula Planning Scheme. Poolbeg Planning Scheme EIS. May 2008.

Teagasc and Forest Service, Dept of Marine and Natural Resources (2006). National Soils Map.

Teagasc and Forest Service, Dept of Marine and Natural Resources (2006). National Subsoils Map.

Wheeler, A.J., Walshe, J. and Sutton, G.D. (2000). Geological Appraisal of the Kish, Burford, Bray and Fraser Banks, Outer Dublin Bay Area. Marine Institute.

Whittington, R.J. (1977). A late-glacial drainage pattern in the Kish Bank area and post-glacial sediments in the central Irish Sea. In Kidson, C. and Tooley, M. (Eds.) The Quaternary History of the Irish Sea. Liverpool.

# Appendix C

# Ringsend WwTW Proposed Tunnelling Outfall Cost

# RINGSEND WASTEWATER TREATMENT WORKS EXTENSION



## Ringsend WwTW

## Proposed Tunnelling Outfall Cost

CDM

December 2009

# Document Control Sheet

| | | | | |
|---|---|---|---|---|
| Client | Dublin City Council | | | |
| Project | Ringsend | | | |
| Report | Proposed Tunnelling Outfall Cost | | | |
| Date | December 2009 | | | |
| Project No: 67 511 | | Document Reference: DG23 | | |
| **Version** | **Author** | **Reviewed** | **Checked** | **Date** |
| Draft 01 | Lourda Casserly | Bob Gaudes | Annmarie Jordan | 1/DEC/09 |
| | | | | |
| | | | | |
| | | | | |

# Table of Contents



# Table of Figures

# 1.0 Introduction

## 1.1 Introduction

The Ringsend Wastewater Treatment Works (WwTW) Extension Project involves the planning of further works to maximise its capacity in order to meet future needs and to comply with the Urban Waste Water Treatment Regulations.

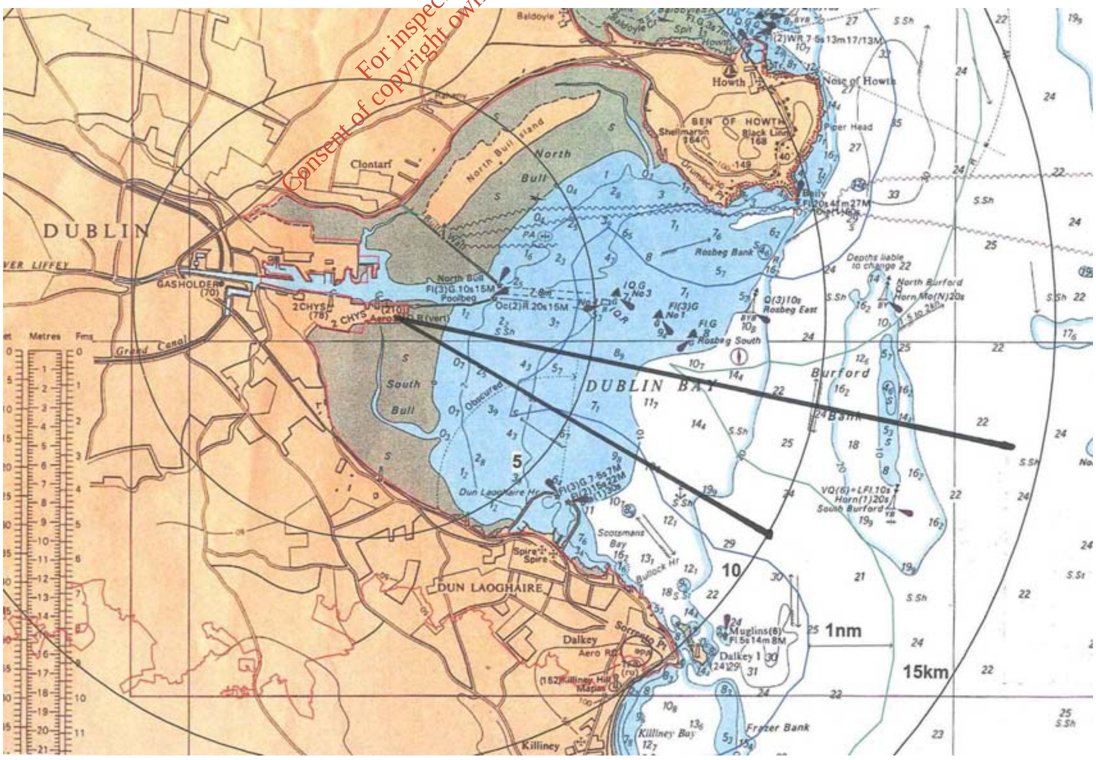
One option being considered is the construction of a long sea outfall, which would bring the treated discharge from Ringsend to a location 7 -10 kilometres out into Dublin Bay. This report outlines the cost involved in construction of a long sea outfall.

## 1.2 Long Sea Outfall

The proposed long sea outfall at Ringsend WwTW will be constructed using a Tunnel Boring Machine (TBM). It is estimated to have a bored diameter of approx 5.90m, an internal diameter of 5.00m and a total length of 10km. One shaft will be constructed for an entry point for the TBM and it is proposed to abandon the TBM at the end of the tunnel. The tunnel will be constructed in the bedrock of Dublin Bay. Further investigation is required to determine the location of the long sea outfall but it is expected to be either in an easterly direction from Ringsend WwTW and end after Burford Bank or be in an east south-east direction. The map below display's the approximate locations for the outfall.



*Figure 1: Map of Dublin Bay and two possible locations for the Long Sea Outfall*






CDM

73369-RT.OUT

## 1.3 Report Outline

The aim of this report is to determine a preliminary cost for constructing a long sea outfall (€/mm DIA/m) and will be further refined after detailed surveys to determine the bedrock geology etc. and detailed design of the outfall is complete.

This report provides a detailed summary of tunnelling costs compiled from the following sources:

- The British Tunnelling Society.
- CDM employees (design and construction of tunnels).
- Australia database of tunnels.
- ASCE, Marine Outfall Construction.
- Other sources including: Mott Mac Donald and Kenny Construction Company.

# 2.0 Data Analysis

## 2.1 Introduction

Using the sources mentioned previously a comprehensive list of tunnels was compiled. Information collated included internal and bored diameter, year constructed, length cost etc. The cost (€/mm diameter/m) was calculated using the total length, internal diameter and cost for construction. The following section details the methodology and data analysis. Information regarding the tunnels can be grouped according to the following and detailed in the following sections:

1. Tunnels with sufficient information and used to calculate the average cost for constructing the tunnel. (Group 1).

2. Tunnels cost data not used to calculate the average cost for constructing the tunnel. (Group 2).

## 2.2 Group 1

This group includes all tunnel data (30) deemed sufficient from which to calculate the cost (See Table 1 below).

Tunnels which were completed prior to 2009 are adjusted using the Construction Cost Index (See Appendix 1 for US (1908-Nov 09) data and Appendix 2 for UK (97-08) data). CCI was calculated using the mid-point of construction.

For UK tunnels constructed prior to '97 - US CCI data was used. For tunnels constructed in Australia, Iraq, Egypt etc. the US CCI data was used. Notes for the calculation of CCI can be seen in the table 1 below.



The cost was calculated using the following steps:

1. Construction Cost Index multiplier: =

   *Construction Cost Index 2009 / Construction Cost Index for mid point of Construction*

2. Cost €/mm DIA/m =

   *CCI multiplier x (Cost / Diameter / Length)*

Cost = €, Diameter = ID in mm, Length = m.

See section 2.2.1 for further cost analysis.



CDM

73369-RT.OUT

*Table 1: Construction Cost Details (Group 1)*

| Number | Location | Name | Use | Type of Soils | Excavation Method* | Machine | Linings | Date | Diameter Finished (m) | Diameter Bore (m) | Length (m) | Cover | Cost Euro/mm diameter/m | Total Cost (€) | CCI | Notes on CCI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Birmingham | Birmingham - Perry Hill to Gravelly | Sewer | Mixed clay, sand gravels in river basin | EPB | Lovat | Precast bolted segments | 1998-2000 | 3 | 3.4 | 2750 | 6-15m | 1.99 | 16,426,689 | 1.72 | 99 and '08 English CCI. See Appendix 1 |
| 2 | Birmingham | Birmingham - Perry Hill to Gravelly | Sewer | Sandstone | EPB | Lovat | Precast bolted segments | 1997-2000 | 3 | 3.4 | 2750 | 6 to 30m (Water Table: Near the surface) | 1.88 | 15,528,244 | 1.75 | Average Q4 '98, Q1 '99 and '08 English CCI See Appendix 1 |
| 3 | Yorkshire | Hull - Humbercare | Sewer | Alluvial Deposits | EPB | Two Lovat RME 167SE series | Precast trapezoidal lining, 1m wide | 1999-2001 | 3.6 | 4.24 | 10600 | Up to 22m | 2.61 | 99,597,080 | 1.63 | 00 and '08 English CCI. See Appendix 1 |
| 4 | Hampshire | Portsmouth - Transfer Tunnel | Sewer | Tertiary Beds of London Clay, a mixture of silts, sands and clays, and Chalk | TBM | 2 Lovat RME 131 | Bolted precast segments | 1999-2001 | 2.9 | 3.4 | 8000 | 21-30m (Water Table: 2.5bar) | 2.19 | 50,703,968 | 1.63 | 00 and '08 English CCI. See Appendix 1 |
| 5 | West Midlands | Walsall - Bescot Crescent | Sewer | Made ground overlying glacial drift deposits, Coal Measures | EPB and microtunnelling | Lovat M115SE and Iseki Unclemole | Precast bolted linings, 1m wide, and pipes | 1994-1995 | 2.44, 1.2 | 2.85, 1.5 | 1182 | 4.5 to 7m. 700mm under Ford Brook (Water Table: Near surface) | 1.57 | 4,534,941 | 1.57 | Have English CCI data for 97-08. Use US data: Dec '94, Jan '95 and Nov '09. See Appendix 2 |
| 6 | Norfolk | Cromer - West Runton Outfall | Sewer | Upper Chalk with flints | EPB | Dosco | Precast trapezoidal segmental lining, 1m wide and 6 segments | 1993-1994 | 2.24 | 2.6 | 2200 | 5m of rock with 18m of sea water and 32m on land (Water Table: 23m) | 2.00 | 9,837,270 | 1.61 | Have English CCI data for 97-08. Use US data: Dec '93, Jan '94 and Nov 09 US CCI data. See Appendix 2 |
| 7 | Nottinghamshire | Mansfield - Outfall | Sewer | Middle Permian Marl and Lower Magnesian Limestone | Roadheader in shield, slurry shield, hand excavation and drill and blast | Dosco SB300 roadheader in shield and on tracks, Herrenknecht SM2 roadheader and AVN slurry TBM | Trapezoidal and one pass smoothbore linings and pipes. | 1993-1995 | 3.55, 3 x 3, 2.59, 1.83, 1.52. 1.5, 1.47, 1.0 | 2.0 - 1.3 | 1713 | NA | 1.83 | 10,529,213 | 1.58 | Have English CCI data for 97-08. Use US data: '94 and '09 (Jan-Nov) CCI data. See Appendix 2 |
| 8 | Leicestershire | Leicester - Abbey | Sewer | Mercia mudstone or Keuper marl and siltstone | EPB | Lovat M115SE | Smoothbore tapered lining, 1m wide | 1992-1995 | 2.44, 1.2 | 3 | 6200 | 3m to 8m (Water Table: Near surface) | 1.12 | 16,935,917 | 1.61 | Have English CCI data for 97-08. Use US data: Dec '93, Jan ' 94 and '09 (Jan-Nov) CCI data. See Appendix 1 |
| 9 | Essex | Thurrock - Southern Trunk | Sewer | Alluvial Thames Flood Plain sands, gravels and clays | TBM | Iseki crunching mole TCM 2140 | Pipes | 1990 | 1.8 | 2.14 | 1200 | 12-14m (Water table: up to 11m) | 2.79 | 6,016,142 | 1.81 | Have English CCI data for 97-08. Use US data: '90 and '09 (Jan-Nov) CCI data. See Appendix 1 |
| 10 | Jersey | Jersey - St.Helier, Surface Water Link and Storage Cavern | Sewer | Fort Regent granite | Tunnel - slurry TBM and raised bore, cavern drill and blast | Tunnel modification TBM by Decon/Markham JV. Tamrock 3 boom for cavern | Precast trapezoidal segmental lining for tunnel | 1994-1997 | 2.7 | Tunnel 3.2, cavern 18 wide - 7 high | 1504 | - | 3.74 | 15,167,510 | 1.56 | Have English CCI data for 97-08. Use US data: Dec '95, Jan '96 and Nov '09 CCI data. See Appendix 2 |
| 11 | Belfast | Belfast Sewer Project | Stormwater Sewer | Rock/Soil | - | - | - | 2007 - 2009 | 4 | - | 9400 | 90m | 2.96 | 111,227,903 | OK | 08 UK CCI data. See Appendix 1 |
| 12 | Spain | Cartagena, CCTPP Combined Cycle Thermal Power Plant | - | Quaternary gravels and sands, breccias, phyllites, Triassic dolomites and Precambrian; black mica schists. | Shielded TBM (EPB) | | Single shell segment lining | 2002 | 4.4 | 5.2 | 2800 | - | 3.55 | 43,712,589.64 | 1.51 | Don't have CCI data for Spain. Use English Data. 02 and 08 data. See Appendix 1 |
| 13 | Germany | Hofoldinger Stollen City of Munich Hochtief AG | - | Quaternary gravels and sands, mostly above GW-level | Shielded open TBM | - | Segment lining inner steel tube | 2000-2006 | 2.9 | 3.45 | 17400 | - | 2.30 | 116,194,307.12 | 1.42 | Don't have CCI data for Germany. Use English Data. 03 and 08 data. See Appendix 1 |
| 14 | Spain | Eurohinca stormwater sewage system, Valdemarin/Madrid | Stormwater | Deposits, alluvial ground, sands, limestone. Water pressure up to 2 bar. | Shielded EPB TBM | - | Single shell - segment lining | 2006-2007 | 2.76 | 3.1 | 1300 | - | 2.80 | 10,043,893.81 | 1.12 | Don't have CCI data for Spain. Used English CCI data. Q4 '06 and Q1 '07. See Appendix 1. |
| 15 | LA | Los Angeles - Outfall Sewer | Sewer | Predominantly soft ground and sandy soils in the Lakewood Formation and hard clay and soft clay in the San Pedro Formation | EPBTBMs (4) | Lovat Tunnel Equipment | Bolted casketed precast concrete segments | 2004 | 3.35 | 4.72 | 18500 | - | 3.34 | 206,938,724 | 1.20 | 04 and '09 (Jan-Dec) US CCI data. See Appendix 2 |
| 16 | Chicago | Chicago - Calumet Tunnel | - | Hard rock. | TBM | Robbins | - | 2003-2004 | 9.83 | - | 18571 | - | 0.90 | 163,896,809 | 1.26 | Dec '03, Jan '04 and Jan '09 US CCI data. See Appendix 2 |
| 17 | Chicago | Chicago - Torrence Ave | - | - | TBM<br>TBM<br>Pipejack<br>-<br> | -<br>-<br>-<br>-<br>- | -<br>-<br>-<br>-<br>- | 2003-2004 | 7.62<br>4.57<br>2.44<br>7.62<br>9.45 | 8.23<br>5.18<br>-<br>-<br>- | 10409<br>2494<br>122<br>93<br>93 | -<br>-<br>-<br>Shaft<br>Lined gate shaft | 1.17 | 118,276,048 | 1.26 | Dec '03, Jan '09 and '09 (Jan - Nov) CCI data. See Appendix 2 |
| 18 | Gary, IN | Gary, IN - Borman Park | Water | Hard rock | TBM | Robbins | Concrete | 2003 | - | 3.5 | 4821 | - | 2.05 | 34,583,982 | 1.28 | 03 and '09 (Jan - Nov) CCI data. See Appendix 2 |
| 19 | Oak Creek, WI | Oat Creek, WI - Expansion | Cooling Water | Limestone | TBM | Robbins | - | 2005-2007 | - | 8.35 | 2804 | - | 3.33 | 77,873,880 | 1.10 | 06 and '09 (Jan - Nov) CCI data. See Appendix 2 |
| 20 | Shanghai | Upgrade and extend Shanghai's Sewerage System | Sewerage | - | - | - | - | Completed in 2007 | 2.4 - 3.5 | - | 30000 | - | 2.36 | 248,117,755 | 1.08 | Don't have CCI data for China. Use US data: '07 and '09 (Jan-Dec) CCI data. See Appendix 2 |

*Table 1 (Continued): Construction Cost Details (Group 1)*

| Number | Location | Name | Use | Type of Soils | Excavation Method* | Machine | Linings | Date | Diameter | | Length (m) | Cover | Cost Euro/mm diameter/m | Total Cost (€) | CCI | Notes on CCI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Finished (m) | Bore (m) | | | | | | |
| 21 | Kings County, Washington | Brightwater Conveyance System | Sewer | Interglacial deposits | TBM | Lovat EPB TBM | concrete segmental-lined | 2010 | 5.12 and 1.2 - 1.8(microtunnel) | - | 10150 | - | 2.70 | 140,348,530 | - | - |
| 22 | Kings County, Washington | Brightwater Conveyance System | Sewer | Interglacial deposits | TBM | Lovat EPB TBM | concrete segmental-lined | 2011 | 5.87 and 1.8 microtunnel | - | 5023 | - | 2.33 | 68,568,753 | - | - |
| 23 | US | Combined Sewer Overflow Storage Tunnel | Sewer | - | - | - | - | 2001-2008 | 9.14 | - | 4825 | - | 3.05 | 134,429,466 | 1.18 | Average Dec 04 - Jan 05 and Nov 09 US CCI data. See Appendix 2 |
| 24 | Milwaukee Wisconsin | Northeast Side Relief Sewer | Sewer | Limestone | Tunnel Boring Machine for the main tunnel Drill and shoot technique for the rock portion of the access shafts and for connecting and starter tunnels Smaller diameter (1.5 - 2.1 meter) shafts were excavated by down hole drilling techniques | Refurbished 1973 or 1974 Robbins TBM equipped with six 350 hp variable speed drive motors | Generally 304 millimetres thick unreinforced cast-in-place concrete liner Single mat reinforcing steel was added to the liner at junctions with shafts and connecting tunnels. | 2002-2005 | 6.1 | 6.78 | 11430 | - | 1.28 | 89,558,181.33 | 1.26 | Average Dec '03 - Jan '04 and Nov '09 US CCI data. See Appendix 2 |
| 25 | Boston, MA | - | Sewer | - | - | - | - | 1995 | 8.1 | - | 15200 | - | 2.84 | 350,127,439.64 | 1.57 | 95 and '09 (Jan - Nov) US CCI data. See Appendix 2 |
| 26 | Auckland | Hobson Bay Sewer | Sewer | East coast bay formation, a Miocene marine sedimentary sequence, consisting of moderately weathered to unweathered sandstones, siltstones, and mudstones with probable cemented pockets and lenses of poorly sorted sand to boulder-sized conglomerate | TBM | Lovat RME 170SE | - | 2010 | 3.7 | - | 3000 | - | 2.64 | 29,271,833 | - | - |
| 27 | India | Tapovan, Vishnugad Headrace Tunnel | Hydro-Electricity | Gneisses, Quarzites, schists with height overburden; plastic deformations of strata | Double Shield TBM | - | Single shell - segment lining | 2006-2010 | 5.6 | 6.5 | 8000 | - | 1.56 | 70,078,459.69 | 1.03 | Don't have CCI data for India. Used US CCI. '08 and '09 (Jan - Nov) US CCI data. See Appendix 2 |
| 28 | Shanghai | - | Sewer | - | - | - | - | 1995 | 4.2 | - | 1600 | - | 3.59 | 24,091,736.34 | 1.57 | Don't have CCI data for China. Used US CCI. '95 and '09 (Jan - Nov) US CCI data. See Appendix 2 |
| 29 | Iraq | Gavoshan | - | Flysh formations of silty, sandy and limy shales with silt-stone and sandstone layers. Large limestone blocks, sandy and limy nodules. Rock partly intruded by volcanic material. Rock water above tunnel. Eternal water pressure limited to 5 bar. | Shielded TBM (EPB) | - | Single shell segment lining | 1999-2004 | 4.3 | 5.3 | 7600 | - | 3.18 | 103,951,447.25 | 1.33 | Don't Iraq CCI data for Iraq. Use US data. Average Dec 01 - Jan 02 and Noc 09. See Appendix 2 |
| 30 | Cairo, Egypt | El Salaam Syphon under Suez Canal. | - | Silty sand, soft and stiff slay | Shielded TBM (Slurry) | - | Single shell segment lining | 1996-2000 | 5.3 | 6.3 | 4800 | - | 1.71 | 43,398,648.65 | 1.45 | Don't have CCI data for Egypt. Use US data. 98 and 09 (Jan-Nov) See Appendix 2 |

*Abbreviations

| | |
|---|---|
| EPB | Earth Pressure Balance (Machine) |
| TBM | Tunnel Boring Machine |
| EPBTBM | Earth Pressure Balance Tunnel Boring Machine |

### 2.2.1 Cost Analysis

See Table 2 below for the calculated Average and Median cost for constructing the long sea outfall.

*Table 2: Cost Analysis*

| | Average (€/mmDIA/m) | Median (€/mmDIA/m) |
|---|---|---|
| British Tunnels | 2.24 | 2.00 |
| European Tunnels | 2.38 | 2.24 |
| Non-European Tunnels | 2.38 | 2.50 |
| All Tunnels | 2.38 | 2.34 |

The above costs are based on data from Group 1. Table 2 shows the average costs for constructing a tunnel is €2.38. The difference in cost between the UK and others is due to the construction of shafts, etc. included in the cost.

Using the average cost for European, Non-European and All Tunnels the cost for constructing (ID = 5m, Length = 10km) the Ringsend long sea out fall is €119 million (i.e. €2.38 x 5,000 x 10,000 = €119 million).

The tunnels constructed recently and of relevance for the cost analysis include:

1. Belfast Sewer, Northern Ireland. (Table 1: No. 11).

This is an underground sewer network constructed in Belfast using a tunnel boring machine (TBM) with a diameter of 4m, length 9.4km. Work for this tunnel was completed in 2009 and cost approximately €112million, €2.96 /mm DIA /m.

2. Brightwater Conveyance System, Kings County, Washington, America (Table 1: No. 21,22)

The Brightwater Conveyance System consists of two tunnels namely the Central and West tunnel.

The Central tunnel is constructed using a Herrenknexht mixshield slurry TBM with an internal diameter of 5.12m and length 10.15km. Completion date for this tunnel is late 2010 and will cost approx €140million, €2.70 /mm DIA/m.

The Western tunnel is constructed using a Lovat EPB TBM with an internal diameter of 5.87m and length 5km. This tunnel is due to be completed in early 2011 and will cost approx €70million, €2.33 /mm DIA/m.

3. Northeast Side Relief Sewer, Milwaukee Wisconsin, America (Table 1: No. 24 )

This sewer was constructed using two refurbished '73 and '74 Robbins TBM with diameter 6.1m, length of 11.4km and construction was finished in 2005. This project finished at a total cost of approximately €71 million, €1.28 /mm DIA/m.

4. Tapovan, Vishnugad Headrace Tunnel NTPC, India (Table 1: No. 27)

Construction for the headrace tunnel is due for completion in 2010. The tunnel is constructed using a double shield TBM. The diameter is 5.6m and total length of 8km. The cost for constructing this tunnel is approximately €68 million, €1.66 /mm DIA/m. However, when factoring in the difference between Indian and Irish labour costs the adjusted unit cost is in the neighbourhood of €2.50 / mm DIA/m.

The tunnels mentioned above (1-3) have additional costs included e.g. shafts etc. The long sea outfall for Ringsend would have only one access and diffuser outlet.

Figures 2 and 3 below are created using all data from Group 1 and display the relationship between diameter, length and cost (€/mm DIA/m). The data doesn't show a good correlation; hence a cost estimate can't be determined from the above figures.



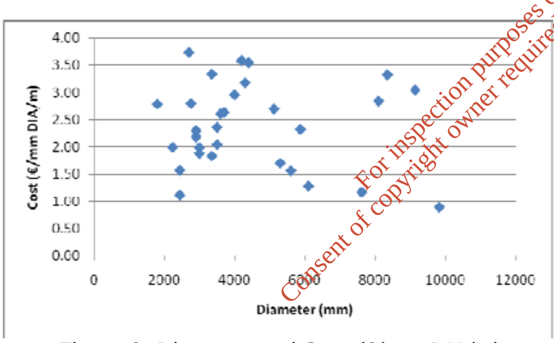



*Figure 2: Diameter and Cost (€/mm DIA/m)*

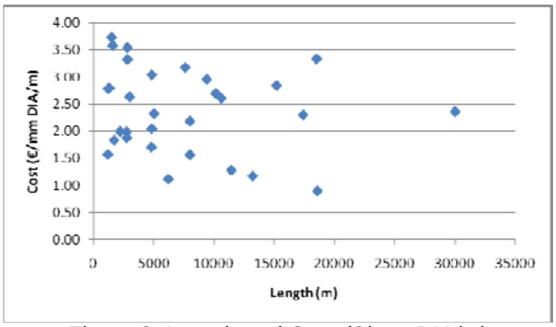



*Figure 3: Length and Cost (€/mm DIA/m)*

From analysing the data the cost for constructing the long sea outfall will be approximately €2.00 - €2.50 /mm DIA/m. (€100 million - €125 million).

## 2.3 Group 2

This group includes all tunnel data without sufficient information or disregarded due to a valid reason. This group includes 56 tunnels, details can be seen below in Table 3.

*Table 3: Construction Cost Details (Group 2)*

| Number | Location | Name | Use | Type of Soils | Excavation Method* | Machine | Linings | Date | Diameter Finished (m) | Diameter Bore (m) | Length (m) | Cost Euro/mm diameter/m | Total Cost (€) | CCI | Notes on CCI | Notes on Cost |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | London | London Ring Main - Coppermills to Stroke | Water | Reading Beds - clay marl with some limestone | Shield with cutter boom | Zublin and Dosco | Pipes, 340mm thick, 5m long | 1987-1989 | 2.2 | 2.88 | 3000 | - | - | - | - | Construction year pre 1990. Missing Data: Cost. Not TBM |
| 2 | London | London Docklands - Royal Docks | Sewer | Woolwich & Reading Beds, Flood Plain gravels and London Clay | TBM EPB (2.1m) and Mixshield (1.8m) | Lovat (EPB) Herrenknecht (Mixshield) | Pipes | 1988-1991 | 2.1, 1.8 | 2.5, 2.15 | 5200 | 1.95 | 18,276,178 | 1.83 | Have English CCI data for 97-08. Use US data: '89, '90 and '09. See Appendix 2 | Construction year pre 1990 |
| 3 | London | Nunhead to Depford Water Main | Water | London Clay and Woolwich & Reading Beds | Trenchless and tunnelling | Iseki Uncle Mole and Akkerman | Pipes | 1998-2000 | 1.2, 0.6 | 0.8, 1.4 | 1800 | 6.37 | 13,753,786 | 1.72 | 99 and '08 English CCI. See Appendix 1 | Includes 2.2km of unknown construction. "3km of which 1.8km in tunnel". Not TBM |
| 4 | Lothian | Edinburgh - Outfall | Sewer | Colliery arches | Drill and blast | - | Cast in situ concrete lining with Stelmo telescopic shutter | 1973-1976 | 3.66 | 4.5 horseshoe | 2800 | 1.27 | 13,005,039 | 4.05 | Have English CCI data for 97-08. Use US data: '74, '75 and '09 (Jan-Nov). See Appendix 2 | Construction year pre 1990. Not TBM |
| 5 | Lothian | Edinburgh - Easter Interceptor Sewer | Sewer | Boulder clay and beach deposits overlying Carboniferous strata | Shield and compressed air | - | Precast concrete segmental lining with cast in situ concrete secondary lining | 1971-1972 | 2.29, 1.67 | 2.9, 2.3 | 2213 | 1.01 | 5,123,703 | 5.14 | Have English CCI data for 97-08. Use US data: '71, '72 and '09 (Jan-Nov). See Appendix 2 | Construction year pre 1990. Not TBM |
| 6 | Dorset | Weymouth and Portland - Underhill | Sewer | Silty sand | Hand shield. Variation hand shield with compressed air. | NA | Precast smoothbore and bolted linings | 1981-1983 | 1.2, 1.8 | 1.5, 2.1 | 360 | 0.77 | 496,204 | 2.24 | Have English CCI data for 97-08. Use US data: '82 and '09 (Jan-Nov)US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 7 | London | London - Western Deep Sewer | Sewer | London Clay | Shield with backhoe | Decon | Precast expanded and bolted linings | 1988-1990 | 2.5 | 2.85 | 3300 | 2.24 | 18,506,834 | 1.88 | Have English CCI data for 97-08. Use US data: '89 and '09 (Jan-Nov)US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 8 | Dorset | Poole - Surrey Road | Sewer | Sandy clay | TBM | Lovat | Pipes | 1983-1984 | 1.8 | 2.2 | 2180 | 0.47 | 1,848,981 | 2.09 | Have English CCI data for 97-08. Use US data: '83, '84 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction Year pre 1990 |
| 9 | London | London - Edmonton | Sewer | Sand and gravel, above silts and clays | Hand shield with sand trays available. | - | Pipes | 1991-1992 | 1.5, 1.35, 0.9 | 1.9 - 1.2 | 840 | 1.72 | 1,947,569 | 1.76 | Have English CCI data for 97-08. Use US data: Dec '91, Jan '92 and Nov 09 US CCI data. See Appendix 2 | Not TBM. |
| 10 | Surrey | Burgh Heath Interceptor | Sewer | Clays, sand and chalk | Shield and backhoe | Clarke Engineering, Oldham | Pipes | 1990-1991 | 1.5 | 1.9 | 915 | 1.16 | 1,594,421 | 1.80 | Have English CCI data for 97-08. Use US data: Dec '90, Jan '91 and Nov 09 US CCI data. See Appendix 2 | Not TBM - Shield and backhoe (JCB) |
| 11 | Kent | Margate - Relief Sewer | Sewer | Chalk | Hand excavation without shield except under railway line | - | Spun concrete Flexible lining | 1985-1986 | 1.2, 1.0, reinverted to 0.6 | 1.5, [illegible] | 4400 | 1.02 | 5,365,558 | 2.02 | Have English CCI data for 97-08. Use US data: '85, '86 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 12 | Northern Ireland | Belfast - Duncrue Street Sewage Treatment Works | Sewer | Alluvial clay with sand layers | Shield with well point dewatering | - | Bolted segmental lining | 1992-1995 | 3 | 3.5 | 360 | 549.87 | 593,855,233 | 1.61 | Have English CCI data for 97-08. Use US data: Dec '93, Jan '94 and Nov '09 CCI data. See Appendix 2 | Cost for whole scheme |
| 13 | Merseyside | MEPAS - Birkenhead - Green Lane | Sewer | Sandstone, Alluvial Deposits to running sands | Hand shield with some blasting (2.4m pipejack) and TBM in rock (segments) | Lovat M100 | Pipes (1.2m and 2.4m) and precast smoothbore lining, 1m wide (2.4m) | 1990-1991 | 2.4 - 1.2 | 2.87, 1.5 | - | - | 7,972,669 | 1.80 | Have English CCI data for 97-08. Use US data: Dec '90, Jan'91 and Nov '09 US CCI data. See Appendix 2 | Not all TBM. Missing data. |
| 14 | Devon | Newton Abbott - Aller Valley | Sewer | Silty sands and gravels with boulders | Shield and compressed air up to 0.7bar, shield (900mm). Pipejack and microtunnelling. | Decon (1.2m) | Precast smoothbore lining (1.2m) and pipes (900mm) | 1990-199[illegible] | 1.2, 0.9 | 1.5, 1.2 | 551 | 16.58 | 10,961,593 | 1.80 | Have English CCI data for 97-08. Use US data: Dec '90, Jan'91 and Nov '09 US CCI data. See Appendix 2 | Not TBM. |
| 15 | Aberdeenshire | Aberdeen - Sea Outfall | Sewer | Granite | Drill and blast | Tamrock twin boom | Secondary lining with 21m long shutter at an average of 61m/week on three shifts with a maximum of 60m/week | 197[illegible]-1984 | 2.5 | 3.1 by 3.1 | 2500 | 1.50 | 9,374,469 | 1.66 | Have English CCI data for 97-08. Use US data: '81 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990. |
| 16 | London | London - Islington Seven Sisters | Sewer | London Clay | Shield with backactor | James Howden | Expanded wedge block lining 140mm thick and [illegible] segments | 1986-1987 | 2.54 | 2.84 | 1700 | 1.16 | 5,016,921 | 1.97 | Have English CCI data for 97-08. Use US data: '86, '87 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 17 | London | London - Subiton Hogsmill | Sewer | London Clay | TBM | Lovat | Expanded wedge block lining, 1m wide | 1997-1998 | 2.6 | 3 | - | - | 17,715,282 | 1.78 | Q4 '97, Q1'98 and '08 English CCI data. See Appendix 1 | Missing data. |
| 18 | Oxfordshire | Oxford - West Oxford | Sewer | Terracec Gravels and Oxford Clay | Hand excavation , TBM and microtunnelling | Akkerman, Iseki Unclemole | 25m/week ([illegible]) | 1997-1998 | 1.2, 1.0, 0.6 | 1.5, 1.2, 0.8 | 5500 | 1.85 | 12,203,861 | 1.78 | Q4 '97, Q1'98 and '08 English CCI data. See Appendix 1 | Included in the cost is hand excavation and micro tunnelling |
| 19 | East Sussex | Hastings - Bathing Water | Sewer | Estuarine deposits, peat, sand, sedimentary rock, mudstone, siltstone and limestone | Mixshield | Herrenknecht | Trapezoidal seven segment lining with shear pads. 300mm thick with invert segment [illegible]mm thick. EPDM gaskets | 1997-1999 | 6.5 | 7.5 | 1600 | 7.99 | 83,070,507 | 1.75 | '98 and '08 English CCI data. See Appendix 1 | Not TBM. |
| 20 | Kent | Sandwich Bay - Ramsgate Storm Tanks | Sewer | Chalk | Two main drives with roadheader, other drives hand shields | Dosco | Precast concrete segmental linings | 1993-1995 | 4.5, 2.44, 0.9 | 5, 2.8, 1.2 | 1280 | - | - | - | - | Geology. Missing Data. |
| 21 | Kent | Isle of Grain Power Station Shafts | Water | Alluvial silt overlying London clay | - | - | Precast concrete segmental rings 6m diameter prestressed vertically with Macalloy bars | 1972 | 5.4 | 7 | 100 | 5.01 | 2,706,838 | 4.89 | Have English CCI data for 97-08. Use US data: '72 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Construction year pre 1990 |
| 22 | London | London - Hounslow - Brentford Area Surface Water | Sewer | Clay and saturated sands and mixed faces. Pipejack in silty clay and river gravels, and clay | Hand shield with compressed air up to about 1 bar. Pipe jack some dewatering | - | One pass lining and pipes | 1991-1992 | 1.35, 1.2 | 1.7, 1.5 | 1730 | 1.04 | 2,434,592 | 1.76 | Have English CCI data for 97-08. Use US data: Dec '91, Jan '92 and Nov '09 US CCI data. See Appendix 2 | Not TBM. |
| 23 | Suffolk | Sizewell B Power Station | Cooling Water | Loose sand and gravel beach deposits | Trench excavation | - | Precast units 100m long in nine 11m lengths. Precast in Teeside | 1988-1990 | 4.5 square | 5.5 square | 900 | - | - | - | - | Not TBM. Missing data. Construction year pre 1990 |
| 24 | Yorkshire | Sheffield - Don Valley Stage 3 | Sewer | Coal Measures | Drill and blast within a shield with a loader | Haggloader | Precast bolted concrete lining, 750mm and gaskets | 1986-1988 | 4.11, with branch tunnels 1.91 | 4.7, 2.3 | 1134 | 2.22 | 10,338,819 | 1.94 | Have English CCI data for 97-08. Use US data: '87 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction Year pre 1990 |
| 25 | Yorkshire | Sheffield - Don Valley Stage 2 | Sewer | Coal Measures | Shield with roadheader | Dosco SB400 in Stelmo shield | Precast bolted concrete tapered lining with gaskets, 750mm wide | 1983-1986 | 4.11, 1.96 | 4.65, 2.3 | 3300 | 1.49 | 20,253,769 | 2.05 | Have English CCI data for 97-08. Use US data: '84, '85 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 26 | Yorkshire | Sheffield - Don Valley Phase 4 | Sewer | Coal Measures | Drill and blast in shield | Decon | Precast bolted concrete trapezoidal lining with gaskets, 1m wide. secondary lining 2.65m internal diameter | 1989-1991 | 2.9 | 3.47 | 2266 | 3.42 | 22,463,492 | 1.81 | Have English CCI data for 97-08. Use US data: '90 and '09 (Jan-Nov) US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 27 | Avon | Bristol - Northern Foul Water Interceptor | Sewer | Carboniferous limestone, quartzite, sandstone, coal measures, interbedded sandstone and mudstone. Triassic dolomitic conglomerate | Drill and blast | Tamrock RMHS 305M 3 boom hydraulic jumbo | Nominal 350mm cast insitu concrete | 1990-1993 | 3.66 | Horseshoe 4.36 high by 3.2 wide | 2900 | 1.65 | 17,527,746 | 1.76 | Have English CCI data for 97-08. Use US data: Dec '91 and Jan '92 and Nov'09 US CCI data. See Appendix 2 | Not TBM. Construction year pre 1990. |
| 28 | Lothian | Edinburgh - Western Interceptor | Sewer | Boulder clays and beach deposits overlying Carboniferous strata | Shield and compressed air. Some blasting | - | Precast concrete segmental bolted linings with cast insitu concrete secondary lining | 1972-1974 | 3.07 | 3.7 | 2800 | 1.11 | 9,514,500 | 4.52 | Have English CCI data for 97-08. Use US data: '73 and '09 (Jan-Nov) CCI data. See Appendix 1 | Not TBM. Construction year pre 1990 |
| 29 | Lancashire | Bury - North South Interceptor | Sewer | Mudstone, shale and sandstone | TBM modified with disc cutters | Lovat | Precast butt end jacking pipes | 1984-1985 | 1.6 | 2 | 1820 | 1.64 | 4,778,918 | 2.05 | Have English CCI data for 97-08. Use US data: '84, '85 and '09 (Jan-Nov) CCI data. See Appendix 2 | Construction year pre 1990. |

*Table 3: (Continued): Construction Cost Details (Group 2)*

| Number | Location | Name | Use | Type of Soils | Excavation Method* | Machine | Linings | Date | Diameter Finished (m) | Diameter Bore (m) | Length (m) | Cost Euro/mm diameter/m | Total Cost (€) | CCI | Notes on CCI | Notes on Cost |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 30 | Kent | Folkestone - Interceptor | Sewer | Gault Clay overlying Folkestone Beds overlying Sandgate Beds | EPBs | Lovat | Precast smoothbore linings | 1997-1999 | 2.44, 2.87 | 2.80, 3.20 | 1900 | - | - | - | - | Not TBM. Missing Data |
| 31 | Cambridgeshire | Cambridge - Riverside | Sewer | Gault Clay | Pipejack with shield and backhoe | Decon shield and Iseki Unclemole | 2.1, 1.5m and 1.2m pipejacks, short length of smoothbore | 1991-1994 | 2.1, 1.5, 1.2 | 2.6, 1.9, 1.5 | 6000 | 1.42 | 17,859,124 | 1.70 | Have English CCI data for 97-08. Use US data: Dec '92, Jan '93 and Nov '09 CCI data. See Appendix 2 | Not TBM. Pipejack and backhoe (JCB) |
| 32 | Carmarthenshire | Cross Hands Tunnel | Water | Coal Measures | Drill & blast | - | Cast insitu concrete | 1968-1972 | 1.93 | 2.8 | 6400 | 11.13 | 137,446,824 | 6.20 | Have English CCI data for 97-08. Use US data: '70 and '09 (Jan-Nov) CCI data. See Appendix 2 | Not TBM. Construction year pre 1990. |
| 33 | Cornwall | Falmouth - Storage Tank | Sewer | Siltstone/Mudstone with clay seams | Drill and blast in 2.4m pulls | - | Sprayed steel fibre reinforced concrete lining, 100mm thick | 1996-1997 | 2.9 | 3.1 by 3.1 | 450 | 2.16 | 2,812,466 | 1.49 | Have English CCI data for 97-08. Use US data: Dec '96, Jan '97 and Nov '09 CCI data. See Appendix 2 | Not TBM. |
| 34 | Cornwall | Brixham - Stormwater Retention Tunnel | Sewer | NA | Drill and blast | - | - | 1996-1997 | 2.1 wide by 2.6 high | 3.1 | 450 | - | - | - | - | Not TBM |
| 35 | Rutland | Empingham - Water Transfer Tunnel | Water | Upper lias with bands of limestone | TBM | Priestley | Expanded wedge block lining, 680mm wide | 1972-1973 | 2.54 | 2.85 | 13900 | 0.59 | 20,810,225 | 4.70 | Have English CCI data for 97-08. Use US data: '72, '73 and '09 (Jan-Nov) CCI | Construction year pre 1990 |
| 36 | London | London - Northern and Southern Links | Water | London clay with bands of sandstone and gravel intrusions | Backhoe in shield | Herrenknecht/Zoker refurbished concrete lining | Pipes and precast expanded concrete lining | 1987-1989 | 1.82 (pipejack), 2.54 (tunnel) | 2.2, 2.85 | 7900 | 1.57 | 31,498,548 | 1.90 | Have English CCI data for 97-08. Use US data: '88 and '09 (Jan-Nov) CCI data. See Appendix 2 | Not TBM. Construction year pre 1990 |
| 37 | East Sussex | Brighton and Hove Stormwater | Stormwater Sewer | Upper Chalk | EPB | Wirth - Howden | Unreinforced precast concrete lining, 1.5m wide | 1993-1996 | 6 | 6.7 | 5100 | 1.14 | 34,995,363 | 1.58 | Have English CCI data for 97-08. Use US data: Dec '94, Jan '95 and Nov '09 CCI data. See Appendix 2 | Geology. |
| 38 | Brixham, Torbay, | Torbay Sewage Treatment Scheme | Sewer | Limestone | - | Dosco | Fibre reinforced PCC bolted segmental rings | 2004 | 4.25 | - | 485 | 4.97 | 10,249,092 | 1.32 | 04 and 08 UK CCI data. See Appendix 1 | Cost includes constructing pumping station shaft - using drill and blast. |
| 39 | St Leonards on Sea, East | Hastings & Bexhill UWWTD | Sewer | Un-cemented sandstone to limestone | TBM | Herrenknecht Mixshield | - | 2000 | 6.5 | - | 1800 | 6.66 | 77,916,500 | 1.63 | 00 and '08 UK CCI data. See Appendix 1 | M&E work included, Stormwater pumps, DWF pumps, Penstocks, RBI and Odour control |
| 40 | Folkstone, Kent, UK | Folkstone Relief Sewers | Sewer | Water bearing sands | Lovat EPB TBM | Lovat | - | 2000 | 2.44, 2.87, 1.35 | - | 2200 | 5.17 | 32,616,209 | 1.63 | 00 and '08 UK CCI data. See Appendix 1 | Cost includes constructing shafts (1 accommodate pumping station, 1 storage tank, hand drives to connect to existing sewer, 6 shafts for access flow and collection, rising mains and connections for storm and storage tanks) |
| 41 | London | Thames Water | Sewer | - | - | - | - | 1990's | - | - | 80000 | - | - | - | - | Missing Data |
| 42 | Düsseldorf, Germany | Central Sewer Düsseldorf | Sewer | - | - | - | - | 1991-2001 | - | - | 12500 | - | 304,768,883 | 1.52 | Don't have CCI data for Germany. Use US data: '96 and '09 (Jan-Dec) CCI data. See Appendix 2 | Missing data, construction method |
| 43 | Zurich | Glattstollen TBA Zurich | - | Loose ground from deposits from lakes and morains, sand and marlstones. Water pressure max. 2bar | Shielded TBM (Slurry) | - | Single shell segment lining | 2002-2005 | 4.5 | 5.3 | 5700 | - | - | - | - | Missing Data |
| 44 | California | California Inland Feeder Project | Water | Hard rock | TBM | - | Welded steel cylinder pipe, Cellular concrete backfill | [illegible] | 4.8 | - | 12716 | 1.67 | 101,850,418 | 1.28 | 03 and '09 (Jan-Dec) US CCI data. See Appendix 2 | Doesn't include cost of tunnel: Involved with inspection staff management, correspondence and coordination with the contractor and client, reviewing and processing monthly pay requests, preparing monthly reports and cost estimates for change order work, reviewing construction schedules and coordinating third party entities |
| 45 | Cairo | Greater Cairo Wastewater tunnels | Sewer | Alluvial deposits of silts and sands located below the water table. Limestone | - | - | - | 1990 | - | 1.2 - 5 | 50000 | - | - | - | - | Missing Data |
| 46 | Chicago | Chicagoland Underflow | Storm water | - | - | - | Concrete lined | 2002 | 3.51 | - | 3725 | 4.02 | 52,575,847 | 1.31 | 2002 and 2009 (Jan - Nov) data. See Appendix 2 | Includes cost for shafts etc and 40 foot by 60 foot control (service) building at the ground surface; and site work, excavation, site grading, utilities, and final landscaping |
| | Chicago | Chicagoland Underflow | Storm water | - | - | - | Concrete | 2002 | 2.59 | - | | | | | | |
| 47 | Portland, Oregon | East Side Big Pipe Project | Sewerage | - | TBM | - | - | 2011 | 6.705 | - | 9656 | 4.40 | 284,978,427 | - | - | Includes costs for 9000ft new pipeline and 7 shafts |
| 48 | Alkimos | Alkimos Main Sewer | Sewer | - | - | - | - | 2009 | 1.95 | - | 7700 | 14.54 | - | - | - | Cost includes extra piping and excavation of 3M cu.m. excavation for the WWTP |
| 49 | San Diego, CA | - | Sewer | - | - | - | - | 1995 | 3.4 | - | 5490 | 5.00 | 93,328,850.85 | 1.57 | 95 and '09 (Jan - Nov) US CCI data. See Appendix 2 | Higher unit rates associated: there is no separate accounting for the mobilization cost |
| 50 | Seattle, WA | - | Sewer | - | - | - | - | 1995 | 3.66 | - | 168 | 5.45 | 3,351,893.75 | 1.57 | 95 and '09 (Jan - Nov) US CCI data. See Appendix 2 | Higher unit rates associated: there is no separate accounting for the mobilization cost |
| 51 | Boston, MA | - | Sewer | - | - | - | - | 1995 | 4.3 | - | 7650 | 4.81 | 158,306,955.63 | 1.57 | 95 and '09 (Jan - Nov) US CCI data. See Appendix 2 | Inter-island tunnel costs include prorated shares of facilities, spoil disposal, and service costs |
| 52 | Melbourne | Melbourne Main Sewer Replacement | Sewer | - | - | - | - | 2008-2012 | 1.8 | - | 2300 | 34.11 | 141,205,099 | 1.03 | - | Cost includes 6 shafts 10-15m deep and 142m crossing of Yarra River. 2.5km of new local branch and reticulation sewers. Total Cost of project |
| 53 | Shanxi Province, China | Wanjiazhai water transfer project | Water | Karstic limestone and mudstone formations | TBM | - | Hexagonal 'honeycomb' type segmental lining with four segments to a ring | 2002 | 3.6 | - | 109370 | 0.44 | 174,281,461 | 1.31 | Don't have CCI data for China. Use US data: '02 and '09 (Jan-Dec) CCI data. See Appendix 2 | Cost just for PM and project control, specialist. Involved in the development... don't think it's the cost for the tunnel. |
| 54 | Abu Dhabi | Upgrade Abu Dhabi's Sewerage Collection System | Sewer | - | - | - | - | - | 3 | - | 80000 | 3.14 | 754,740,207 | 1.03 | Don't have CCI data for the United Arab Emirates. Use US data: '08 and '09 (Jan-Dec) CCI data. See Appendix 2 | Includes cost for 2 large pumping stations. Missing Data: Construction Year |
| 55 | Lesotho, South Africa | Lesotho Highlands Water Project Delivery Tunnel (South Section) | Water | Sandstone | TBM with drill blast river crossings | - | Precast Concrete Segments | 1993 | 4.5 | - | 16000 | 7.09 | 510,243,901 | 1.64 | Don't have CCI data for South Africa. Use US data: '93 and '09 (Jan-Dec) CCI data. See Appendix 2 | Includes costs for review of Highlands Delivery Tunnel Consultants, initial design, prepare working drawings, supervise construction and commission the work. |
| 56 | Lesotho, South Africa | Lesotho Highlands Water Project Delivery Tunnel (Northern Section) | Water | Soft marl rock | TBM | - | Precast Concrete Segments | 1996 | 4.5 | - | 22000 | 5.52 | 546,689,894 | 1.64 | Don't have CCI data for South Africa. Use US data: '02 and '09 (Jan-Dec) CCI data. See Appendix 2 | Costs include assisting contractors proposals for constructing the tunnel in soft rock. |

*Abbreviations

| | |
|---|---|
| EPB | Earth Pressure Balance (Machine) |
| TBM | Tunnel Boring Machine |
| EPBTBM | Earth Pressure Balance Tunnel Boring Machine |

### 2.3.1 Data Analysis

The tunnelling data in Group 2 was excluded for the following reasons:

1. Year constructed: Any tunnel constructed before 1990.

2. The tunnelling method was not tunnel bore machine.

3. Including cost for pumping stations, etc.

4. Missing data to calculate cost.

5. Geology (e.g. Chalk is easy to tunnel and would result in cheaper cost (The tunnel for Ringsend will be constructed in the bedrock)).

Table 4 below identifies the total number of tunnels excluded for the above reasons. As can be seen the main reasons are due to the construction method and the construction year.

*Table 4: Total Number of Tunnels Excluded from Analysis*

| | Total Number of Tunnels Excluded* |
|---|---|
| **Not TBM** | 26 |
| **Construction year pre 1990** | 21 |
| **Additional Cost** | 19 |
| **Missing Data** | 11 |
| **Geology** | 2 |

*Total will not sum to 56 – due to a number of tunnels being omitted for two or more reasons.

# 3.0 Conclusions

From preliminary design, it was determined that the Ringsend outfall would have a pipe diameter of approximately 5m. Further investigation is required to identify the location of the outfall. As detailed in Section One there are two options for the long sea outfall – for this report it is assumed the length is 10km.

Section Two details the cost analysis of the tunnel information. The cost for constructing tunnels with tunnel bore machines increases as the diameter and length of the tunnel increases. Outlined in section two are four key tunnels used to assess the cost for constructing the tunnel. These tunnels were constructed in the past number of years or are under construction. All four examples are of similar diameter and length for comparison to the Ringsend Outfall.

The aim of this report is to assess the cost for constructing an outfall. An extensive list of tunnels previously constructed was collated and included cost data. Analysis of this data determined an estimated cost of €2.00 - €2.50 /mm DIA/m. (€100 million - €125 million) for constructing the Ringsend Outfall.

# 4.0 References

Australia Tunnel Database Online:

< http://www.docstoc.com/docs/13690975/Australian-Tunnel-Database---ATS---Australasian-Tunnelling-Society>

British Tunnelling Society Online:
<http://www.britishtunnelling.org.uk/database.php>

Charles G. Gunnerson, Jonathan A. French (Eds) (1988), Wastewater Management for Coastal Cities. The Ocean Disposal Option.

Engineering News Record Magazine Online

<http://enr.ecnext.com/coms2/article_echi091101constIndexHi>

Kenny Construction Online:

< http://www.kennyconstruction.com/groups/tunnel/>

Mott Mac Donald Online:

< http://www.tunnels.mottmac.com/projects/>

Northern Ireland Water Online:

< http://www.niwater.com/belfastsewersproject.asp>

Portland Online:

< http://www.portlandonline.com/cso/index.cfm?a=249789&c=31727>

Robert A. Grace (2009). Marine Outfall Construction.

UK Office of National Statistics Online:

<http://www.statistics.gov.uk/downloads/theme_commerce/CSA-2009/Opening-page.pdf>

Underground Professional Services Online:

< http://www.unps.co.uk/>

World Tunnelling Magazine Online:

< http://www.world-tunnelling.com>

# Appendix 1: US Construction Cost Index

US Construction Cost Index:
(http://enr.ecnext.com/coms2/article_echi091101constIndexHi)

## Construction Cost Index History

**HOW ENR BUILDS THE INDEX:** 200 hours of common labor at the 20-city average of common labor rates, plus 25 cwt of standard structural steel shapes at the mill price prior to 1996 and the fabricated 20-city price from 1996, plus 1.128 tons of portland cement at the 20-city price, plus 1,088 board ft of 2 x 4 lumber at the 20-city price.

**ENR'S CONSTRUCTION COST INDEX HISTORY (1908-2009)**

| 1913=100 * Revised | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | ANNUAL AVERAGE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **1990** | 4680 | 4685 | 4691 | 4693 | 4707 | 4732 | 4734 | 4752 | 4774 | 4771 | 4787 | 4777 | 4732 |
| **1991** | 4777 | 4773 | 4772 | 4766 | 4801 | 4818 | 4854 | 4892 | 4891 | 4892 | 4896 | 4889 | 4835 |
| **1992** | 4888 | 4884 | 4927 | 4946 | 4965 | 4973 | 4992 | 5032 | 5042 | 5052 | 5058 | 5059 | 4985 |
| **1993** | 5071 | 5070 | 5106 | 5167 | 5262 | 5260 | 5252 | 5230 | 5255 | 5264 | 5278 | 5310 | 5210 |
| **1994** | 5336 | 5371 | 5381 | 5405 | 5405 | 5408 | 5409 | 5424 | 5437 | 5437 | 5439 | 5439 | 5408 |
| **1995** | 5443 | 5444 | 5435 | 5432 | 5433 | 5432 | 5484 | 5506 | 5491 | 5511 | 5519 | 5524 | 5471 |
| **1996** | 5523 | 5532 | 5537 | 5550 | 5572 | 5597 | 5617 | 5652 | 5683 | 5719 | 5740 | 5744 | 5620 |
| **1997** | 5765 | 5769 | 5759 | 5799 | 5837 | 5860 | 5863 | 5854 | 5851 | 5848 | 5838 | 5858 | 5826 |
| **1998** | 5852 | 5874 | 5875 | 5883 | 5881 | 5895 | 5921 | 5929 | 5963 | 5986 | 5995 | 5991 | 5920 |
| **1999** | 6000 | 5992 | 5986 | 6008 | 6006 | 6039 | 6076 | 6091 | 6128 | 6134 | 6127 | 6127 | 6059 |
| **1913=100** | **JAN** | **FEB** | **MAR** | **APR** | **MAY** | **JUN** | **JUL** | **AUG** | **SEP** | **OCT** | **NOV** | **DEC** | **ANNUAL AVERAGE** |
| **2000** | 6130 | 6160 | 6202 | 6201 | 6233 | 6238 | 6225 | 6233 | 6224 | 6259 | 6266 | 6283 | 6221 |
| **2001** | 6281 | 6272 | 6279 | 6286 | 6288 | 6318 | 6404 | 6389 | 6391 | 6397 | 6410 | 6390 | 6343 |
| **2002** | 6462 | 6462 | 6502 | 6480 | 6512 | 6532 | 6605 | 6592 | 6589 | 6579 | 6578 | 6563 | 6538 |
| **2003** | 6581 | 6640 | 6627 | 6635 | 6642 | 6694 | 6695 | 6733 | 6741 | 6771 | 6794 | 6782 | 6694 |
| **2004** | 6825 | 6862 | 6957 | 7017 | 7065 | 7109 | 7126 | 7188 | 7298 | 7314 | 7312 | 7308 | 7115 |
| **2005** | 7297 | 7298 | 7309 | 7355 | 7398 | 7415 | 7422 | 7479 | 7540r | 7563 | 7630 | 7647 | 7446 |
| **2006** | 7660 | 7689 | 7692 | 7695 | 7691 | 7700 | 7721 | 7722 | 7763 | 7883 | 7911 | 7888 | 7751 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **2007** | 7880 | 7880 | 7856 | 7865 | 7942 | 7939 | 7959 | 8007 | 8050 | 8045 | 8092 | 8089 | 7966 |
| **2008** | 8090 | 8094 | 8109 | 8112* | 8141 | 8185 | 8293 | 8362 | 8557 | 8623 | 8602 | 8551 | 8310 |
| **2009** | 8549 | 8533 | 8534 | 8528 | 8574 | 8578 | 8566 | 8564 | 8586 | 8596 | 8592 | | |

| ANNUAL AVERAGE | | | | | | | |
|---|---|---|---|---|---|---|---|
| **1908** | 97 | **1931** | 181 | **1954** | 628 | **1977** | 2576 |
| **1909** | 91 | **1932** | 157 | **1955** | 660 | **1978** | 2776 |
| **1910** | 96 | **1933** | 170 | **1956** | 692 | **1979** | 3003 |
| **1911** | 93 | **1934** | 198 | **1957** | 724 | **1980** | 3237 |
| **1912** | 91 | **1935** | 196 | **1958** | 759 | **1981** | 3535 |
| **1913** | 100 | **1936** | 206 | **1959** | 797 | **1982** | 3825 |
| **1914** | 89 | **1937** | 235 | **1960** | 824 | **1983** | 4066 |
| **1915** | 93 | **1938** | 236 | **1961** | 847 | **1984** | 4146 |
| **1916** | 130 | **1939** | 236 | **1962** | 872 | **1985** | 4195 |
| **1917** | 181 | **1940** | 242 | **1963** | 901 | **1986** | 4295 |
| **1918** | 189 | **1941** | 258 | **1964** | 936 | **1987** | 4406 |
| **1919** | 198 | **1942** | 276 | **1965** | 971 | **1988** | 4519 |
| **1920** | 251 | **1943** | 290 | **1966** | 1019 | **1989** | 4615 |
| **1921** | 202 | **1944** | 299 | **1967** | 1074 | | |
| **1922** | 174 | **1945** | 308 | **1968** | 1155 | | |
| **1923** | 214 | **1946** | 346 | **1969** | 1269 | | |
| **1924** | 215 | **1947** | 413 | **1970** | 1381 | | |
| **1925** | 207 | **1948** | 461 | **1971** | 1581 | | |
| **1926** | 208 | **1949** | 477 | **1972** | 1753 | | |
| **1927** | 206 | **1950** | 510 | **1973** | 1895 | | |
| **1928** | 207 | **1951** | 543 | **1974** | 2020 | | |
| **1929** | 207 | **1952** | 569 | **1975** | 2212 | | |
| **1930** | 203 | **1953** | 600 | **1976** | 2401 | | |

# Appendix 2: UK Construction Cost Index

UK Office of National Statistics Online:
(http://www.statistics.gov.uk/downloads/theme_commerce/CSA-2009/Opening-page.pdf), Chapter 5, Table 5.4.

## Table 5.4 Resource cost index of infrastructure[1]

| Great Britain | | | | Index 1995 = 100 |
|---|---|---|---|---|
| | | | Derived indices | |
| Year | Quarter | Combined Index | Labour & plant | Materials |
| **1997** | **Q1** | 104 | 106 | 102 |
| | **Q2** | 104 | 106 | 103 |
| | **Q3** | 106 | 108 | 104 |
| | **Q4** | 106 | 109 | 104 |
| **1998** | **Q1** | 107 | 109 | 105 |
| | **Q2** | 108 | 110 | 106 |
| | **Q3** | 109 | 113 | 105 |
| | **Q4** | 108 | 113 | 104 |
| **1999** | **Q1** | 108 | 113 | 103 |
| | **Q2** | 109 | 115 | 103 |
| | **Q3** | 111 | 120 | 104 |
| | **Q4** | 111 | 120 | 104 |
| **2000** | **Q1** | 113 | 121 | 106 |
| | **Q2** | 114 | 122 | 108 |
| | **Q3** | 118 | 126 | 110 |
| | **Q4** | 118 | 128 | 110 |
| **2001** | **Q1** | 117 | 126 | 109 |
| | **Q2** | 118 | 126 | 112 |
| | **Q3** | 120 | 130 | 112 |
| | **Q4** | 120 | 129 | 112 |
| **2002** | **Q1** | 120 | 129 | 112 |
| | **Q2** | 124 | 130 | 120 |
| | **Q3** | 129 | 137 | 122 |
| | **Q4** | 129 | 137 | 123 |
| **2003** | **Q1** | 131 | 138 | 124 |
| | **Q2** | 133 | 138 | 128 |
| | **Q3** | 135 | 143 | 128 |
| | **Q4** | 135 | 144 | 127 |
| **2004** | **Q1** | 136 | 144 | 129 |
| | **Q2** | 142 | 146 | 138 |
| | **Q3** | 147 | 153 | 141 |
| | **Q4** | 148 | 154 | 142 |
| **2005** | **Q1** | 150 | 155 | 145 |
| | **Q2** | 151 | 157 | 146 |
| | **Q3** | 154 | 167 | 143 |
| | **Q4** | 155 | 167 | 144 |

| | | | | | |
|---|---|---|---|---|---|
| **2006** | **Q1** | | 157 | 167 | 148 |
| | **Q2** | | 160 | 169 | 152 |
| | **Q3** | | 166 | 173 | 160 |
| | **Q4** | | 166 | 171 | 162 |
| **2007** | **Q1** | | 168 | 171 | 166 |
| | **Q2** | | 170 | 173 | 167 |
| | **Q3** | | 172 | 178 | 167 |
| | **Q4** | **(r)** | 173 | 181 | 166 |
| **2008** | **Q1** | | 180 | 183 | 176 |
| | **Q2** | | 188 | 188 | 187 |
| | **Q3** | | 197 | 195 | 199 |
| | **Q4** | **(p)** | 192 | 190 | 195 |

**Notes**
p = provisional, r = revised.
1. The resource cost index of infrastructure (FOCOS) gives a measure of the notional trend of costs to a contractor of changes in the cost of labour, materials and plant by application of the price adjustment formulae for civil engineering works (1990 Series) to a cost model for an infrastructure project.

**Source of data:** Construction Market Intelligence, Department for Business, Innovation and Skills
**Contact:** Richard Cornell 020 7215 3628