Remediation Strategies and Water Quality of estuarine impoundments

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A thesis submitted in partial fulfilment of the requirements of the Council of the University of Durham for the Degree of Doctor of Philosophy (PhD)

2003

Abstract

Remediation Strategies and Water Quality of Estuarine Impoundments

Jens Lamping

The implementation of amenity barrage schemes, such as the recent projects on the Tees Barrage and at Cardiff Bay, has gained popularity as part of urban regeneration projects in the UK. The predecessors of these schemes on the rivers Tawe, Lagan and Wansbeck offer valuable information on the impacts and remediation strategies that maybe required to sustain good water quality in the newly created upstream impoundment. Depending on the design, the stored water body is affected by: density stratification; flow velocity reduction; siltation and contaminant accumulation; all of which can lead to periods of poor water quality following barrage construction.

This thesis identifies the principle dynamics leading to low water quality within a partial exclusion system (River Tawe) through numerical and observational analysis of monitoring data recorded over the last nine years. Temporal variation of water quality within total exclusion system (Tees Barrage) is described from continuous monitoring records covering a three-year period. The data for both designs showed significant seasonal variation and flow was identified as a major factor in the process of low water quality development. In addition, the influence of the tidal regime was determined for the oxygen and salinity dynamics in the partial exclusion impoundment.

Due to influx of saline water, partial exclusion systems are likely to suffer from saline stratification, which restricts the mixing processes in the water column and often leads to DO depletion in the lower layers of the impoundment. Several remediation strategies have been applied and proposed to prevent this process, including measures to break-up (Mixers), to prevent (Baffles) or to flush out (Sluicing) the stratified water body. Each of these principles is presented and discussed in this work separately, to determine its feasibility for the future management of water quality in partial exclusion impoundments. The installation of a mixing system in the Tawe impoundment was proved to be successful in breaking up saline stratification and recommendations are given for further adjustment. Flushing was ruled out as an effective measure to improve water quality due to its inability to completely expel low dissolved oxygen water. A future installation involving a floating boom/skirt system was tested in laboratory experiments and the general feasibility of concept was confirmed, whilst design criteria for the prototype were established.

Declaration and Copyright

I confirm that no part of the material presented in this thesis has previously been submitted by me or any other person for a degree in this or any other university. In all cases material from the work of others has been acknowledged.

Signed:

Date: 20 / 11 / 03

Acknowledgements

I would like to thank my supervisor, Dr Fred Worrall, for directing the flow of ideas into the meaningful format of this thesis and for keeping his three minute 'pub promise'.

Special thanks go to Wibke for following me to an island and enjoying it.

The sometimes harsh conditions of fieldwork on the rivers required the sturdy help of Julian Wright, Steve Anderton, Bernard McEleavey, Steve Richardson, Ruth Stadnik, Mareike Liess and Alex Gardiner, all of which deserve a big thanking. On site, my thanks go to Norman Howitt of the Cambois Rowing Club for allowing storage of the RV, Terry Garnick of Wansbeck District Council for assistance on the flushing trial, Allan Slater and his team at the Tees Barrage for rescuing my sonde and providing shelter for the equipment and David Lindsay at the EA for the use of equipment at Low Moor Gauging Station. Furthermore, to Sam Taylor, Keith Naylor, Emma Washburn, Zak Barrett and the Morgan Family for the warm Welsh welcome provided during our Swansea fieldwork days. Facilities and assistance during the flume tank experiments was provided by Dr. Richard Burrows and Kenny Whyte at the University of Liverpool. A substantial amount of data was made available by the Environmental Agency of England and Wales, where I would like to thank Paul Edwards and Claire Buckler for their co-operation throughout the project. In addition, data was provided by the archive services at the BODC and the BADC, for which I am much obliged.

The shortfalls and tricks of dealing with large data sets in strange formats were bravely handled by Dave Stevenson and Garry Wilkinson, while Karl Pedersen provided substantial information on GIS software. They are thanked for long hours of ITS support. Particular mention must go to my proof readers Todd Rae, Paul Gordon, Jen Jennings and Lisa Pollack, who managed to iron out the German edginess and shortened those long sentences, which were initially part of this work. Special thanks go to Nick Baker for last minute formatting efforts.

Finally, this thesis is dedicated to all my friends in Durham, who made the time very much worthwhile which, apart from the aforementioned, include Jon Bennie, Susi Roy, John Collins, The German Society, Sophie Bassett and my band Jurassic Lark. Thanks for special support go to Mrs. Grant and to Norbert for home supplies and international friendship.

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Nomenclature

a	Underflow opening within a baffle/boom-skirt system [cm]
ACD	Above (Admiralty) Chart Datum [m] – Newlyn (approximately lowest astronomical tide)
AOD	Above Ordnance Datum [m]
BADC	British Atmospheric Data Centre
BOD	Biological Oxygen Demand [mg/l]
BODC	British Oceanographic Data Centre
CCS	City and County of Swansea (former SCC)
CSO	Combined Sewer Overflow
DO	Dissolved Oxygen [mg/l] or [%]
EA	Environmental Agency
EAC	Environmental Advice Centre Ltd.
EC	Electrical Conductivity [mS/cm]
FBM	Flow Balance Method
FSI	Freshwater-Saltwater Interface
MHWSL	Mean High Tide Water Spring Level [m ACD]
MNWSL	Mean Neap Tide Spring Level m [m ACD]
TidesOT	Overtopping regime of tides, negative value defines number of non-overtopping tides
NRA	National Rivers Authority (EA, from 1996 onwards)
Q_{sample}	Flow measurement taken at the time of monitoring [m^3/s]
Q_{max24}	Maximum flow measured following the highest tide that preceded monitoring [m^3/s]
Q_{mean24}	Mean flow measured following the highest tide that preceded monitoring [m^3/s]
Q95	95 percentile exceedance flow of daily gauged flows
Q10	10 percentile exceedance flow of daily gauged flows
RV	Research Vessel
SCC	Swansea City Council (later CCS)
TDC	Teeside Development Corporation
z	Height of a layer or interface from tank bottom upwards [cm]
ρ_s	Density of Saltwater [kg/m^3]
ρ_f	Density of Freshwater [kg/m^3]

1 Introduction

This thesis describes the water quality impacts arising from impoundment of estuaries and seeks to evaluate different types of remedial measures that have been introduced or proposed to mitigate these effects. This research was undertaken as part of the EPSRC-funded 'Sustainability in managed Barrages' (SiMBa) project, which aimed to understand the long-term prospects for estuarine impoundments and intends to develop future management strategies for barrage operators. Environmental impacts of estuarine barrages were summarized by Shaw (1995), Jones et al. (1996), Burt and Cruickshank (1996) and Phillips and Williams (2000), while guidelines for future projects were issued recently by Burt and Rees (2001). Descriptions of individual estuarine barrage schemes in the UK and their effects on water quality have been presented by Worrall and McIntyre (1998) for the River Wansbeck barrage, by Shackley and Dyryndra (1996) and Evans and Rogers (1996) for the Tawe Barrage, Burt and Rees (2001) for the Tees Barrage and Crompton (2002) for the Cardiff Barrage scheme (Figure 1-1). Other barrage projects within estuaries worldwide include the Venice Lagoon (Bernstein and Cecconi, 1996) and the Delta scheme in the Netherlands (Scholten et al., 1990).

Figure 1-1 Major barrage schemes in the UK and Ireland, approximate location (reproduced from Burt and Rees, 2001)

1.1. Definitions

The term impoundment refers to the upstream water body, stored as a result of introduction of a closing structure to a flowing stream. A reservoir defines a more specific storage volume of freshwater nature, which mainly fulfils regulatory and drinking water purpose. An impoundment, however, does not relate to this definition and includes fluids from fluvial, marine or even industrial and urban water sources. Impoundments are created by a damming structure referred to as a barrage, barrier or, sometimes, a weir which is installed within the tidal reach of a river. Although the term barrage is often used in a military or political sense, it is also applied as a broader description for the regulating structure which encloses the main river channel within an estuary or inlet. Barrages fulfil the principal purpose of modifying or preventing the up-estuary tide propagation while maintaining a constant water level upstream. They have been erected for centuries to gain land from the sea or make use of the potential energy associated with the tidal regime. These days, barrage schemes act as power generating facilities, tidal surge protection structures, transport assistance, amenity projects or a combination of these.

Tidal Power Generation

Power generation schemes use the potential energy of the stored tidal water to drive low head turbines. The energy gained is approximately proportional to the square of the tidal range and projects are only practical where mean tidal ranges above 4 m are experienced (Burt and Rees, 2001). The cyclical nature of energy production, arising from the diurnal tidal regime, is usually incompatible with the demand period, which is one of the main negative aspects. Nevertheless, the principle appears attractive due to its simplicity and the renewable resource applied.

One of the oldest tidal power generation projects, operational since 1967, is the La Rance Barrage in France which produces 544 GW/yr (Briscoe, 1984; Retiere, 1994). Experience gained from this scheme lead to the proposal of much larger facilities around the world, where high mean tidal ranges are experienced (Table 1-1). Although conditions in the UK are favourable for tidal power generation in many areas (e.g., estuaries of the Bristol Channel), existing barrages, such as the Tawe scheme only include small hydropower generators (96 KW) which are not applied on a regular basis.

Location	Output in GW/yr
Severn Barrage, UK	19,000
Tuguk, Russia	16,020
Mezen Bay, Russia	15,000
Garolim, Korea	8,830
Khambat, India	3,900
Cumberland Basin, Canada	3,300
Mersey Barrage, UK	1,450
Luoyuanwan, China	1,300
Secure Bay, Australia	1,070
Lequinwan, China	1000
Wyre, UK	131

Table 1-1 Prospects for future tidal power barrages (Haws, 1996)

The latest and so far biggest project, the Cardiff Bay Barrage, is described as a multi-function scheme but despite one of the largest tidal ranges in the world (14 m) in this area, the scheme does not generate electricity due to incompatibility of desired impoundment water level and the overtopping regime (Burt, 2001). The largest scheme proposed for the UK is the Severn Estuary Barrage (Table 1-1), which has not yet emerged from the design stage due to economic reasons. Experience from the La Rance scheme (Parker, 1993) and the Annapolis Tidal Power project (Tidmarsh, 1984) showed that environmental implications are generally less than anticipated, since the basic principle is delay of tidal movement and not restriction as in other projects.

Tidal surge protection

The aim behind these barrages is to assist flood control in urban areas where tidal regime and fluvial flows meet. The structure itself, more appropriately referred to as a barrier, is only temporarily applied when the need arises while otherwise being 'transparent'. The movable gates of flood protection barriers are kept open under normal river flow conditions to discharge water and allow navigation. In case of forecasted flood events, the gates are raised to exclude the tides from vulnerable areas upstream.

The surge barrier on the River Thames is one of the largest and most prestigious schemes in the UK (Gilbert and Horner, 1984), while smaller projects exist on the Humber estuary and at Barking Creek (Figure 1-1). Worldwide, the need for flood protection increases due to rising sea levels and larger schemes, such as the Delta Scheme in south-west Holland

and the Venice Lagoon project, give evidence to this trend. Since flood protection schemes are only applied during a very limited period, which is usually characterized by high flows, they are not of major consideration in this study.

Amenity barrages

These types of barrages are constructed to improve the amenity value of the impounded water body and the surrounding area. One of the oldest amenity schemes in the UK is the Wansbeck Barrage, completed in 1975, which was followed later by similar projects on the River Tawe (1992), River Lagan (1994), River Tees (1994) and the largest amenity scheme so far in the UK, the Cardiff Bay Barrage (2002). The recent additions demonstrate that implementation of amenity barrages gained considerable popularity in urban regeneration projects. The main reason for this lies in the replacement of the tidal regime by a constant water level, so that diurnal exposure of 'unsightly' tidal mudflats is removed. The former tidal reach of a river can then be rebuilt into an amenity lake which raises public acceptance and leads to modern riverside developments and improvement of recreational facilities (Hall, 1996). Property and land values are likely to increase` as well as the aesthetics of the area as a whole. Nevertheless, existing projects also showed that the radical change of hydraulic and hydrodynamic patterns, brought by these schemes, is associated with a catalogue of negative impacts.

The implications range from siltation issues (Worrall and McIntyre, 1998; Burt and Littlewood, 2001), algal blooms (Reynolds, 1996), saline stratification and water quality problems (Evans and Rogers, 1996; Kawara et al., 1998), to radical changes of flora and fauna (Dyrynda, 1996; Shaw, 1995). It is generally part of the feasibility studies to address and evaluate these impacts beforehand and some projects, such as the Usk Barrage, have been rejected on grounds of expected environmental implications (Jones et al., 1996). However, experience with the existing schemes showed, that the unique features of estuaries are beyond the grasp of mathematical modelling and that important environmental issues often disappeared in the process of economic decisions. A legislative framework defining long-term responsibilities in combination with guidelines issued for planners (Burt and Rees, 2001) and the results of on-going post-monitoring will eventually improve future projects, but, for now, remedial measures present the only solution to the observed problems.

The immediate threat of water quality impacts to river ecology and environment make this aspect possibly the most important one, which is why this study focuses on the existing and proposed remediation strategies, which were developed to counteract the problems. Three major UK amenity barrages (Figure 1-1, underlined) of different designs were chosen as examples, to describe impacts on upstream water quality and the measures to mitigate those effects. The main difference between the designs is the ability of saltwater to enter the impoundment via overtopping of the crest during spring tides. In this case, the structure is termed part-tide or partial exclusion barrage, whereas a design that completely prevents tidal propagation upstream, is referred to as a tidal or total exclusion barrage. Two of the discussed examples are partial exclusion barrages, with the River Wansbeck Barrage being overtopped by approximately 30 % of all incoming tides and the River Tawe Barrage showing a 71 % overtopping rate. The total exclusion system and its effects on upstream water quality, is represented by the Tees Barrage, which was designed to completely separate the tidal waters from the upstream freshwater river (W.S. Atkins (Northern), 1989).

1.2. Principle dynamics

An estuary is generally described as zone of high energy input, since tidal frame and fluvial source meet at this point, which causes considerable amount of turbulence (Dyer, 1997). Depending on their tidal range, estuaries can be classified into four categories (Davies, 1964), which are:

- Microtidal < 2 m
- Mesotidal < 4 m, > 2 m
- Macrotidal < 6, > 4 m
- Hypertidal > 6 m

The tidal class in combination with the predominant fluvial conditions then determines the pattern of up-estuary saltwater propagation. Due to the density difference between marine saltwater ($\rho_s \approx 1033$ kg/m^3) and riverine freshwater ($\rho_f \approx 1000$ kg/m^3), density stratification often occurs within an estuary. The resulting salinity regime is the basis for another classification, which divides into well-mixed, partially mixed/stratified, fjords type and highly stratified systems (Cameron and Pritchard, 1963; Turner, 1973). Most of the UK estuaries are of partially or highly stratified type, with a so called 'salt-wedge' travelling upstream on the rising tide (Hansen and Rattray, 1966; Uncles and Stephens, 1996). Ebb tide advection currents and the amount of river discharge then determine the scope of salt-wedge flush out. Some pockets of salt water remain in depressions of the river bed but are regularly replenished within the tidal cycle and eventually experience a flush out or dilution, depending on the river flow conditions (Schroeder et al., 1990; Kurup et al., 1998; Simpson et al., 2001).

A barraged estuary shows a completely different behaviour as a result of the disturbed hydrodynamic conditions brought by the blocking structure. The marine and fluvial energies entering the estuary are largely reduced and energy reaches minimum levels where former maxima existed (Figure 1-2).

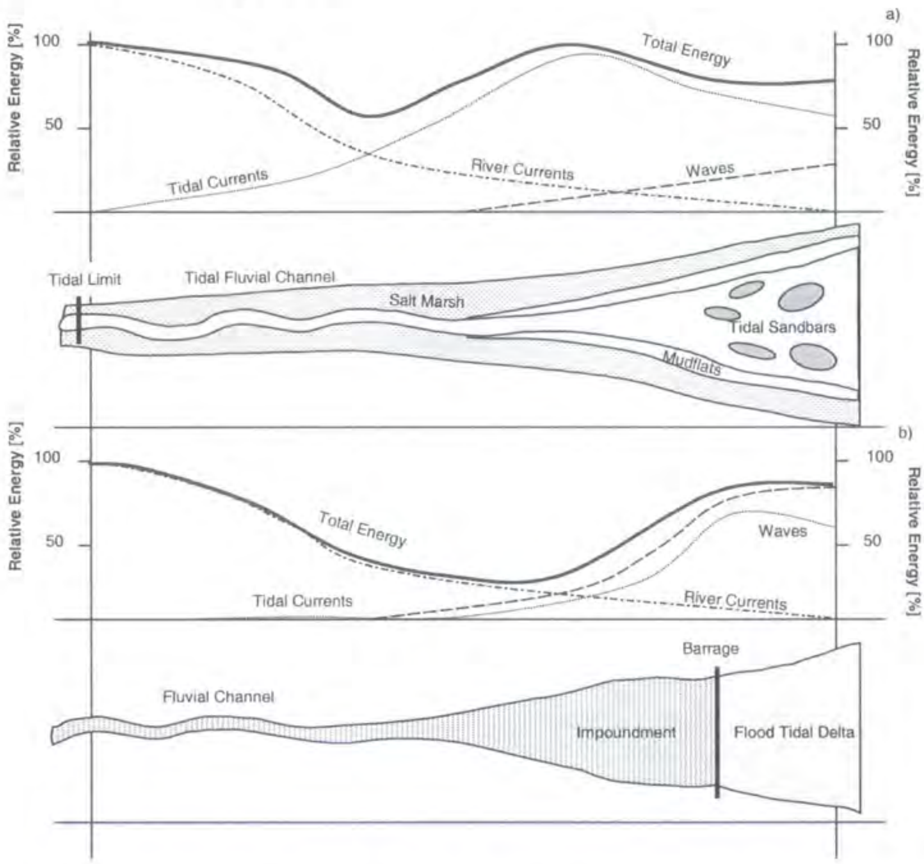

Figure 1-2 Distribution of relative energy pre-and post-impoundment (after Dyer, 1997)

This can lead to long periods of stagnant conditions within the impoundment depending on the river discharge and relative height of the barrage crest to the tidal range. In partial exclusion systems, it was observed (Evans and Rogers, 1996) that the overtopping seawater plunges to the ground on entry which forces the lighter freshwater to flow seawards on top (Figure 1-3). Although mixing of the two water bodies initially occurs in the vicinity of the barrage, it is quickly replaced by a density interface, also known as the halocline, which allows the characteristic salt wedge to travel upstream below the fresh water surface layer.

Upstream progression of the salt wedge is dependent on spring tide levels, intensity of the freshwater flows and geometry of the river bed (Dyer, 1997).

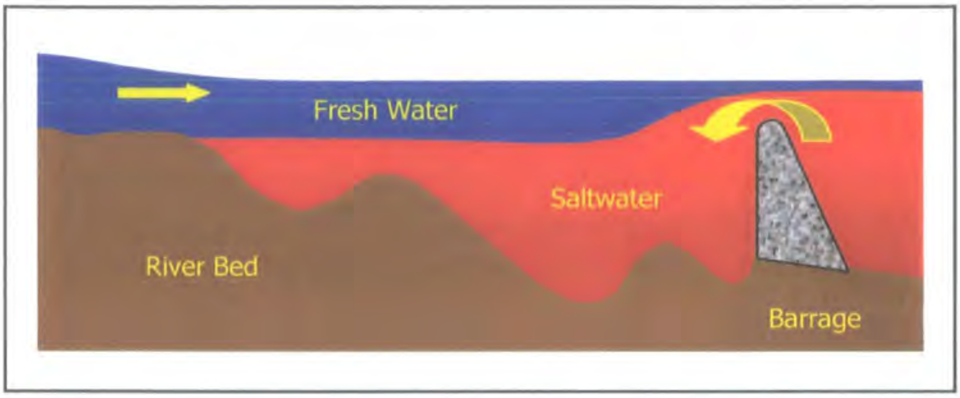

Figure 1-3 Saltwater intrusion into an impoundment during spring tide overtopping (schematic)

After overtopping stops, the freshwater starts to erode the static lower layer at the halocline. The rate of erosion is related to the density difference and the velocity of the layers in the vicinity of the interface (Schijf and Schönfeld, 1953; Harleman, 1961).

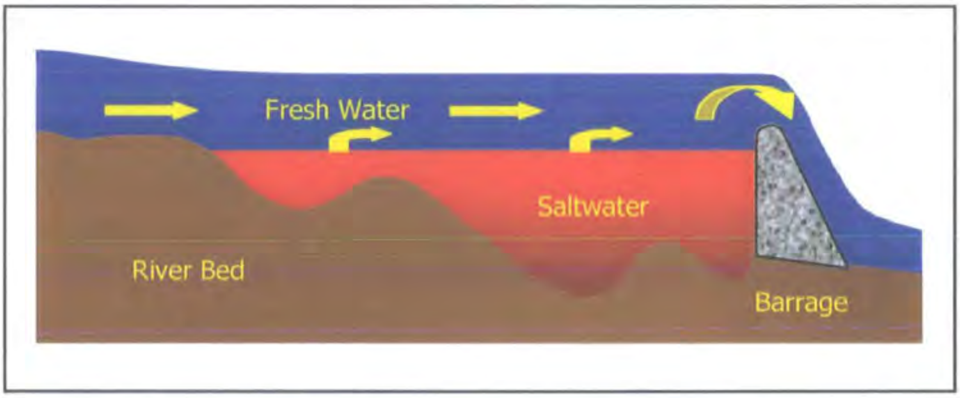

Figure 1-4 Erosion of salt-wedge during non-overtopping conditions (schematic)

In an open estuary, the salt water erosion would be accompanied by the retreating dynamics of the salt wedge during ebb tide, requiring less energy for the freshwater to flush out the seawater (Godin, 1999; Simpson et al., 2001). In contrast, impounded estuaries are characterized by the captive nature of the stratified water body which can be severely affected by the stagnant conditions over the time. In the absence of high river flows to provide erosion at the interface and without saline restoration through overtopping tides, rapid oxygen reduction occurs within the salt water body. Studies by Henry (1992), Worrall and McIntyre (1998) and Edwards and Bishop (2000) indicated that this is mainly due to sediment oxygen demands, which are particularly high in the lower parts of impounded estuaries, where increased settlement of particulate organic matter is observed. The overlain freshwater layer forms a barrier for atmospheric re-oxygenation while the strong density interface does not allow oxygen exchange within the water column, although the freshwater is saturated at most times. Conditions degrade further when low fluvial flows coincide with long periods of non-overtopping during neap tide cycles (Figure 1-5).

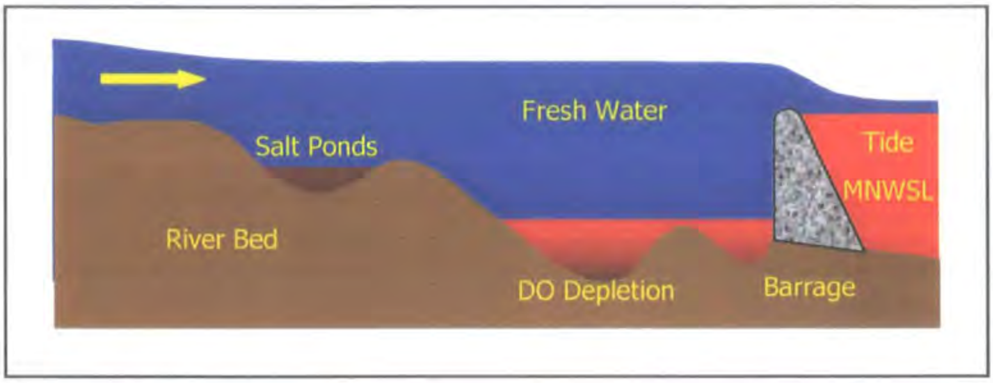

Figure 1-5 Prolonged replenishment of salt wedge during neap tide cycle (schematic)

Substantial DO reductions occur in pockets of salt water that are trapped in river bed depressions by the falling halocline, which has detrimental effects to aquatic inhabitants and can also be accompanied by recycling of metals and nutrients from sediments (Evans and Rogers, 1996). In order to prevent the impacts of saline stratification, several remedial measures have been suggested, which include mixing (Figure 1-6), flushing of the system and prevention of saline intrusion by baffles.

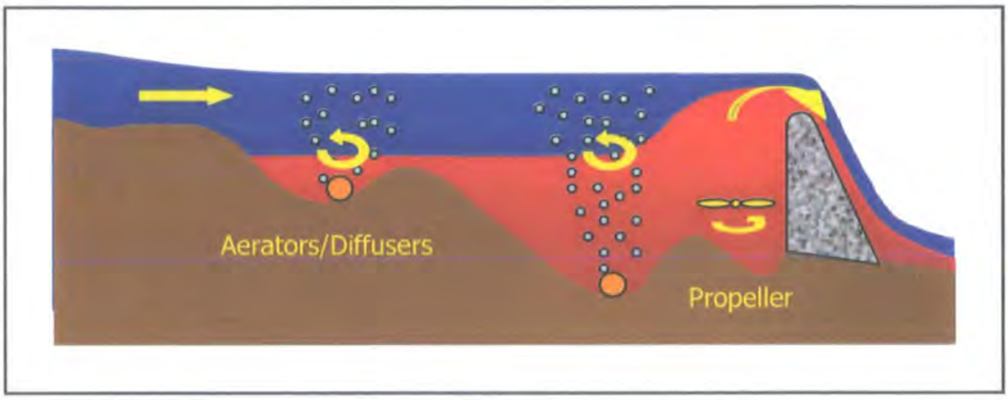

Figure 1-6 Mixing devices in stratified estuarine impoundments (schematic)

However, a comprehensive assessment of these devices has not been carried out yet and this study aims to address this shortfall through discussion of field and laboratory installations (Section 1.3).

Although total exclusion systems are not directly considered as remedial measures, they could be seen as a response to the experience gained from partial exclusion designs. The recently built amenity schemes on the River Tees and at Cardiff Bay follow the concept of completely excluding tidal waters, to prevent the negative impacts of density stratification. However, they could be subject to a weaker form of density stratification as a result of temperature differences, which was observed in river impoundments (Wunderlich, 1971). These freshwater impoundments, or also referred to as reservoirs, have thermal regimes similar to lakes with the addition of substantial inflow and outflows (Wunderlich and Elder, 1967). Sunlight warm up of the epilimnial layer during the spring-summer season can create large variations in temperature within the water column (Figure 1-7), as observed in large freshwater impoundments on the Tennessee River (Churchill and Nicholas, 1967).

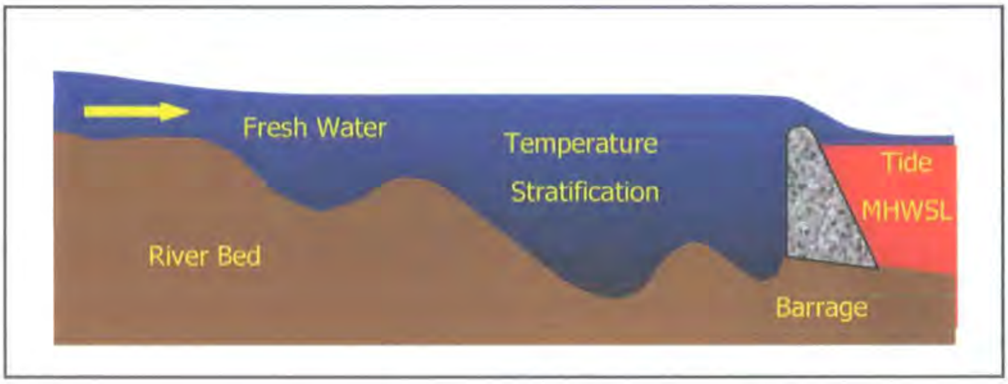

Figure 1-7 Total exclusion system (schematic)

Petts (1984) suggested that impoundments of more than eight meter depth are subject to thermal stratification, depending on residence time and withdrawal point. The Tees Barrage impoundment, which is included in this study, has a maximum impoundment depth of eleven meters and might therefore experience thermal stratification during low flow summer months. In addition, other water quality impacts might be implied as a result of river impoundment, as described in post-impoundment monitoring studies by Van Sluis and Lijklema (1984) for the Nakdong River in China, and Chang and Wen (1997) for the Feitsui Reservoir in Taiwan. However, a continuous water quality record has not been produced for a freshwater impoundment so far, which was attempted as part of this study to examine water quality variation and their controls.

1.3. Scope of this work

The research presented in this thesis is primarily concerned with remediation strategies that evolved as a result of the experienced water quality impacts in partial exclusion impoundments. This study lists and evaluates state of the art design and operational measures that are applied and proposed within UK amenity barrage schemes and evaluates their performance. The main aims of this study are defined as follows:

- To identify adverse water quality effects arising from barrage construction in partial exclusion systems and to determine the role of freshwater inflow.
- To describe and evaluate remediation principles that mitigate the water quality impacts in partial exclusion systems.
- To describe temporal variation and impacts on water quality in a total exclusion system.

Each of the following chapters covers one remediation principle, although the total exclusion system described in chapter 5 has to be regarded as a remediation design, rather than an operational measure. The chapters are subdivided into the general layout of Introduction, Methods, Results, Discussion and Conclusion, so that they can be distributed separately as a possible guideline for present barrage operators and promoters of future barrage schemes. The work, carried out over a period of three years, included:

- Water quality surveys covering a wide range of parameters for different operational conditions within a partial exclusion impoundment.
- Continuous water quality monitoring within a total exclusion system to identify temporal variations.
- Acquisition of large amounts of historical survey and monitoring data for a partial exclusion system to identify impact on water quality.
- Flume tank experiments to assess feasibility of an alternative remediation concept for partial exclusion systems.

The results of this work are reported in 4 Chapters which are outlined as:

<u>Mixing Devices</u>

The events leading to the implementation of an aeration scheme following impoundment of the River Tawe are described. The role of freshwater flow and tidal regime on the stratification patterns in the impoundment is analysed and performance of the remediation scheme is assessed.

Boom/Skirt System

A novel boom/skirt system was tested for its hydraulic and hydrodynamic feasibility as a management option for salinity stratified impoundments during flume tank experiments. The results outline the main parameters to be considered for future field installations.

Flushing

The classic approach for water quality improvement of impoundments is assessed during a flushing trial in the River Wansbeck. Results are discussed in light of hydrodynamic behaviour of stratified water bodies and additional observations made during a similar trial on the River Tawe.

Total Exclusion

Observations from a three year continuous water quality monitoring programme in the total exclusion impoundment of the Tees Barrage are presented. The effect of the barrage and the role of freshwater flow on temporal variation of water quality parameters is discussed.

Conclusions

A summary of the individual conclusion sections is given including a brief discussion on implications of these results and further research needs that evolved from the finding.

2 Mixing Devices – The Tawe impoundment

2.1 Introduction

The aim of this chapter is to introduce the principle dynamics within a density-stratified impoundment and to discuss the application of mixing devices as a remedial strategy for water quality improvement. The example considered is the Tawe Barrage, which became operational in 1992 as a partial exclusion scheme that converted the partially mixed estuary with a macrotidal regime into a stagnant, highly stratified amenity lake (Shackley and Dyrynda, 1996). The results of several post-impoundment water quality surveys by the NRA/EA Wales (Evans and Rogers, 1996) and the University of Wales (Dyrynda, 1994) indicate the presence of a static salt wedge under non-overtopping conditions that leads to severe deterioration of DO levels in the impoundment. Similar impacts were experienced in the River Lagan Impoundment, Belfast, where a partial exclusion weir was commissioned in 1937 (Wilson, 1985) and a new barrage scheme became operational further downstream of the old weir in 1994 (Watts and Smith, 1994; Millington, 1997). The formation of a stable two layer system considerably restricted mixing processes within the water column of the upstream impoundments, causing DO depletion of the saline layers during non-overtopping periods and low flows (Section 1.2). The regular occurrence of these severe impacts on water quality led to the implementation of artificial mixing devices, which have been applied successfully to reduce temperature stratification in reservoirs and lakes (Cooke et al., 1993; Oskam, 1995). Although the so-called 'Aerators' showed no significant effect on the degree of saline stratification within the Lagan impoundment (Burt and Rees, 2001), the installation of a larger scheme started in 1998 on the River Tawe. Results from the trial period in 1997 and from the first phase of a two phase installation scheme in 1999 indicated that the equipment was successful in providing mixing of the layers (Edwards and Bishop, 2000; Taylor et al., 2002). A review of the complete system, however, including the second and final phase of installation, was still missing; this study addresses the issue. Furthermore, all available monitoring data for the period prior to the installation of the remediation equipment was analysed for temporal and spatial variations in salinity patterns and DO conditions with respect to flow and tidal regime. The aim was to identify the importance of factors leading to the degradation of water quality in the impoundment and to give operational recommendations for times when the mixing equipment could be applied most effectively. The following section gives a summary of events that led to the introduction of the remediation scheme, which is followed by analytical sections covering the pre- and post-aeration data.

2.2 The River Tawe

2.2.1 Hydrology

The river Tawe is a major physical feature of the Lower Swansea Valley, draining an area of some 227 km^2. It originates in the Brecon Beacons and flows 48 km in a south westerly direction from its source at Moel Feudwy (590 m above sea level) to Swansea Bay in the Bristol Channel (Figure 2-1).

Figure 2-1 River Tawe catchment map

Within the lower Swansea Valley, the Tawe becomes a narrow and straightened estuary channel confined through large scale tipping and by docks and developments along the river banks. The Tawe has a flow range from less than 1 m^3/s to more than 200 m^3/s, which have been observed after exceptionally heavy rainfalls. Hence, several publications (Hilton, 1967; Dyrynda, 1994) describe the Tawe as a 'flashy' river.

The flashy nature of the Tawe results from fluvial flows, which consist primarily of surface runoff and the melting of accumulated snow in the catchment. In addition, the mountainous character of this relatively small catchment leads to heavy annual rainfalls with an uneven distribution, further promoting high and rapid rates of runoff. As a result, the base flow is comparably low with a 75 exceedance flow of 3 m^3/s (Figure 2-2), while the mean daily flow is given as 12.13 m^3/s for a 46-year-period (National River Flow Archive, 2003), raising its sensitivity towards pollutants contributing to the flow.

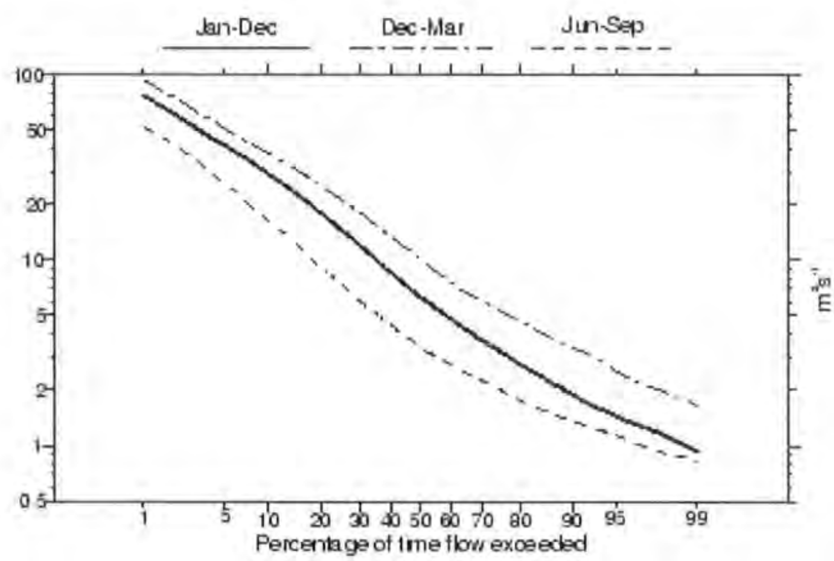

Figure 2-2 Flow duration curve for gauged daily flows 1957-2003 (National River Flow Archive, 2003)

2.2.2 History of the Tawe Estuary

The industrial history of the Lower Swansea Valley began in 1717 with the opening of the first copper smelting works. By 1850, the area had become the world's metallurgical capital. During the late 19th century, the non-ferrous smelting works were replaced by iron, steel, tinplate and chemical industries, which then moved away towards the larger integrated plants on the coastal plains. This migration of industry continued until the 1960s, leaving behind Britain's largest single area of industrial dereliction (Bromley and Humphrys, 1979), including 285 ha of abandoned houses, 7 million tons of slag and furnace waste, and resulted in a highly polluted river. Rapid development and the rise and fall of industries led to substantial alterations of the river course, which, prior to 1800, meandered through the city of Swansea and discharged into an open bay area called Fabians Bay (Figure 2-3). With the start of the industrial developments, the bay was transformed into a harbour. The original meander was cut across by an artificial straight channel, known as New Cut, to convert the isolated bend into a tidal dock. With the expansion of the lower estuary docks, these North docks were later abandoned and infilled.

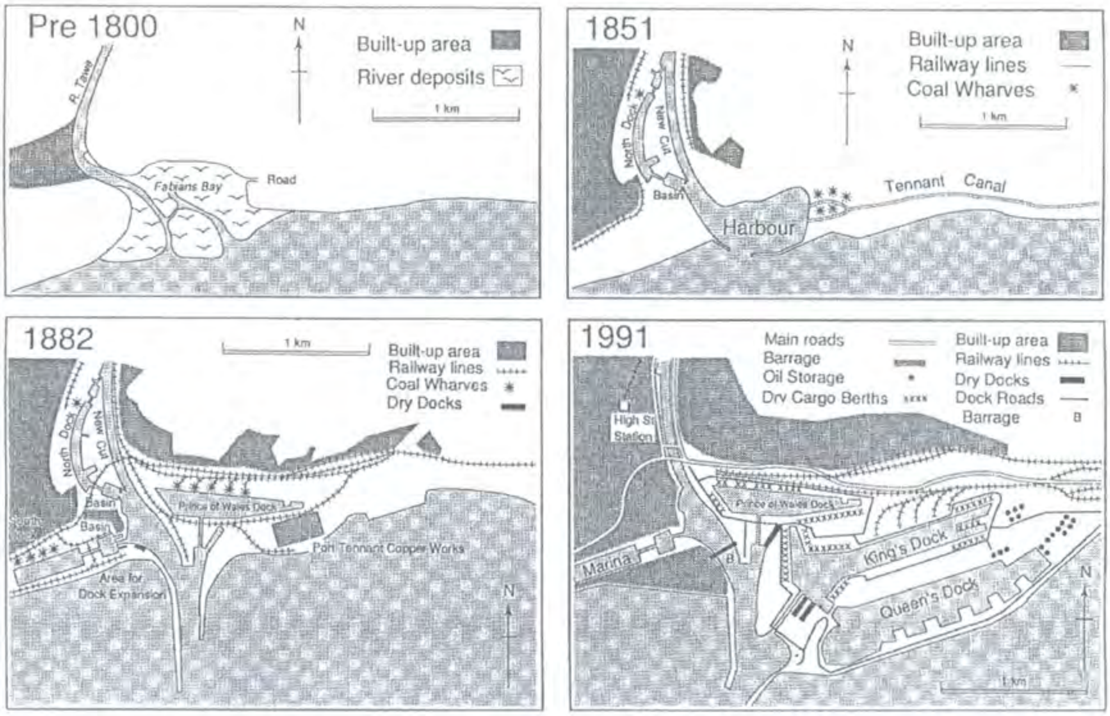

Figure 2-3 Evolution of the Tawe Docks (Dyrynda, 1994)

2.2.3 Pre-barrage physical and water quality characteristics

Prior to barrage construction, the tidal influence of the physical estuary extended from the breakwaters of Swansea Bay to the tidal head at Beaufort weir, 7 km upstream. The original estuary was partially stratified and experienced a macrotidal cycle with a maximum tidal range of 10 m at the river mouth, resulting in regular flushing of water and exchange of sediment (Dyrynda, 1990). The sharp rise and fall of water level exposed the sediment to the atmosphere on a regular basis, allowing sufficient oxygen supply for degradation processes.

The high and rapid runoff conditions lead to the absence of any form of water storage capacities to abstract process water or to use the reservoir for dilution of discharges. The sensitivity of the system towards effluents and the historically contaminated sediments is apparent during low summer flows. Surveys carried out for the Lower Swansea Valley

Project in 1964, and by the Oceanographic Department of the University of Wales, Swansea, from 1981-1988, already pointed out the interrelationships between low flows and the presence of readily oxidizable wastes in creating reduced oxygen conditions in the stratified Tawe estuary (Hilton, 1967; Broyd et al., 1984; Dyrynda, 1990). At this point, low oxygen conditions were limited to the upper reaches during low tide conditions, suggesting that stagnant seawater was partly responsible for the findings. The feasibility report for the construction of a barrage across the Tawe estuary (Atkins Research and Development, 1983) did not anticipate the significance of these results enough during their modelling. Furthermore, their recommendations were impaired by ongoing decisions over the final height of the barrage, which left them either with a total or partial exclusion system for their assessment. Finally, a budget based decision was made to set a barrage height, which allows the intrusion of seawater under high water spring tide conditions, leading to the stratified impoundment described below.

2.2.4 Key dates in the Barrage Development

1961 – The Lower Swansea Valley Project evolved as a partnership between Swansea Borough Council (later City and County of Swansea) and the University College of Swansea (later University of Wales, Swansea), in conjunction with a number of engineering and agricultural companies, to develop programmes for environmental improvement and restoration of the derelict areas along the river. After four years of surveying and experiments, suggested actions involved landscaping, creation of leisure and commercial parks and the development of an amenity area by the means of a 'movable weir', which submerges the derelict shores of the estuary (Hilton, 1967; Bromley and Humphrys, 1979; Bridges, 1988).

1978 - Swansea City and West Glamorgan County Councils commission W.S. Atkins to undertake a feasibility study for a barrage across the river Tawe, with an envisaged barrage height of 5.10 m AOD (= 10.10 m ACD). By the time the numerical models were calibrated, tested and production runs completed for this total exclusion scenario, the suggestion was made by the council to lower the barrage height considerably. Since the new height would allow the intrusion of seawater, which would alter the outcome of study substantially, W.S. Atkins recommended termination of the study until a definite decision on the barrage height has been taken.

1984 - Swansea City Council (Tawe Barrage) Bill is introduced to the Parliament and heard by a House of Lords Committee.

1985 - Petition in the House of Lords against the Swansea City Council (Tawe Barrage) Bill by the Pontardawe & Swansea Angling Society, Inco Europe Angling Section – Clydach Refinery and The Tawe & Tributaries Angling Association. The Petitioners raise concerns about their property rights, fish migration, water quality aspects and fishing prospects, requesting protective measures to be included before passing the Bill. (House of Lords Session 1984-1985, Petition against Tawe Barrage Bill).

1986 – Royal assent for the Swansea City Council ('Tawe Barrage') Act granted *'to construct a barrage and lock across the river Tawe in the City of Swansea and other works, and to aquire lands; to provide for the control and development of part of the river for recreational navigation; to confer further powers on the Council; and for other purposes'* (Swansea City Council Act, 1986)

1987 – W.S. Atkins Engineers produce a preliminary design followed by a period of consultations and approvals.

1988, September - The City Council approves the final detailed design and decides on financing the project, which is supported by a sum of £ 4.6 million out of the European Regional Development Fund. Tenders for the several works are invited shortly afterwards and Shepard Hill & Co. from Cardiff are awarded with the civil engineering contract (£ 12,776,000).

1989, February – Barrage Construction (Figure 2-4) Works commence on site starting with:

- Stage 1 (-early 1991) construction of Navigation lock within a temporary doubled walled coffer dam while the flow passes in a restricted channel.
- Stage 2 (-late 1991) construction of primary wear foundation and fishpass within a central enclosure while flow is diverted to navigation lock and temporary eastern channel.
- Stage 3 (-June 1992) construction of secondary weir foundation and eastern abutment within an eastern enclosure while flow passes mainly through navigation lock at high velocities.

- Stage 4 (-July 1992) removal of all coffer-dams and installation of prefabricated weir crest blocks. Closure of the lock to raise impoundment water level to crest height on the 26/07/92.

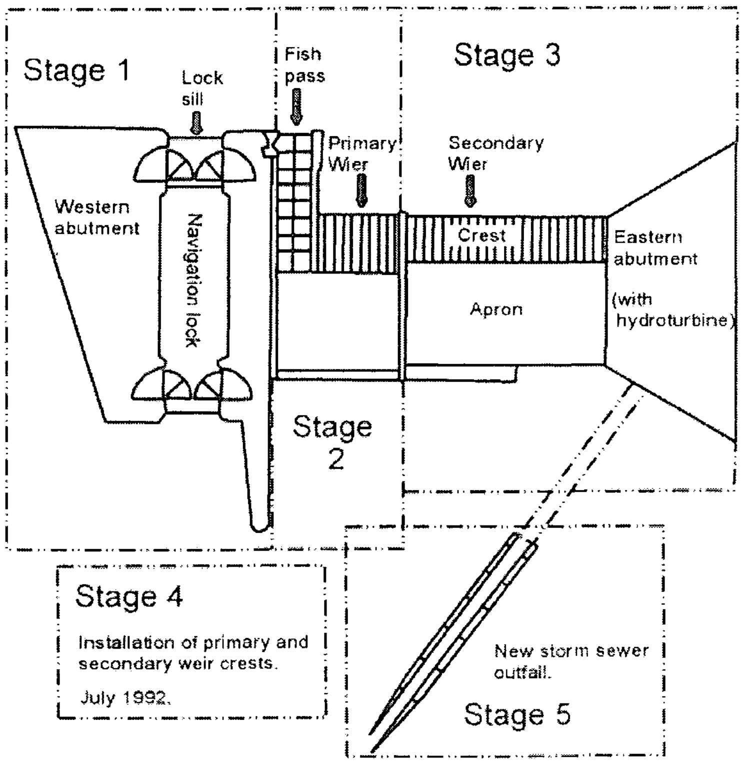

Figure 2-4 Plan views showing phases of barrage construction (Dyrynda, 1994)

1992, August - Official commissioning of the Barrage after 3 years construction time.

1993 – Concerns about the water quality in the impounded area are expressed during several meetings and in correspondence between the NRA and SCC after saline stratification and oxygen depletion was recorded during several monitoring runs. Emergency plans for the evacuation of boats and the drainage of the impoundment through operation of low level penstocks, incorporated in the barrage design (Appendix B), are discussed.

1994 - Report of a study (Dyrynda, 1994) undertaken during the period of 1990-1994 by the University of Wales, Swansea, in co-operation with World Wide Fund For Nature -UK, Countryside Council Wales and the NRA indicates negative impacts on water quality, vegetation and sedimentation following barrage completion.

1998, August – Test trials with an aeration/mixing system installed by the Environmental Advice Centre Ltd. (EAC) to assess performance in breaking stratification. The system includes 15 'aerators' installed temporarily at an upstream site and one 'Banana Blade' mixer adjacent to the fish pass upstream of the barrage. After successful completion of the trials, CCS proposed to award the contract to EAC to supply a full mixing/aeration system to be installed in two phases.

1999 – Phase 1 Installation of 29 aerators in addition to the existing 15 trial aerators.

2000 – Phase 2 concludes the remediation scheme with another 43 aerators installed.

2.2.5 Post-barrage water quality conditions: Dealing with stratification

The installation of a major structure like the Tawe Barrage changed both the physical nature and water quality characteristics of the estuary. Macrotidal conditions were replaced by low energetic saline inputs overtopping the barrage only during high water levels of spring tides. This still counts for approximately 71 % of tides (Jones et al., 1996), but reduces the period as well as the dynamics involved in the tidal cycle (Section 1.2).

2.2.5.1 Flow Spates & Sluicing – The early years

Profiling surveys carried out by the NRA on the Tawe estuary directly after impoundment already indicated the presence of an extensive static salt wedge fixed behind the barrage, which was covered by an overlain freshwater layer. The monitoring of 06/08/92, 24/09/92 and 01/10/92 (Figure 2-5) showed a highly stratified impoundment, with saline intrusion up to 4.25 km distance from the barrage, accompanied by reduced DO levels in the bottom saline layers.

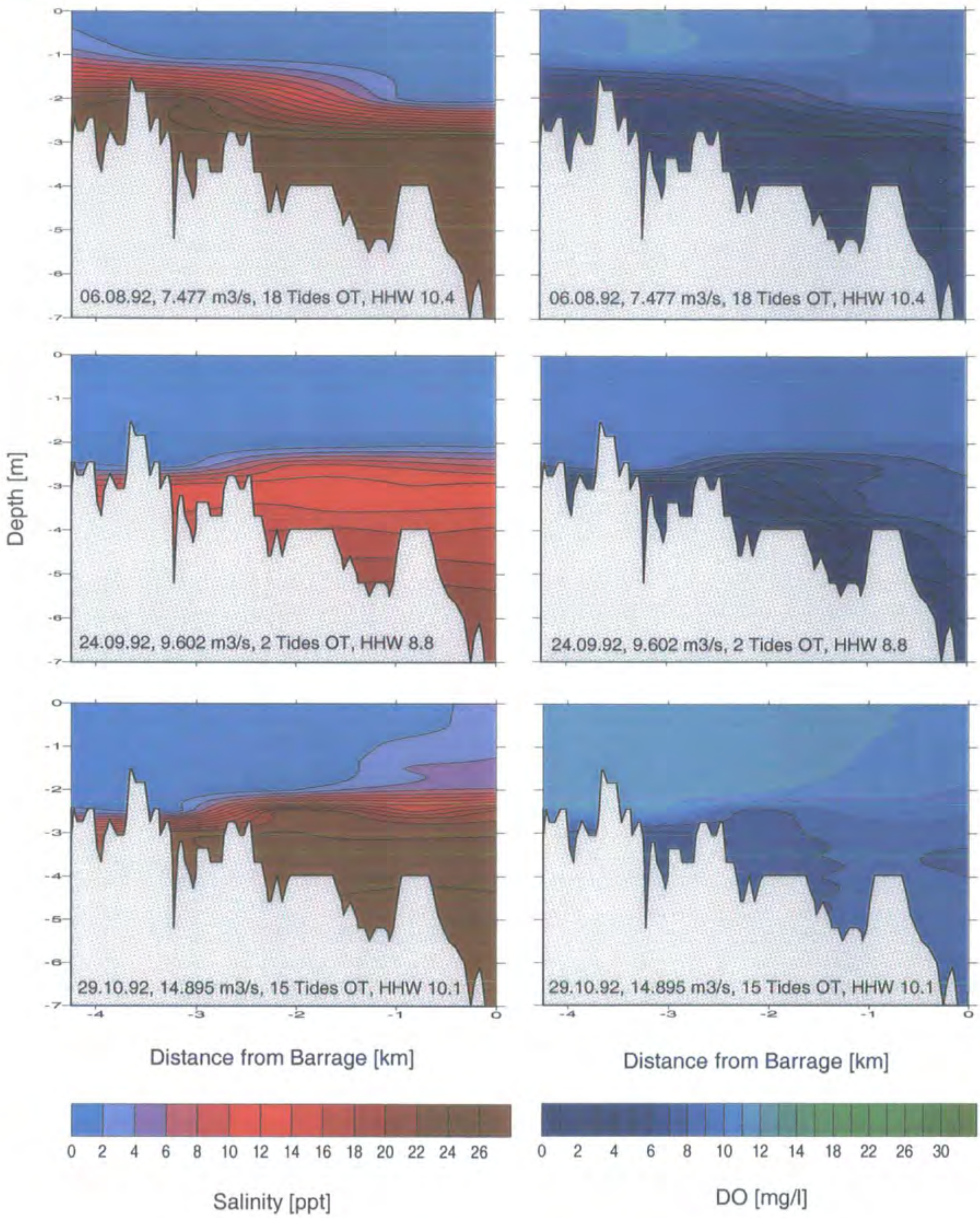

Figure 2-5 River Tawe Salinity and DO contours after impoundment in 1992

Fluvial flow spates, however, marked the end of the summer months (Figure 2-6), and alleviated the water quality problems, so that no further action was required to improve water quality in 1992.

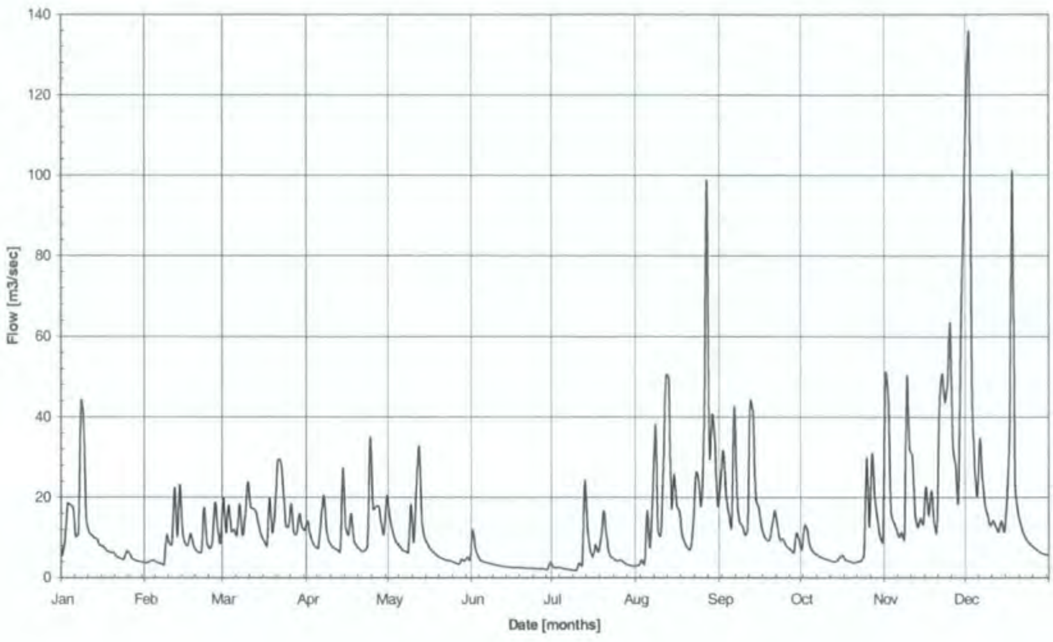

Figure 2-6 Tawe Flow Data 1992 (EA Wales)

In 1993, the effect of saline water release through penstock sluicing was investigated to counteract the degradation of the impounded water quality. The sluicing trial was carried out from 30/03/93 - 02/04/93, with engineers operating two penstocks at full discharge while an NRA/SCC team monitored the water quality conditions. Although the trial was partially successful in reducing upstream penetration of the saline wedge, the water volume required to operate the scheme as a remedial measure proved to be immense. On the basis of calculated penstock operations times, and with particular respect to the dry summer conditions, the effectiveness of sluicing for managing the impoundment water quality was doubted. The results of the sluicing trial are discussed in more detail in Section 4.2. It was concluded, that an alternative long-term solution for the management and protection of water quality, especially in the upper impoundment, was required and that monitoring should be continued for further investigation of water quality variation. Furthermore, an emergency plan was requested by the NRA to drain the impoundment completely, allowing a water exchange in the event of a pollution incident or a radical drop in oxygen levels. The option is allowed for in the Swansea City Council (Tawe Barrage) Act (1986), authorizing the NRA to request this drastic measure. Nevertheless, this action has serious logistical implications for the marina (e.g., boat damage, lock operation) and impairs the operation of the fish pass. Emergency procedures were discussed and a protocol for a 24 hour action plan was agreed between the NRA and SCC, with the costs incurring during such an incident to be borne completely by SCC.

By the end of 1993, monitoring still showed a persistent stratification with DO depletion of the lower layers. Nonetheless, the conditions were not as bad as anticipated due to regular occurrence of flow spates (Figure 2-7) and an intensive use of the low level penstocks as part of the lock gate operation.

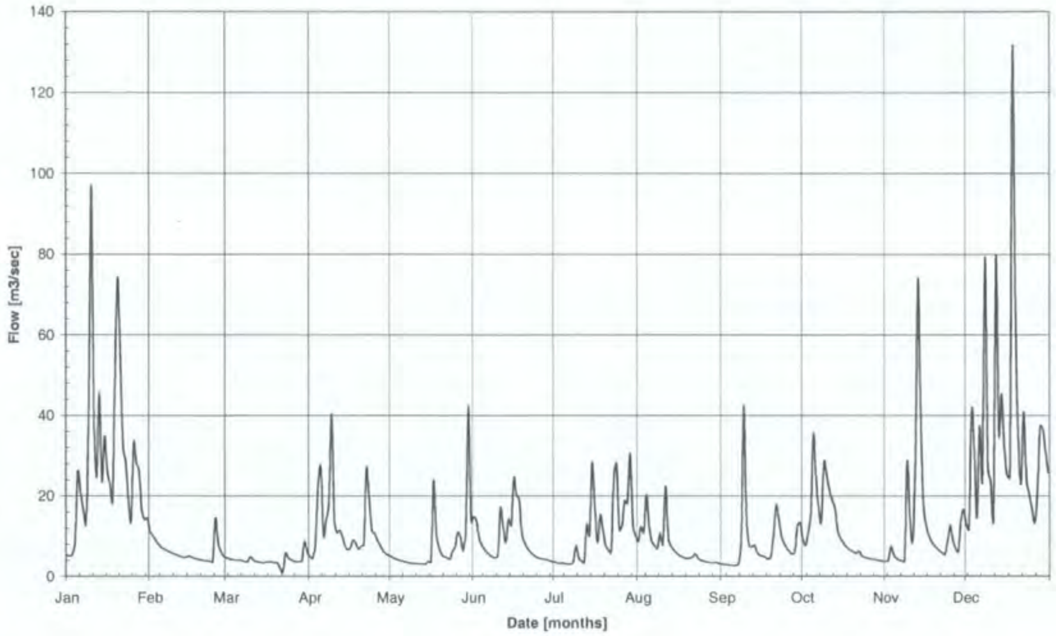

Figure 2-7 Tawe Flow 1993 (EA Wales)

In 1994, the water quality of the impoundment deteriorated substantially due to low freshwater inputs coinciding with neap tides on several occasions. This led to an official request by the NRA in June for a total drainage of the impoundment in accordance with Tawe Barrage Act. Considering the involved logistical problems for the Marina, SCC responded with maximum sluicing on a regular basis, after overtopping started, to reduce the saline influx right at the barrage. At sites further upstream, where low oxygen conditions were persistent, a pump and a compressor, operated from a boat, were used as additional remediation devices to mix and re-oxygenate the lower layers. Nevertheless, these measures proved to be insufficient in solving the problem while public interest and the concerns by the NRA were growing (Hayward, 1994). Claims by the NRA, that the degraded water quality conditions posed a risk to fisheries and aquatic life, led to the proposal of regular sluicing regimes to counteract the summer episodes of hypoxia. Sluicing schedules were established by SCC according to the tidal conditions and required lock operation times; these were carried out as routine sluicing operation by the lock operator.

The joint water quality monitoring programme between SCC and the NRA was intensified in 1994 to gather more information about the dynamics in the impoundment, making this the largest data set (Table 2-2) available for the Tawe impoundment.

In 1995, penstock sluicing was included in the routine operation of the barrage from March onwards to minimise the risk of water quality deteriorations from the start of the season. The issued sluicing schedules were applied in accordance with the available freshwater flow and minimum water heights required within the impoundment. Due to the encountered limitations of penstock operation as a remedial measure, consideration was given to direct remediation devices that could be applied locally. A joint visit to the Lagan impoundment, Belfast, where mixing devices were installed in 1994, was undertaken. In addition, an emergency response plan was established which specified the actions required for certain water quality conditions, and high-risk periods were identified based on the observations made during the summers of 1993 and 1994 (Rogers and Bryson, 1994; Dyrynda, 1994). These high risk periods were targeted for intensive monitoring and penstock operation, to carry out more sluicing assessments unimpaired by tidal replenishment. The very dry conditions from April to September 1995 (Figure 2-8), resulted in limited sluicing possibilities and competing water demands within the impoundment, since lock operation, fish pass flow and sluicing had to be satisfied. The Hydroelectric Power Plant (ITT EL 7570, max flow: 1.7 m^3/sec, max. output: 96 kW) incorporated in the barrage design and scheduled to go online, was now used to back-pump water into the impoundment for several weeks to maintain water levels.

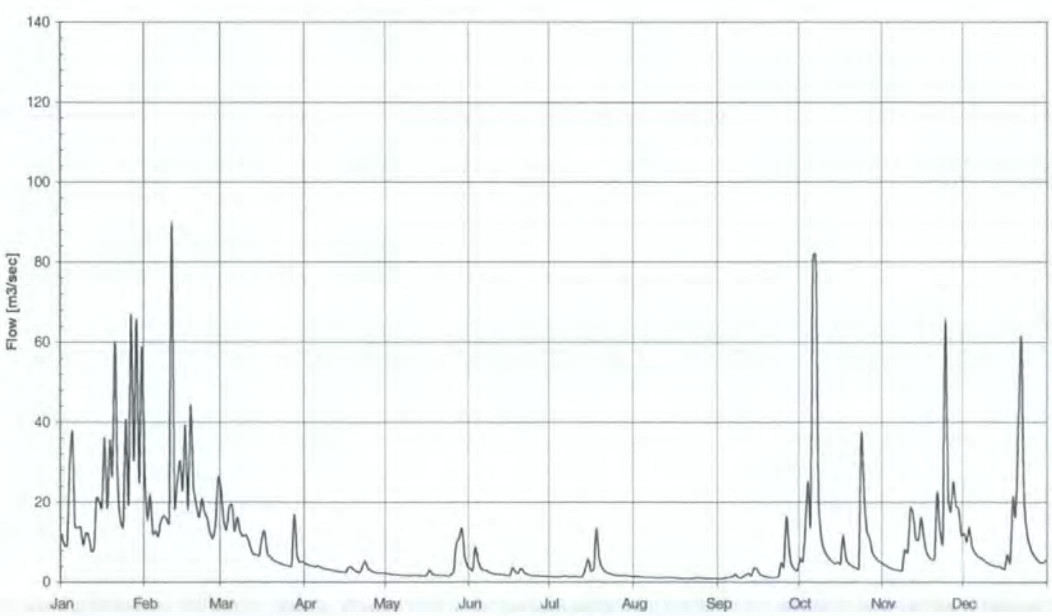

Figure 2-8 Tawe Flow 1995 (EA Wales)

2.2.5.2 Modelling & Seeking for alternatives

1996 marked a change in the remediation strategy, after a budget had been allocated to SCC for examining further remediation work. Dr. Ken O'Hara from SGS Environment Ltd., Liverpool, presented his approach of a combined aeration/mixing system to break stratification in the impoundment during a Joint Technical Group Meeting in March. The joint water quality monitoring was still ongoing and a sluicing program started in June, targeting neap tide periods until October. The transition of the NRA into the Environmental Agency resulted in a new definition of the liaison between the involved parties, including a proposal for new standards of DO conditions in the impoundment (Figure 2-9). The short-term objective was to apply the standard within the top 1.5 m of the water column and the long-term objective was to achieve conditions above the standard for the whole water column.

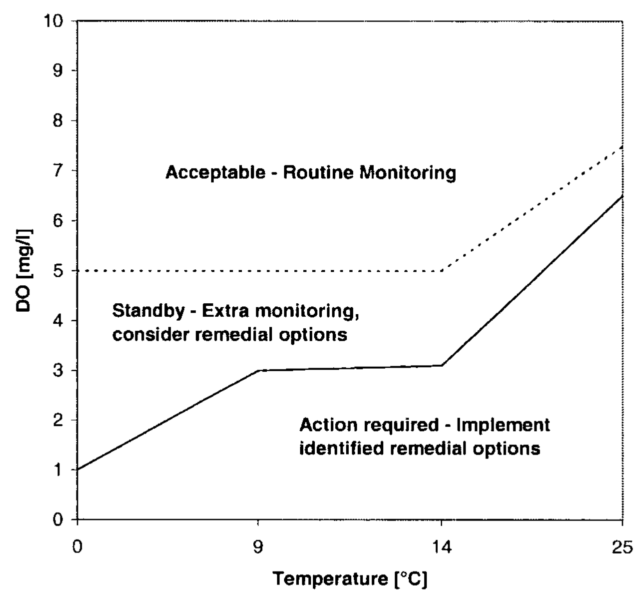

Figure 2-9 Water Quality Standards to protect salmonids in the Tawe impoundment (Edwards and Bishop, 2000)

This new outline of standards led to several occasions when further remedial action was requested by the EA throughout the summer period of 1996. Furthermore, toxic blue-green algae formation was found in the impoundment and the local press was publishing more critical articles about the effects of the barrage on the water quality of the Tawe (Turner, 1996). In response, official press releases were made by the authorities, pointing out that remedial action through partial drainage was ongoing. The pressure, however, was growing on SCC to find a more effective alternative for sustaining acceptable water quality in the impoundment. By the end of 1996, the poor water quality conditions were alleviated through extensive sluicing and the regular occurrence of flow spates (Figure 2-10).

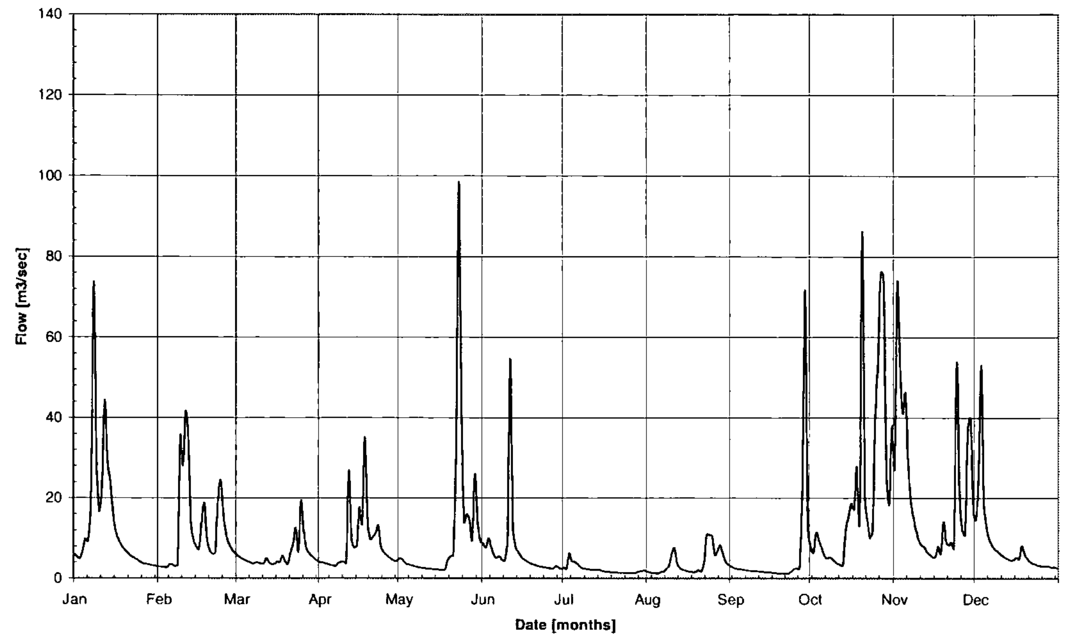

Figure 2-10 Tawe Flow 1996 (EA Wales)

Consequently, the focus in the beginning of 1997 was thrown on understanding and modelling the processes contributing to the deterioration of water quality in the Tawe. For this, the EA commissioned HR Wallingford to develop a predictive model for the hydrodynamic and water quality conditions in the Tawe, including an evaluation on the effectiveness of air injection as a remedial measure. HR Wallingford fed their software tool WQFLOW-3DSL (Three dimensional water movement and water quality model) with the necessary variables, e.g. river flow data, barrage and impoundment dimensions, lock operation hours, and validated it against water quality data collected during the period May-June 1994. Although the model reproduced the main features of stratification and the dissolved oxygen patterns of the impoundment, it lacked accuracy in predicting concentrations when compared with the observed data. The simulated flushing rates for different flow conditions indicated that the model predicted no significant scouring of the saline wedge below 45 m^3/s, while experience of the EA and SCC monitoring showed a complete flushing at fluvial flows of 40 m^3/s. The main reasons for these errors may have been the small data set used for the model calibration or the limited information on factors such as sediment oxygen demand and organic loads; these limitations were highlighted in the final report (HR Wallingford, 1997).

2.2.5.3 Remedial Approaches

The modelling results made clear what was already known to the stake holding parties, i.e., that hypoxic conditions will occur in the impoundment on almost all neap tides during low fluvial flows and at high summer temperatures. The review of the water quality monitoring in 1997 supported these findings by indicating several breaches of the short-term objective (Figure 2-9) after overtopping occurred, and a rare compliance with the long-term objective during the summer period (April to September). With the limited success and logistically complicated operation of sluicing, SCC had to investigate further options for remedial measures, so that tenders were invited in 1997 for the allocated remediation budget received in 1996. Of the 14 tenders submitted, four were short-listed for further interviews (Table 2-1).

Consultant	Approach
SGS Environment Ltd., Liverpool (Dr. Ken O'Hara, later Environmental Advice Centre Ltd., Colwyn Bay)	• Ceramic diffuser Stones fed through armoured airline by shore based compressors (2.0 KW oil free, vane, single phase). • 1 ITT Flygt Mixer (1.5 kW, 35 rpm, Banana Blade) situated behind Fish Pass
WS Atkins, Wales	• Air/Oxygen Injection by Vessel • Self raising flaps located on the weir top to reduce sea water ingress
Rendel Palmer & Tritton, Newport	• Oxygen Injection by Vessel or hypolimnion aerators • Movable steel gates or rubber dam on top of the weir to retain additional flushing volume • Installation of low level sluice gates
Hyder Consulting	• Propeller Mixer Stations consisting of duty and standby units on adjustable guide rails fixed to the side embankments • Fixed submersible pump jet aerator units • Floating propeller aspirators • Compressed Air Injection (Air Lift)

Table 2-1 Companies short-listed for remedial action

The well-specified and robust concept, which was already presented during the technical meetings in 1996, and the fact that it was the lowest bid, made the SGS approach the most appropriate for SCC. Therefore, Dr. Ken O'Hara and his team of subcontractors were commissioned to undertake an on-site trial assessment of the equipment in 1998, with the prospect of being granted the full contract. The trial data, as well as data from the subsequent installation phases, is presented and discussed in the following.

2.2.6 Aim of this study

The research in this chapter is part of a collaboration between SCC and University of Durham to analyse a range of factors that could lead to the degradation of water quality within the Tawe impoundment, and to assess the performance of the implemented remediation scheme. Several short-term studies have been conducted (Dyrynda, 1994; HR Wallingford, 1997), and the seasonal monitoring data was published as part of the yearly reports produced between the EA Wales and SCC (Stonehewer et al., 1995; Edwards and Bishop, 2000). An extensive analysis of all available data, however, has not been performed to date, due to the different data formats and limited time capacities available at the stake holding parties.

2.3 Methodology

2.3.1 Study sites

The following sites within the Tawe impoundment have been consistently sampled during the monitoring program which started in 1993:

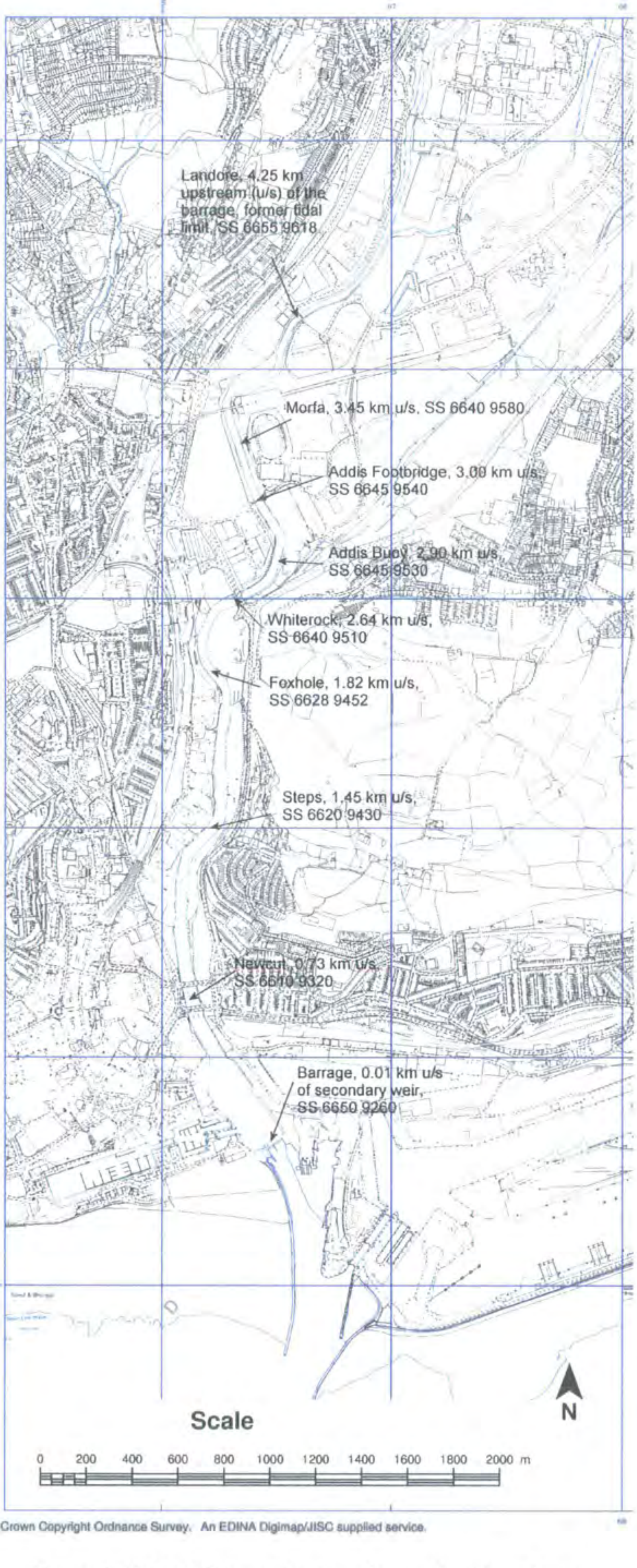

Figure 2-11 EA/SCC monitoring sites on the River Tawe

2.3.2 Data Sets/Sampling Protocol

The study is based on water quality data sets collected alternately by SCC and the NRA (later EA Wales) during the years 1994-2002. The data sets were collected in fulfilment of the agreed procedures between these parties to protect water quality in the Tawe impoundment. The majority of readings were only available in a print out format and had to be transferred into spreadsheets to carry out further analysis. An additional data set resulted from fieldwork conducted by the University of Durham in June 2000, to compare results with the SCC and EA data sets and to include the data in the overall SiMBa project (Wright, 2003). To remain consistent, the Durham University monitoring took place at the same sites as the monitoring of SCC and the EA Wales.

Year	Period	Monitoring Days	Equipment	Collected by	Initial Format
1994	04/01 - 21/11	52	YSI 3800	SCC/NRA	Printout
1995	23/02 –16/11	24	YSI 3800	SCC/NRA	Printout
1996	29/05 – 01/10	22	YSI 3800	SCC/EA	Printout
1997	23/04. - 01/10	14	YSI 3800	SCC/EA	Printout
1998	07/04 – 28/09	25	YSI 6820	SCC/EA	Spreadsheet
1999	19/05 – 15/09	15	YSI 6820	SCC/EA	Spreadsheet
2000	12/05 – 25/09	10	YSI 6820	SCC/EA	Spreadsheet
2000	10/06 – 21/06	12	HI Meters	Durham University	Spreadsheet
2001	14/05 – 24/09	12	YSI 6820	SCC/EA	Spreadsheet
2002	18/04 – 01/10	13	YSI 6820	SCC/EA	Spreadsheet

Table 2-2 Characteristics of the Tawe Data Sets

2.3.3 Measuring Equipment

Up until 1997 the SCC/EA data sets were obtained by using a 3800 YSI/Grant Ltd. Data logger system, which consists of a 3815 YSI sonde capable of reading multiple water quality parameters connected to a 3812 Grant Ltd. Data Logger for additional editing (Figure 2-12). During the early years of monitoring (Table 2-2), the data was printed out directly from the logger. In 1998, a 6920 YSI sonde replaced the old model and it was decided to download the readings for further analysis. Both sondes were calibrated regularly with YSI calibration solutions. The DO and pressure sensor were conditioned and calibrated before each monitoring run. Depth of measurement was determined through the pressure sensor included in the sonde setup.

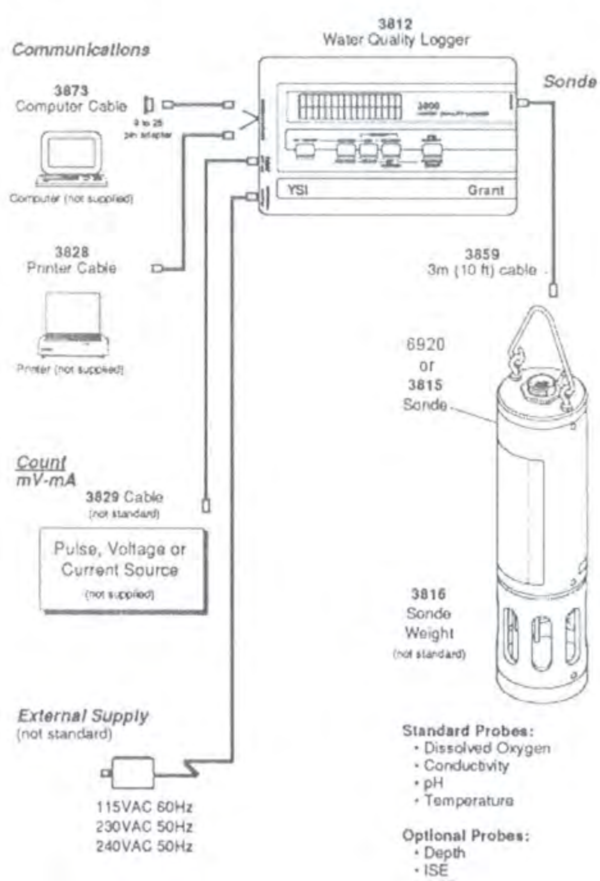

Figure 2-12 Measuring equipment applied

The Durham University data sets were recorded using three separate Hanna Instruments handheld meters (Appendix A.2). The measurement depth was determined through cable markers and all probes were supplied with an anchor weight to withstand flow forces.

2.3.4 Water quality monitoring

The monitoring was carried out on a regular basis, due to an agreed schedule between SCC and the EA targeting summer conditions (May-September). In recent years, more attention was given to periods of neap tides and low flow conditions, following the predictions of studies (Dyrynda, 1994; HR Wallingford, 1997) which identified that poor water quality conditions were likely to occur under these circumstances. Furthermore, monitoring could only take place up to a flow of 17 m^3/s, as measured at Ynystanglws Gauging Station (SS 685 998) (Figure 2-1), since above this level flows then became a safety issue for the crew. However, records for higher flow conditions are included in the data sets, which provided the opportunity to analyse extreme conditions in this study. The monitoring was carried out in at least 0.5 m depth intervals at the agreed sites given in 2.3.1. Parameters recorded included (Appendix A.2):

- Temperature
- pH
- Conductivity
- Salinity
- Dissolved Oxygen
- Depth

The measurements were confined to the centre axis of the deep main channel; the following analysis provides no information on the transverse variability of the parameters recorded.

2.3.5 Environmental Variables

2.3.5.1 Flow

The flow data records for the Tawe were recorded by the EA Wales in 15 min. intervals at Ynystanglws Gauging Station using a pressure transducer level measurement converted into a discharge of cubic meters per second (m^3/s) according to the calculated EA rating curves for this station. The data was received from the EA Wales and has been filtered to reduce it to hourly measurements for statistical analysis.

The flow reading recorded for the nearest full hour to the start of each monitoring run was defined as Q_s (Q_{sample}) in m^3/s. In order to explore long-term responses to the variation of flow, the parameter Q_{ave} was introduced, which is the mean flow for the period preceding the monitoring run since the overtopping regime changed. Other related flow parameters calculated on the basis of tidal conditions and flow, include Q_{max}, the maximum flow recorded after overtopping changed, and Q_{tmax}, the maximum flow following the highest high water level (HHWL) preceding the measurement.

2.3.5.2 Tidal height and overtopping regime

The observed tidal height data was acquired from the British Oceanographic Data Centre (BODC), which holds hourly sea level data from a permanent tidal gauging station at Mumbles (SS 6318 8753) that started recording in 1997. These records were applied during analysis of continuous water quality records to provide maximum accuracy. For the numerical analysis, the tidal prediction tables, produced by the Admiralty for the Standard Port Swansea (UK Hydrographic Office, 1994-2002), were used to cover the complete study period. Comparisons between the observed and predicted levels showed only minor residuals.

The overtopping regime was defined as number of tides overtopping (Tides OT), where a negative value describes the number of tides that do not overtop the barrage construction. The tidal regime and the related parameters (Q_{ave}, Q_{max}, Q_{tmax}) were calculated on the initial assumption that seawater intrusion starts at Tide levels $\geq$ 8.10 m ACD (3.10 m AOD). This assumption was made on the basis of levels given for the weir crest construction (Figure 2-13) stated in the construction drawings by W.S. Atkins Ltd. (W.S. Atkins, 1988).

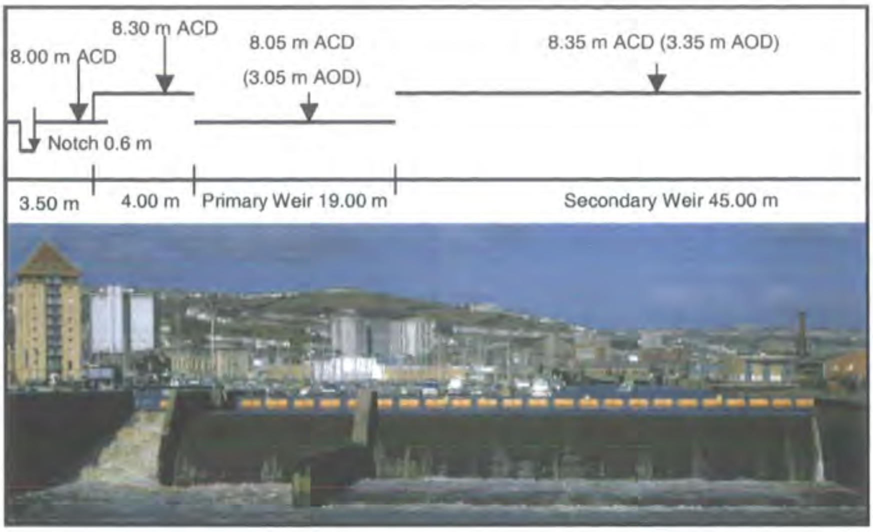

Figure 2-13 Tawe Barrage crest levels (schematic & barrage picture looking upstream)

Considering a drowned flow reduction factor (Figure 2-14) which has to be applied for the calculation of freshwater discharge and saltwater intrusion over a sharp crest weir, the actual overtopping height can only be an approximation, since it is related to the submergence ratio. Although the head of saltwater can be taken as the observed tide level, no complete recordings existed for the head of freshwater within the impoundment under different flow conditions.

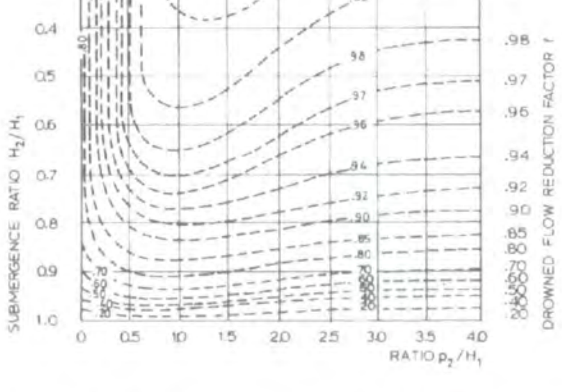

Figure 2-14 Drowned flow reduction factor as a function of submergence ratio H_2/H_1 (Bos, 1990)

Continuous water quality monitoring conducted 1999 in the marina area adjacent to the barrage, indicated that seawater ingress is detected at tidal heights >8.30 m ACD (3.30 m

AOD) (Figure 2-15 and Figure 2-16). The variations in the overtopping height are related to the freshwater flow Q_s, as suggested in Figure 2-14, and could also be influenced by wind direction and external factors such as lock operation and penstock sluicing times. Therefore, the numerical analysis will only consider an overtopping threshold based on construction levels and predicted tidal heights, while the discussion addresses the possible errors in theses factors.

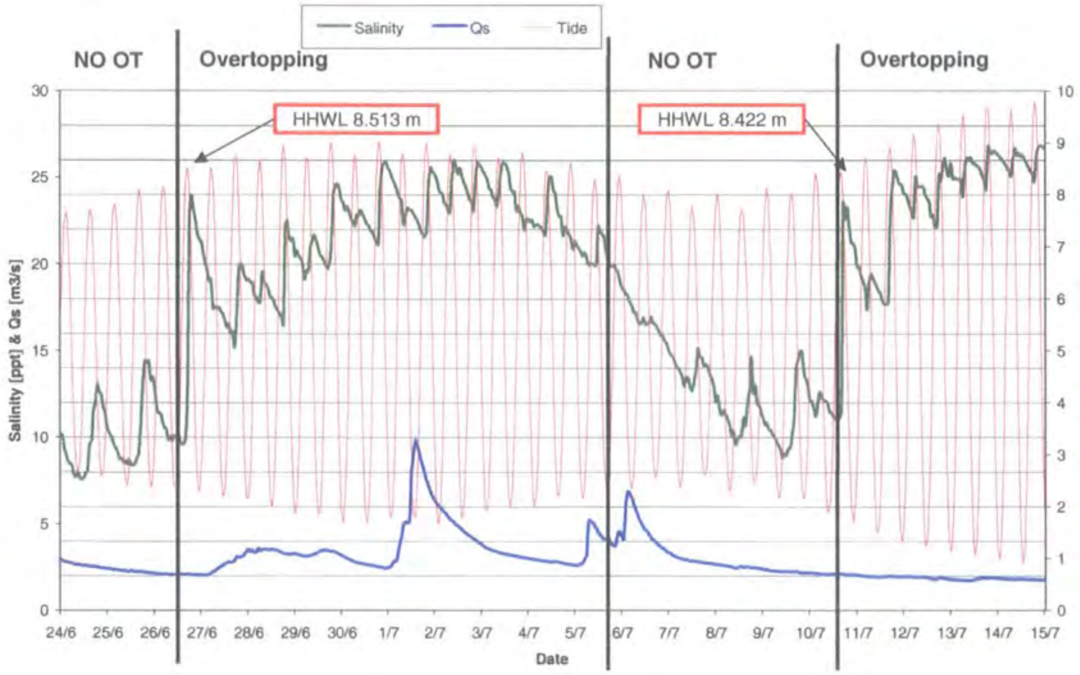

Figure 2-15 Continuous salinity record for Marina Deep during low flows

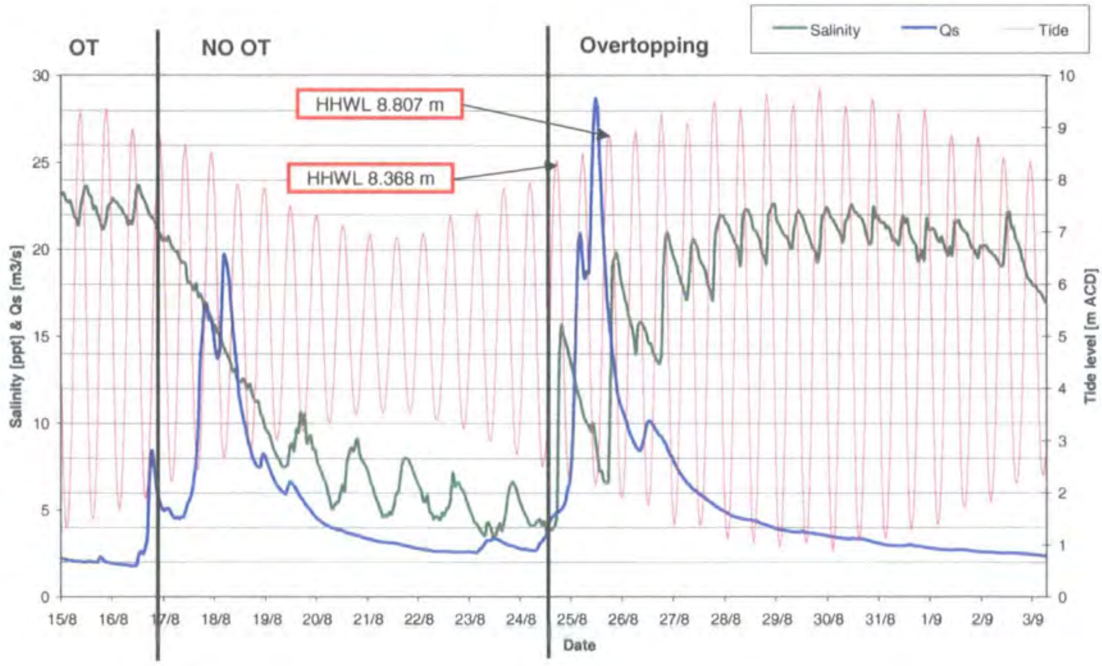

Figure 2-16 Continuous salinity record for Marina Deep during flow spates

2.3.6 Numerical Analysis – Logistic Regression

Considering the binary nature of the parameter in question, breaching or complying with the DO standard of 5 mg/l (Figure 2-9), logistic regression was the most appropriate technique to calculate a predictive model. This method predicts the outcome of a dichotomous dependent variable from continuous explanatory variables $X_1, X_2, X_3, \ldots X_p$ (e.g. Depth, Distance from Barrage, $Q_{freshwater}$, etc.) by applying regression coefficients (b_1, b_2, $b_3, \ldots b_p$) with b_0 as an intercept (Howell, 1997). The basic function for the transformation from a probability scale (0,1) to the scale of continuous variables (∞, $-\infty$) is:

$$Log\left(\frac{p}{p-1}\right) = b_0 + b_1 X_1 + b_2 X_2 + \ldots\ldots. b_p X_p$$

Equation 2-1

In this case, p is the probability of breaching the DO standard (i.e., DO < 5 mg/l) and (p-1) is the probability of compliance (i.e., DO $\geq$ 5 mg/l). The probability should not be interpreted as a probable frequency of breaching the standard, but rather as the probability of monitoring a DO event < 5 mg/l, given the specific combination of environmental factors (Tides OT, Freshwater Flow) and site properties (Distance, Depth, Site Code). This relates to the fact that logistic regression is based on an iterative process of maximum likelihood estimation, rather than least squares fitting. Furthermore, the method allows an assessment of the significance of parameters included in the model by providing the odds ratio for each of the chosen variables.

The statistical software package MINITAB© was used to fit a regression surface to the data, to estimate linear relationships between the explanatory and the response variables. Although major trends across the matrix were already observed during data formatting, the focus here is on verifying these predictors by constructing a best-fit regression model for each year of monitoring, which could be then used for the prediction of future breaches of the DO standard. The logistic regression models were fitted using forward and backward selection procedures for the explanatory variables in order to find the best-fit model based on a minimum number of significant variables. Although significance of parameters was the priority for inclusion, manageability was the second key to the selection of variables, since the model was to be used by the barrage operators.

The data included in the numerical analysis results from the regular water quality monitoring program of SCC and the EA during the years 1994-1998. In this period, no mixing equipment was installed within the impoundment, so that relationships between environmental variables (i.e., Tides OT, Freshwater flow), site properties (depth, distance, site code) and the response variable (DO<5 mg/l) could be explored without any distortion by remedial measures. Monitoring days on which external influences (such as local pollution events, extensive flushing periods and days of test trials with remediation equipment) are reported, were excluded from the data set to minimise the error in the model. The parameters covered by the survey as well as the monitoring locations are given under 2.3.1. In addition, environmental parameters such as freshwater flow (given as Q_s or Q_{ave}), the tidal regime (given as Tides OT) and the highest predicted tide level (given as HHW_{Pred}) were included in the regression matrix.

To best model the difference between conditions that gave rise to a breach of the DO standard (p) of 5 mg/l and conditions that do not (p-1), events of poor water quality were weighted so the number of events was approximately equal for the two conditions. The weighing factors for the events of different years are given under column (W) in Table 2-3 (e.g.,. for 1994 1113 events were weighted by a factor of 5 to give an apparent number of 5556 events for a comparison with 5281 non-events). This approach is used so that the difference between events and non-events is adequately modelled; without the weighting, the analysis could correctly classify 5281 (82.59 %) non-events out of 6394 observations, and yet incorrectly classify all of the events breaching the standard.

2.3.7 Numerical Analysis - Analysis of variance (ANOVA)

In order to explore the difference between monitored DO readings of the pre-aeration period (1994-1998) and the post-aeration period (1999-2002), a factorial approach based on a general linear model (GLM) ANOVA was used. Compared to the 'one at a time' approach, in which one factor is varied while all others held constant, the factorial approach is more applicable to environmental studies in which the important factors are varying together. The ANOVA approach enables the relative magnitude of each of the factors to be measured. It is also possible to account for covariates (factors that are measurable but not controllable) within the design through linear adjustment of the data. In this way, the importance of controllable factors can be considered after having removed the influence of the covariates. Finally, it is also possible to calculate the relative importance of interactions between factors that cannot be quantified in a 'one at a time' approach.

ANOVA tests against the hypothesis that the means of several populations (factor levels) are equal (Null hypothesis H_0), by comparing the variation among the means of these factors with the variation within the population. The estimates of variance are calculated as mean squares (MS) and result in a greater among factor variance than the within factor variance, if the null hypothesis can be rejected (Howell, 1997). The F-Test statistic returns the ratio between the mean squares of factors (MSF) and the mean squares of error (MSE), which is only based on within factor variation. For significant differences among factor means, the F-Test returns values > 1, while smaller values indicate that H_0 is true and no variation among factor means can be assumed. The significance of a factor is given by the probability value (P) for the calculated F-Test ratio and was set to be at least 95% ($P<0.05$).

The model design in this study included the following factors:

- Site Code: Numerical Index (1-9) for the standard monitoring sites on the Tawe (Figure 2-11). Out of the nine sites, seven were included in the ANOVA, since two sites (6-Addis Buoy and 8-Morfa) showed inconsistent monitoring records. The numerical order of the sites increases with distance from the barrage, so that the site code can also be regarded as an unscaled distance code.
- Depth Code: Numerical Index representing the relative depth for each measurement beginning with 1 for the surface reading (approximately 0.1-0.5 m from surface), 2 for middle depth and 3 for the deepest reading that could be achieved without disturbance from the sediment. Although monitoring was carried out in smaller depth intervals of 0.2-0.5 m, the data set had to be filtered to reduce the complexity of the model and to ensure balance of data between monitoring points.
- Tides OT: Describes the tidal regime in number of tides overtopping the barrage, as defined by construction levels and predicted tide level (section 2.3.5.2). Negative values stand for non-overtopping and positive values for overtopping tides.
- Aerators: Numerical index describing the three different stages of remediation equipment applied, with 0 representing no aerators, 1 indicating the first stage of 44 aerators and 2 the second stage of an additional 43 aerators installed.

Furthermore, the covariates salinity (ppt) and freshwater flow (Q_s in m^3/s) were accounted for in the analysis since, it was suggested in section 2.2.5.1 that DO readings vary to a great extent due to these parameters. The final model compared eleven monitoring days at different stages of overtopping (-8,-7,-4,-4,-3,-2,-1,7,9,13,15,17 Tides OT) for the non-aeration period and the aeration period.

2.3.8 Observational Analysis - Contour plots

The monitoring records for salinity and DO from a selection of days, representing the range of environmental conditions, were formatted into xyz coordinates to create longitudinal transect grid files for plotting contour diagrams. After consultation with computational modelling staff of the EA, and referring to the software manual (Golden Software, 1999), it was decided to apply kriging as the most appropriate gridding method, including an anisotropy ratio of 3.

For the trial period described in section 2.5, an un-scaled x axis was applied to reduce distortion of the grid from excessive extrapolation of data points, which result from the irregular spacing of the monitoring sites. Subsequently, the contour plots in this section should be read only in conjunction with the overview given in Figure 2-24. The limitations of using two dimensional contour plots with some data sets consisting of one east and one west bank reading per monitoring site were countered by calculating median z values from duplicates during gridding. This ensures a better representation of the available data set and averages peak readings, which could be created through sampling in the vicinity of the diffusers.

For all other contour plots presented in this study, a scaled x axis was applied, which represents the negative progression of distance from the barrage in km.

2.4 Pre-Aeration Period

2.4.1 Results – Numerical Analysis

Logistic regression was applied to each year separately and several best fit models resulted from the analysis, all of which correctly classified > 80 % of the events observed during the period 1994-1998. Selection procedures of explanatory variables applied during improvement of the model reduced the matrix to the given six parameters (Table 2-3), with all variables and the given constant at a significance level ≥95%.

Data Sets			Regression Coefficients b_p (Odds Ratio) for Predictors (X_p)						
Year	Period	W	Constant	Depth	Distance	Tides OT	HHW Pred	Q_{ave}	Concordance
1994	04.01-06.12	1:5	-1.9741	1.1647 (3.2)	1.0515 (2.86)	0.0654 (1.07)	-0.2871 (0.75)	-0.2134 (0.81)	89.2 %
1995	23.01-21.11	1:3	2.146	1.1588 (3.19)	1.0487 (2.85)	-0.0238 (0.98)	-0.8653 (0.42)	-0.1329 (0.88)	89.7 %
1996	08.02-29.11	1:6	-10.7725	0.848 (2.33)	0.62949 (1.88)	-0.0568 (0.94)	0.9224 (2.52)	-0.1150 (0.89)	80.3 %
1997	08.04.-01.10	1:4	0.1611	1.2408 (3.46)	0.8533 (2.35)	0.0299 (1.03)	-0.4865 (0.61)	-0.2075 (0.81)	83.3 %
1998	07.04-28.09	1:3	11.477	1.1640 (5.16)	0.4616 (1.59)	0.1947 (1.21)	-1.9114 (1.21)	-0.081 (0.92)	86.2 %

Table 2-3 Summary of binary regression analysis 1994-1998

The 1994 data set was by far the largest and therefore will be used as an example to describe the results given in Table 2-3. The best-fit weighted logistic regression for 1994 was:

$$Log\left(\frac{p}{p-1}\right) = -1.9741 + 1.1647(Depth) + 1.0515(Dist) + 0.0654(TidesOT) - 0.2871(HHW_{Pred}) - 2.134(Q_s)$$

Odds ratios: (3.2) (2.86) (1.07) (0.75) (0.81)

This model classified 89.2 % of the 6394 readings correctly, with a significance level of at least 99% for the constant and the variables, suggesting no obvious colinearity between the included variables. The odds ratio values below the estimates indicate a higher importance of the site parameters Depth (3.2) and Distance (2.86), in comparison with the external controls (Tidal regime, Freshwater Flow), which is an indication of the dominantly stratified conditions within the estuary and the strong relationship with the up-estuary dynamics of the salt-wedge. With regard to the forecast of low water quality events, it is more important to know when they occur than where, so that remediation measures can be initiated in advance. Therefore, the external controls, of which only the freshwater inflow (Q_{ave}) requires continuous monitoring, are of specific importance to the barrage management. With an odds ratio of 0.81, Q_{ave} is of similar importance, as the highest predicted Tide level (HHW_{Pred}) preceding a measurement (0.75). The strongest relationship, however, is given by the number of overtopping tides (TidesOT), with an odds ratio of 1.07 suggesting that this parameter is an important control in contributing to low water quality events. With the reduced model variables statistically justified, it was now useful to return to the observed data sets and plot the established relationships among the control parameters.

2.4.2 Results - Observational data analysis for the pre-aeration period (1994 – 1998)

According to the odds ratios for the external parameters (Table 2-3), freshwater inflow, here given as an average flow over the tidal regime period (Q_{ave}), shows a significant correlation with the increase of monitored DO levels below 5 mg/l. (Figure 2-17).

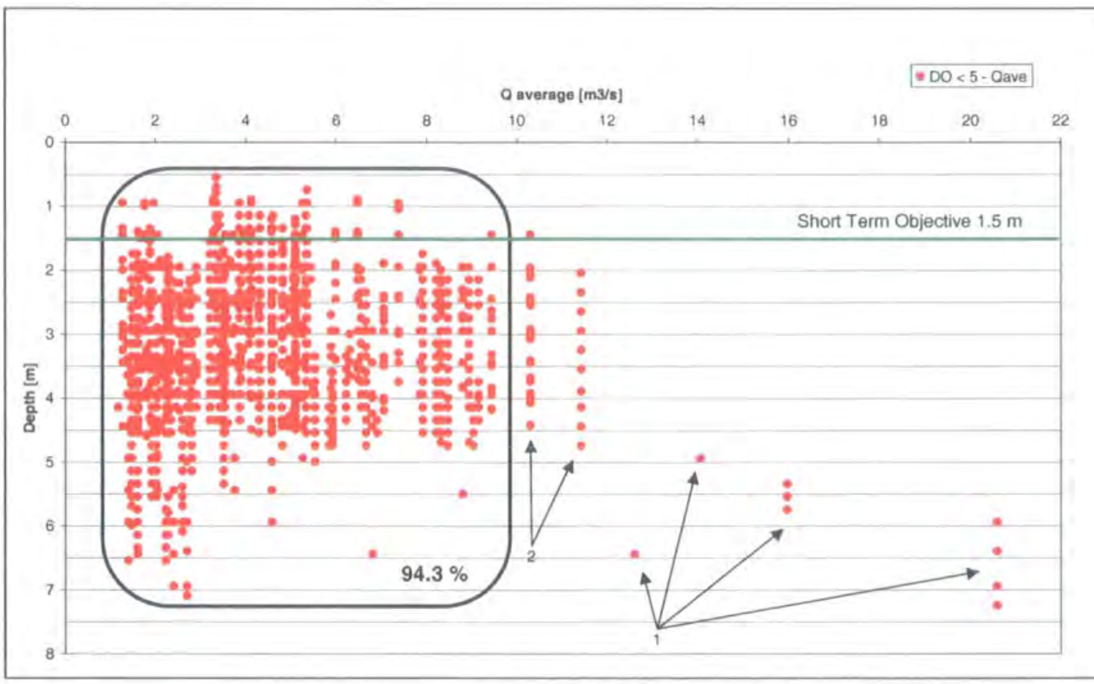

Figure 2-17 DO events (<5 mg/l) vs average flow (Q_{ave}) for the pre-aeration period 1994-1998

It is indicated (Figure 2-17), that the majority of low DO events (94.3 %) were observed at Q_{ave}<10 m^3/s. Monitoring included a range of Q_{ave} from 0.968 – 47.956 m^3/s and no breach of the standard was recorded for Q_{ave}>22 m^3/s. Occasional DO events above 12 m^3/s resulted from deep pockets of trapped brackish water directly behind the barrage (Point 1-Figure 2-17). The two DO events (Point 2-Figure 2-17) at Q_{ave}<12>10 m^3/s were recorded at the deep upstream site Addis (Figure 2-11) during receding flows with an actual Q_s of 5 and 6 m^3/s. Considering the short-term water quality objective (Figure 2-9) of ensuring DO conditions above 5 mg/l within the first 1.5 m from the surface, applied during years 1994-1998, Figure 2-17 shows that, for flow conditions above 11 m^3/s, no breach was recorded.

Some of the years modelled during logistic regression analysis delivered a slightly better fit (1998 - 87.9 %, 1995 – 89.9 %) when the momentary flow measure Q_s was applied instead of Q_{ave}. Thus, this plot should also be included here (Figure 2-18).

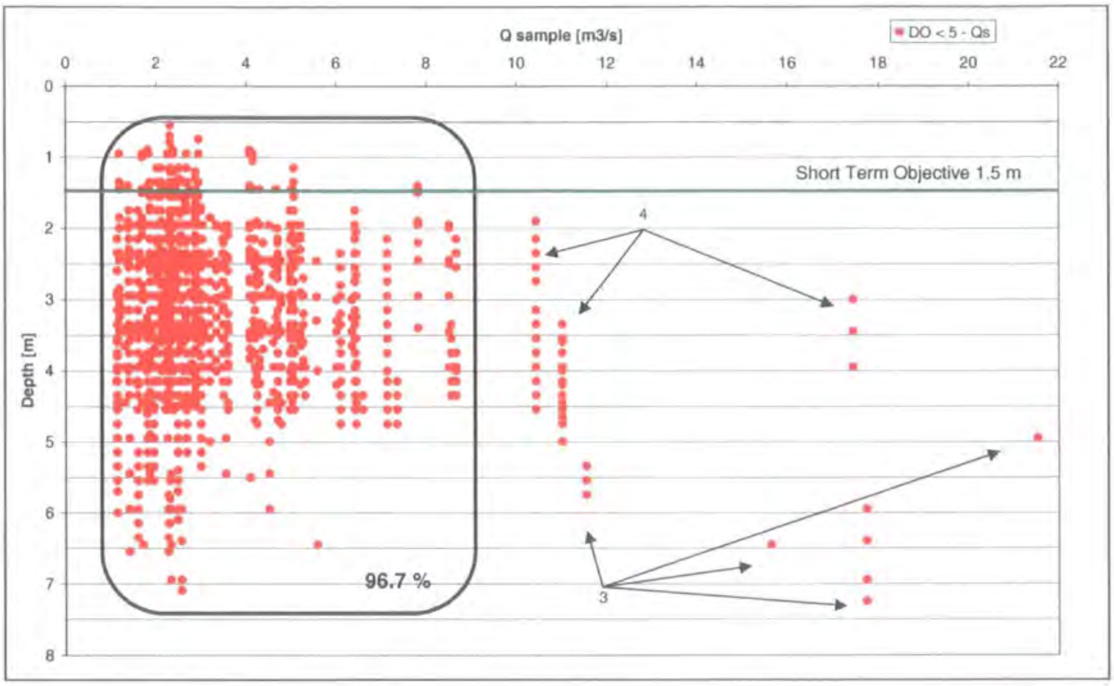

Figure 2-18 DO events (<5mg/l) vs momentary flow (Q_s) for pre-aeration period 1994-1998

Figure 2-18 demonstrates that 96.7 % of the monitored DO events can be narrowed down to a critical flow of Q_s<9 m^3/s. Exceptional DO events were again generated by salt pockets in the vicinity of the barrage (Point 3-Figure 2-18), and also related to externally triggered incidents such as stormwater overflows (Point 4-Figure 2-18) that followed precipitation peaks.

Focussing further on the established discharge limit of 9 m^3/s in Figure 2-19, reveals that the majority (90.5%) of DO events can be restricted to freshwater inflows of Q_s<5.5 m^3/s.

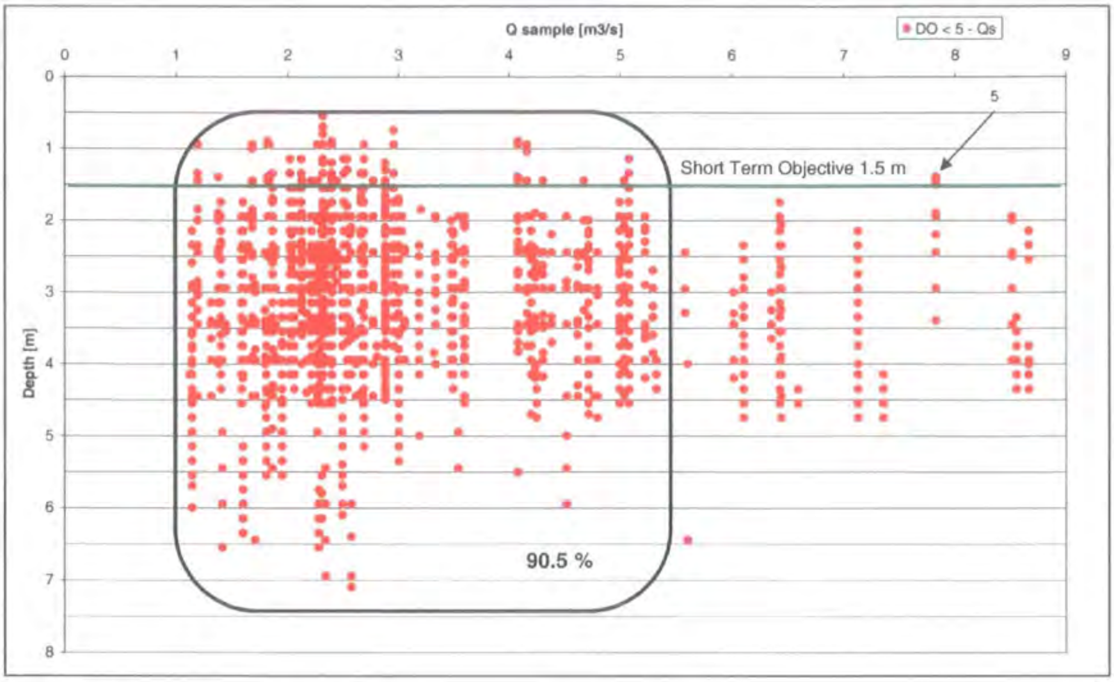

Figure 2-19 DO events (<5 mg/l) vs momentary flow (Q_s) < 9 m^3/s for pre-aeration period 1994-1998

Although DO events within the 9 m^3/s flow limit are now approaching the 1.5 m margin of the short term objective, the boundary was only breached on one occasion above 5.5 m^3/s (Point 5-Figure 2-19). These readings were recorded at the shallow site (Whiterock-Figure 2-11) and appeared to be resulting from upstream pockets of old saltwater being pushed over the elevated Whiterock site during wash out at higher freshwater flows (Q_s=7.828 m^3/s).

Figure 2-20 displays the distance distribution of DO events in relation to the flow conditions; a slight offset (0.05 km) was used for the Q_{ave} values, to improve distinction from the Q_s values. It is demonstrated that the previously established cut-off flow of approximately 10 m^3/s is still applicable for Q_{ave} and Q_s.

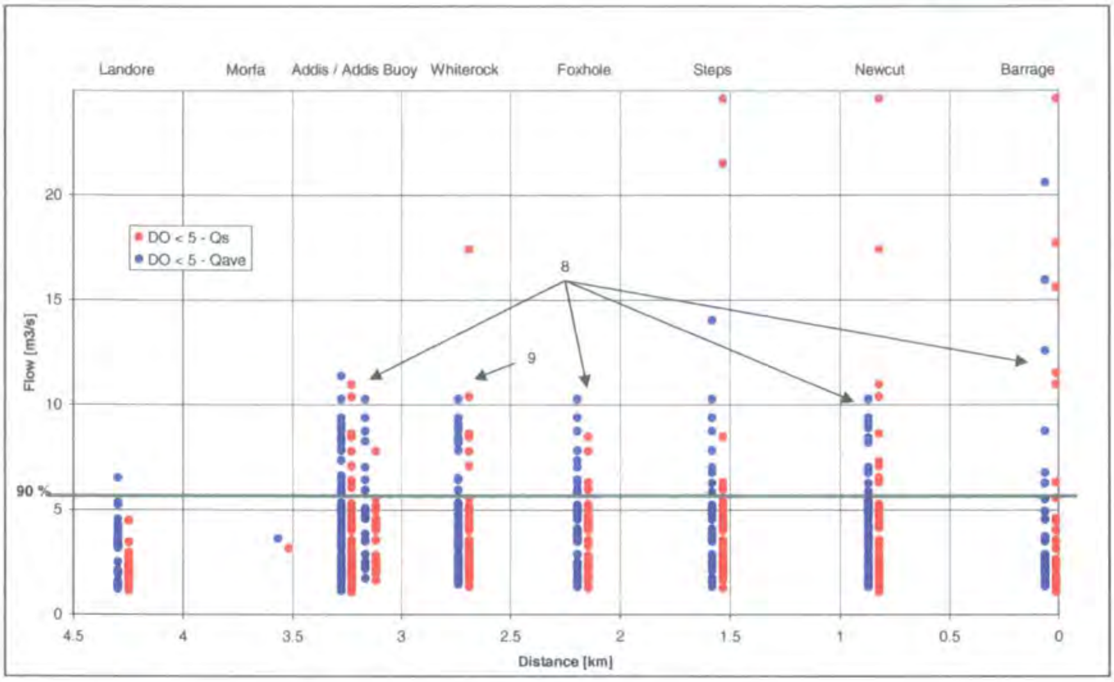

Figure 2-20 Flow vs distance DO events (<5 mg/l) for pre-aeration period 1994-1998

Again, the averaged flow values Q_{ave} show a more even longitudinal distribution below 10 m^3/s, accounting for the majority of recorded events within 3.3 km distance of the barrage, while events at the upper end (>3.3 km) were mainly limited to flows of Q_{ave}<6 m^3/s. As illustrated before (Figure 2-18), the quicker response variable Q_s allows a further distinction by restricting 90 % of all events extending up to the Landore to the 5.5 m^3/s flow margin. The remaining exceptions resulted from localised deep pockets of stagnant water (Point 8-Figure 2-20) and downstream advection of brackish pockets over elevated sites due to enhanced freshwater flow (Point 9-Figure 2-20). Since the distribution of events did not point out a correlation between the extension of saltwater intrusion for momentary flows below 5.5 m^3/s, the predicted HHW level at these flows was plotted against distance where the monitored profile indicated fresh marine water (Salinity >20 ppt, DO >80 %) in Figure 2-21.

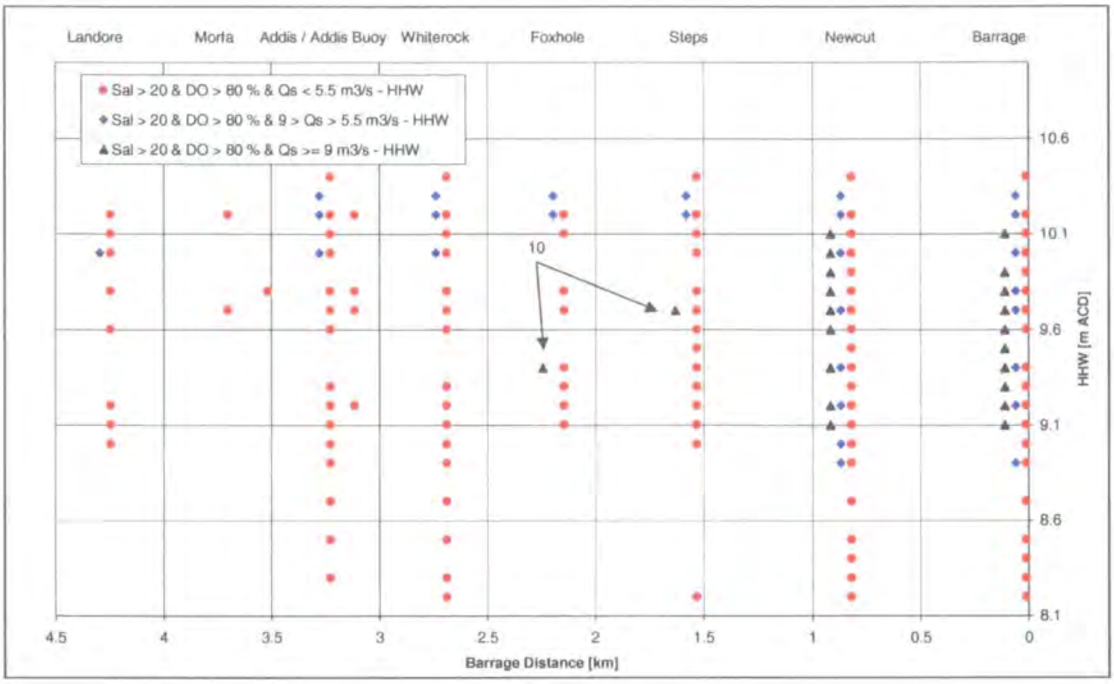

Figure 2-21 Marine water (Salinity >20 ppt, DO >80%) propagation vs tidal height

For freshwater flows below 5.5 m^3/s, seawater ingress was recorded throughout the profile for tide levels higher than 8.9 m ACD. The longitudinal intrusion maximum was strongly reduced for flows above 5.5 m^3/s, leaving all sites above 1 km distance unaffected by the propagation of saltwater up to a HHW of 10 m ACD. Once the tidal level of 10 m ACD was reached, marine water intrusion was recorded up to Landore. For flows above 9 m^3/s, marine waters were restricted to the boundary of 1 km distance from the barrage at all HHW levels. Exceptions (Point 10-Figure 2-21) resulted from readings taken near the halocline, where re-oxygenation took place during vertical shear of stagnant salt pockets, which then appeared to be fresh seawater.

In addition to freshwater flow and tidal height, the tidal excursion period, here defined as Tides OT, was identified as a significant control parameter for increasing the probability of low DO events during numerical analysis (Table 2-3). All recorded events for the years 1994-1998 in relation to amount of tides that had overtopped the barrage at the time of measurement are shown in Figure 2-22.

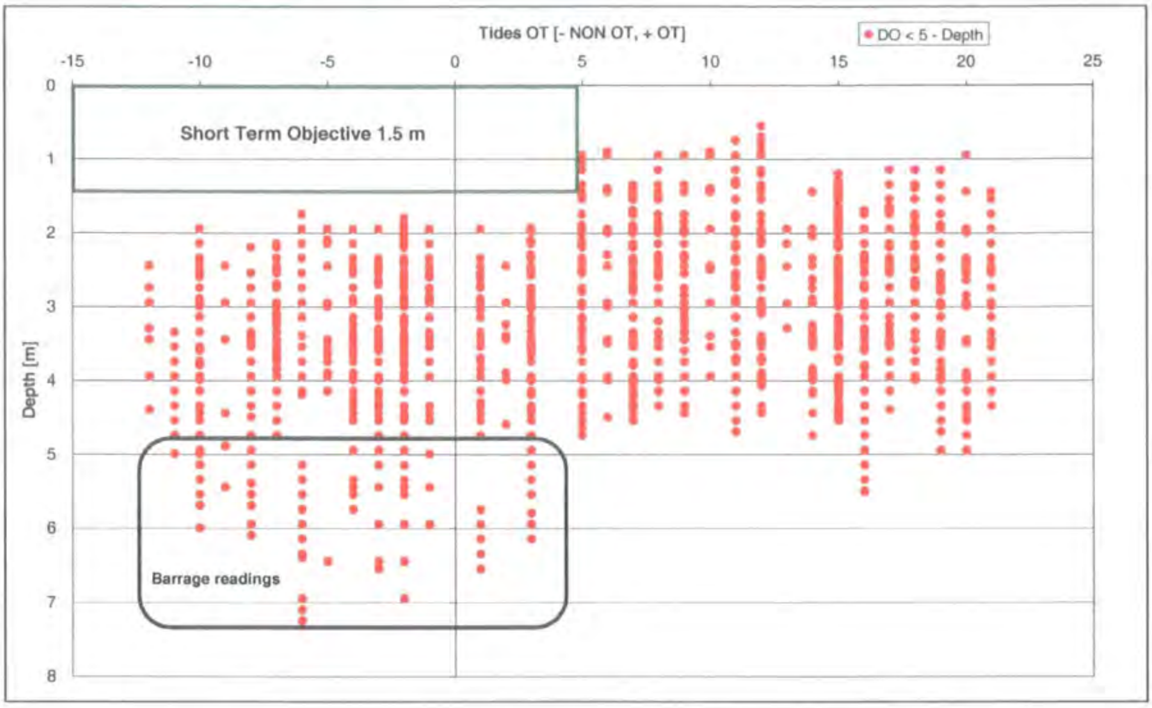

Figure 2-22 Tidal excursion length (Tides OT) vs depth of DO events (<5 mg/l) for pre-aeration period 1994-1998

It is illustrated (Figure 2-22), that DO depletion did not occur within the top 1.8 m from the surface during periods of non-overtopping. This condition was persistent up until five tides overtopped, assuming an overtopping height of 8.10 m ACD for all flow conditions. For excursion lengths of more than five Tides OT, the 1.8 m depth margin was broken, leading to breaches of the short-term objective. DO events at the barrage were mainly detected during non-overtopping periods and periods of up to five overtopping tides, which represents the hydrodynamic behaviour of the salt wedge outlined in section 1.2. In terms of frequency of detection, it should be mentioned that 76% of all recorded events (2032) occurred during tidal overtopping and 24% were recorded during non-overtopping periods.

2.4.3 Discussion

Several authors, including Uncles and Stephens (1996), Schroeder et al. (1990), Kurup et al. (1998) and Borsuk et al. (2001), have pointed out the influence of flow variations on salinity and oxygen dynamics in stratified estuaries. Statistical and observational analysis of data observed during the years 1994-1998 in the Tawe impoundment showed a strong correlation between the distribution of DO events <5 mg/l and the variation of fluvial flow. The momentary flow measurement Q_s proved to be a useful management tool in establishing periods of critical conditions, since 90.5% of the recorded DO events occurred during flows of Q_s<5.5 m^3/s and 96.7% were restricted to flows of Q_s<9 m^3/s. These findings replace assumptions from earlier reports, requiring minimum Q_s in the range of 17 m^3/s (Edwards and Bishop, 2000) to 40 m^3/s (HR Wallingford, 1997) in order to maintain acceptable water quality. Allowing a safety buffer, a critical flow of Q_s ≥12 m^3/s could be a new appropriate threshold for further remediation requirements, based on the observations reported here.

The longitudinal propagation of marine water in relation to the established flow thresholds shows that upstream advection of the freshwater saltwater interface (FSI) is restricted to 1 km of the barrage for Q_s>9 m^3/s during all monitored tide levels. At lower flows of Q_s<5.5 m^3/s the FSI reached the upstream site Landore (4.25 km) during HHW levels ≥9.0 m ACD. The observed tide levels from the tidal gauge at Mumbles indicate that the 9.0 m mark was reached for approximately 440 hours alone in 2002, which could result in a substantial amount of seawater upstream, when coinciding with flows of Q_s<5.5 m^3/s.

Unlike open estuaries in tidally energetic areas (Simpson et al., 2001), the Tawe impoundment is flow- rather than tide-dominated, due to the blocking structure at the inlet. This assumption was supported by the distribution of low DO events recorded in relationship to the excursion period of tidal overtopping (Figure 2-22). A marine dominated estuary would have experienced a stronger correlation between Tides OT and occurrence of low DO events; i.e., more tides reduce the frequency of DO events, but the data observations show that 76% of events recorded fell into periods of OT, as defined by the lowest crest height. If overtopping actually occurs at tide levels above 8.10 m ACD, depends on factors such as the upstream water level and wind direction (Section 2.3.5.2), and can only be assumed for the whole data set. The distribution of events (Figure 2-22), however, demonstrates that periods of non-overtopping feature saline stratification with a 1-2 m strong freshwater surface layer. This stratified system also includes pockets of arrested saltwater directly behind the barrage. Both of these features could be recognized in Figure 2-22 for non-overtopping periods while reaching into excursion lengths of up to 4 overtopping tides. This suggests that no, or only reduced, overtopping took place, which was not enough to introduce mixing within the impoundment. For more than five overtopping tides, deep DO events in the vicinity of the barrage disappeared and DO events reached into the surface layers, which indicates mixing of the water column. These findings comply with the records of the continuous water quality sondes shown in Figure 2-15 and Figure 2-16, which illustrated saltwater intrusion after approximately 2-4 Tides exceeding the lowest crest level of 8.05 m ACD.

2.4.4 Conclusions

Numerical and observational data analysis of water quality data recorded on the River Tawe during the pre-aeration period of 1994-1998 indicated four major findings:

1. The frequency of DO events <5 mg/l is correlated to the freshwater inflow, such that the majority (96.7 %) of events occurred during flows of Q_s<9 m^3/s. Therefore, a recommendation for critical flows triggering DO events (including a safety factor) could be a limit of $Q_s \leq 12$ m^3/s, which represents the mean daily flow gauged since 1957 by the EA.
2. The spatial distribution of DO events <5 mg/l suggested that these are restricted to a limit of 3.3 km distance from the barrage for flows exceeding 9 m^3/s, while events at distances >3.3 km were only recorded for flows of Q_s< 5.5 m^3/s.
3. Marine water intrusion was limited to 1 km distance from the barrage during flows of $Q_s \geq 9$ m^3/s for all predicted tide levels. Propagation of marine water extended up to 4.25 km distance from the barrage only at tide levels above 10.0 m ACD during flows of 9 >Q_s >5.5 m^3/s, while maximum marine intrusion was recorded during all tide levels for flows below 5.5 m^3/s.
4. The tidal regime indicated stratified conditions with a 1-2 m top freshwater layer unimpaired by DO events for all non-overtopping periods and up to 4 overtopping tides, based on the assumption that tidal overtopping starts >8.05 m ACD. For excursion lengths of more than 4 tides overtopping, DO events reached into the epilimnion, while deep events in the vicinity of the barrage disappeared, indicating the influx of seawater accompanied by mixing of the hypolimnetic water within the impoundment.

2.5 Trial Period

2.5.1 Introduction

With the joint water quality monitoring program of SCC and the EA Wales ongoing, the aeration trials started in August 1998, with 15 diffuser tubes (70 mm OD, 0.5 m length-Figure 2-23) deployed in a 280 m river section between Addis Footbridge and Whiterock Bridge (Figure 2-24). Three sets of five tubes were placed in the deep channel, unequally spaced over distances of 30 m, 50 m and 200 m, respectively. They were supplied by a 16 mm ID self-sinking airline with 3 shore based compressors (Reitsche, 3 phase noise reduced vane type, 1.8 kW) housed in cabinets. In addition, a 'Banana Blade' mixer (ITT Flygt, 1.4 m diameter, app. 36 rpm), housed in a stainless steel frame, was installed adjacent to the fish-pass at a depth of 2 m from the surface. Earlier tests at greater depths indicated that better efficiencies were achieved by lifting rather than pushing the stagnant saltwater over the pass.

Figure 2-23 Diffuser Tubes & Airlines

2.5.2 Monitoring

The diffusers were switched on at midday on the 25th August and left running until the 8th September. Vertical profiles were obtained once a day on 25th, 26th, 27th,28th, 30th August and 2nd September. Profiles of salinity, temperature and dissolved oxygen were taken in 0.5 m depth intervals at 6 agreed sites (Figure 2-24) at an appropriate distance from the actual diffuser cone. The oxygen profile of the 30th August had to be rejected during the analysis, due to poor quality of the readings. The mixer unit was started on the 27th August and left running until the 7th September. In addition to the sampling, two bed-mounted YSI water quality probes were deployed 50 m upstream of the last set of diffusers (Site 7-Figure 2-24) at a depth of 1 m and 3 m from the surface. These sondes continuously recorded salinity, dissolved oxygen and temperature values every 30 min., from the 20th August to the 6th September.

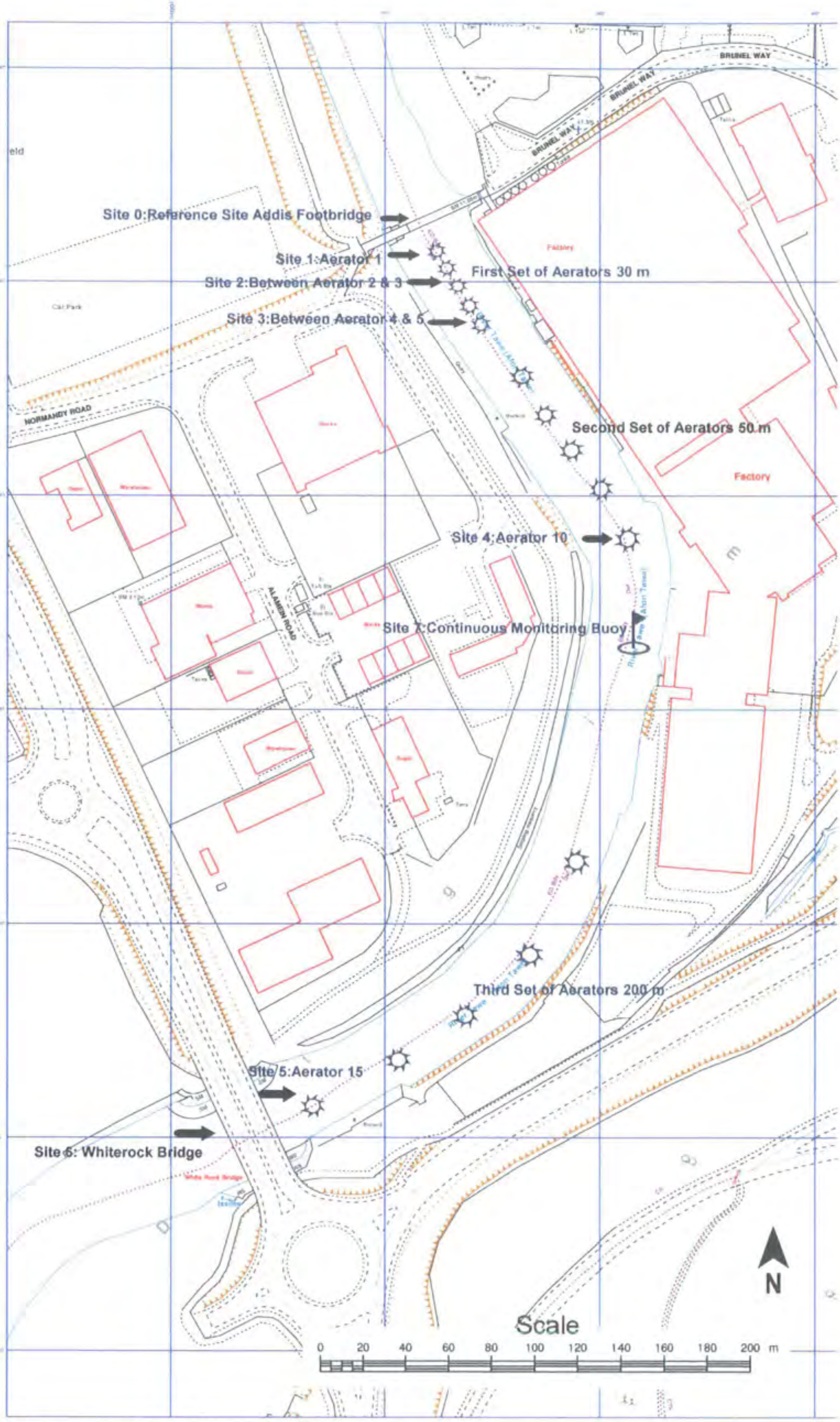

Figure 2-24 Aeration trial equipment overview map

2.5.3 Environmental Variables - Tidal regime and freshwater flow

Predicted tidal heights for Swansea led to the assumption, that the trial period would not be accompanied by overtopping tides. However, the observed tide levels from Mumbles gauging station revealed that intrusion of seawater was still apparent on the next 5 tides after the trial started (Figure 2-25). The extent, to which this affected the trial site 3 km upstream is discussed later.

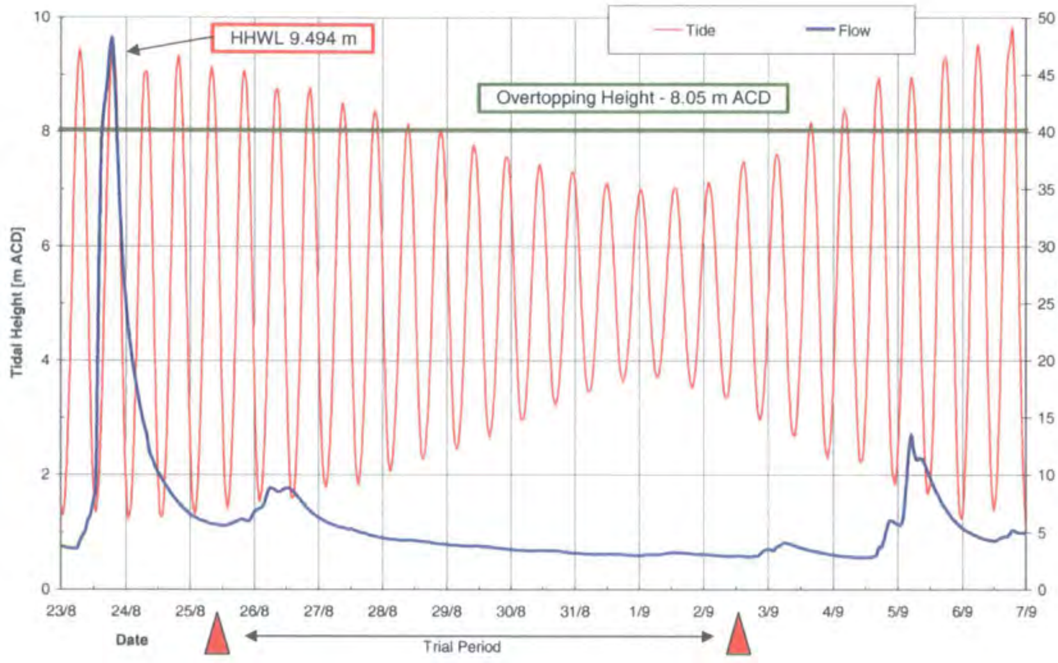

Figure 2-25 Tidal regime and freshwater flow during the aeration trial period 1998

The freshwater flow was marked by flow event of 48 m^3/s two days prior to the start of the trail (Figure 2-25). According to the hydrodynamic models (HR Wallingford, 1997), events of this size would flush out the saline wedge completely. In this case, however, it coincided with the highest spring tide level followed by another three overtopping tides before the trial started, while the flow quickly receded to normal levels. Subsequently, profiles from the first day of the trial show a well defined saline wedge up to Addis Footbridge (Site 0-Figure 2-24), which defined the upstream boundary for monitoring. During the trial, a minor event of 8.8 m^3/s occurred, which had to be considered for assessing the equipment's performance.

2.5.4 Results of the vertical profile monitoring

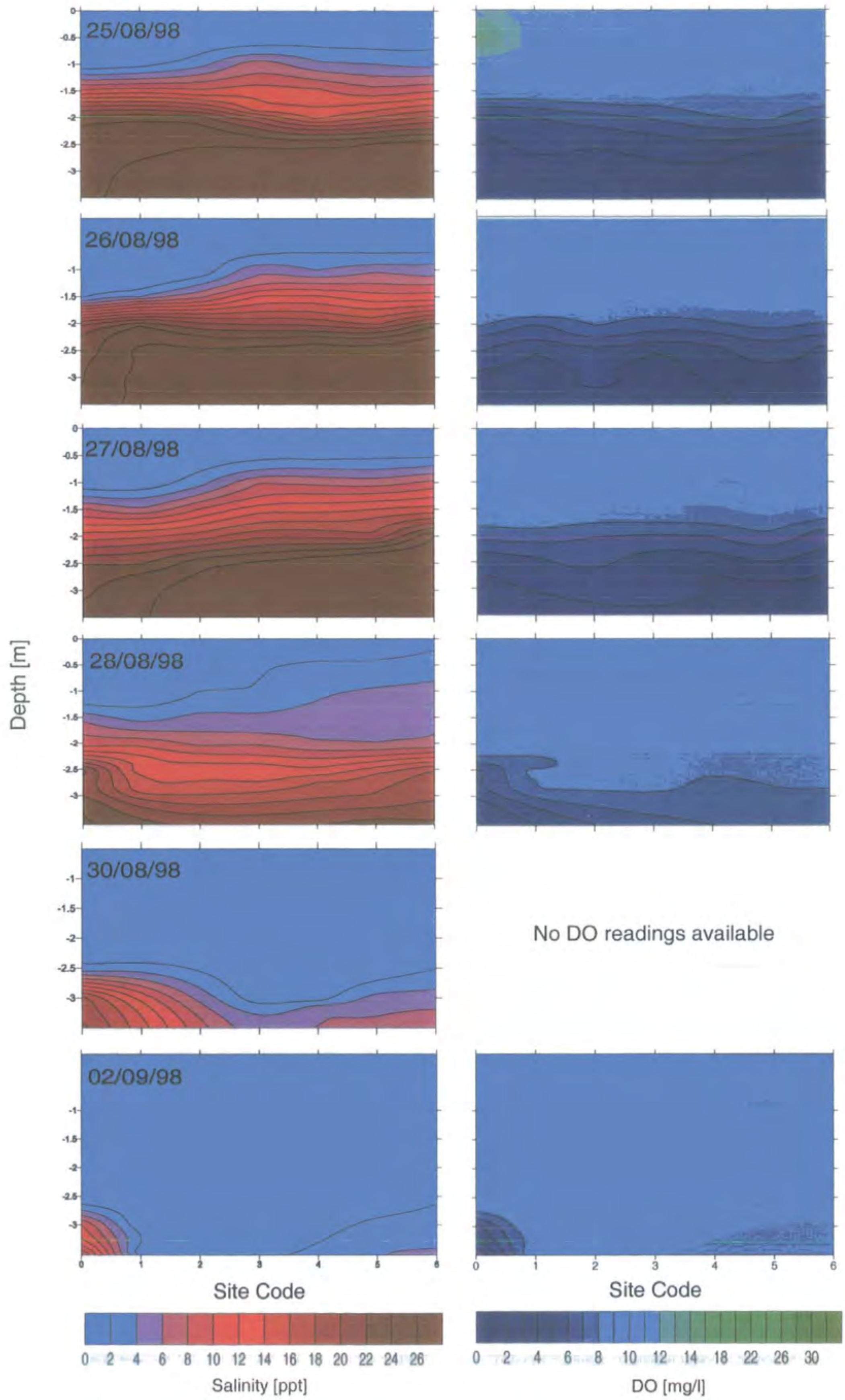

Figure 2-26 Longitudinal salinity and DO contours during aeration trial period 1998

The salinity contours (Figure 2-26) for the first trial day (25/08/98) show a highly stratified profile with a sharp salinity gradient at a depth of 1.5-2.0 m for all 6 monitoring sites, indicating the presence of a halocline. Salinity levels ranged from 0-6 ppt within the overlain freshwater layer to maximum levels of 28 ppt near the river bed. This first profile was taken approximately four hours after the aerators were switched on; no significant disturbance of the stratification is visible. On the second day (26/08/98), the conditions appeared nearly identical, with a similar salinity gradient at 1.5-2.0 m depth and no obvious dilution effect within the profile. The third day (27/08/98) contour shows a slight spatial variation in isohaline depths for the sites 0-5, with a less confined intermediate jump between the upper freshwater and the lower salinity layers. Although the boundary levels of 0 ppt at the surface and 28 ppt for the near bed layer still exist, an increase in isohaline thickness is observed at 1 to 2.5 m depth. This trend continues on the fourth day (28/08/98), with a distinct dilution throughout the vertical profile at the sites 1-6; the disappearance of the sharp salinity gradient (halocline) can be observed. Salinity levels of 12 ppt are now apparent at 2.5 m depth, as opposed to values of 24 ppt on the previous day. On the sixth day (30/08/98), most of the saltlayer is diluted within range of the aerators. High salinities only remained in the deeper parts of the upstream reference site 0, where saltwater was trapped through the last monitoring day (02/09/98), while the other sites indicated no remaining saltwater throughout the water column.

The longitudinal oxygen profiles (Figure 2-26) for the trial period illustrate a very similar pattern when compared with the salinity transects. While the first 2-3 days of the trial show a stratified oxygen distribution with saturated conditions within the top 1.5 m of the water column and oxygen deficiency below this margin, the fourth day (28/08/98) demonstrates increased dilution within the profile. Near bed layers were now above the 5.0 mg/l margin at all sites (1-6) within the range of the aerator impact, while the upstream reference site (0) still suffered from oxygen depletion at a depth below 2.5 m. As with the salinity contours, the trend of trapped water is continued up until the last day of monitoring at the reference site, which resulted in depleted oxygen conditions near the bed, while the remaining sites were saturated throughout the column.

2.5.5 Results from the continuous water quality monitoring

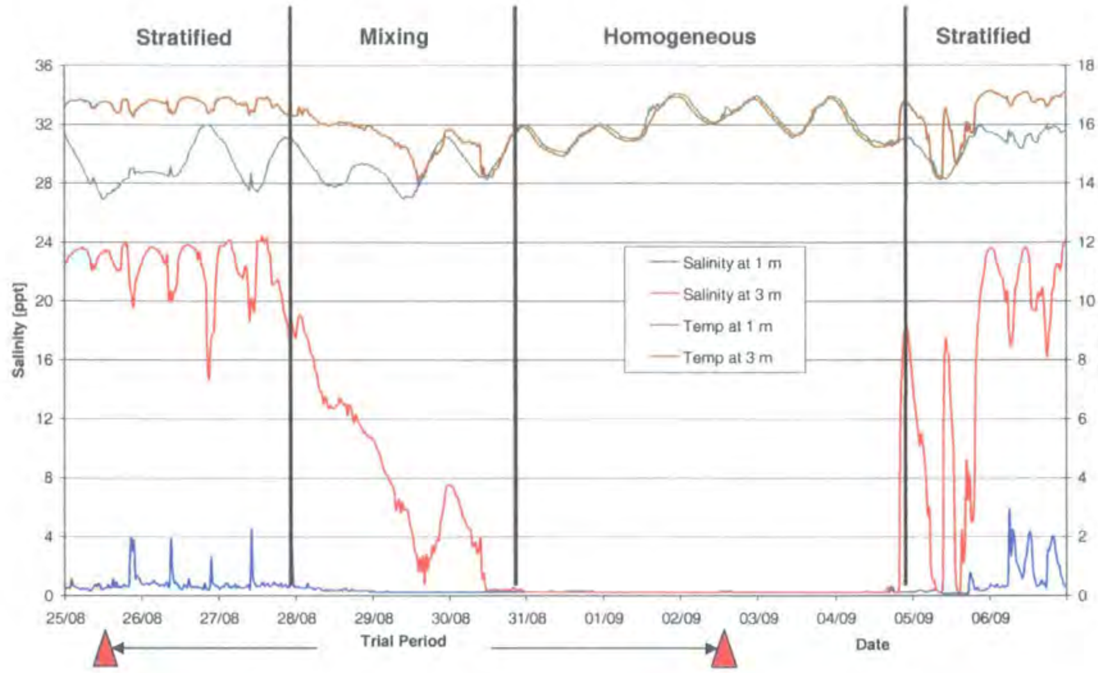

Figure 2-27 Continuous water quality data during aeration trial period 1998

The time series of the mounted water quality sondes at 1 and 3 m depth describe a stratified water column at site 7 (Figure 2-24) for the first three days of the trial (Figure 2-27). This is represented by the large difference in salinity values at 1 m (0.5-4 ppt) and 3 m (20-24 ppt), accompanied by a 2 °C temperature gap between those depths. Wave-like temporal variations triggering peaks for each parameter are apparent, but do not result in mixing of the layers. During the course of the fourth trial day (28/08/98), dilution of the deep layer occurs, resulting in a homogeneous water column by the end of the sixth day (30/08/98). However, with the aerators still running, stratification was re-established quickly during the first overtopping tides entering the impoundment on the 5th of September (Figure 2-25), which restored salinity levels of 20-24 ppt and a temperature gap of 1.5 °C at 3 m depth.

2.5.5.1 Discussion

When comparing the results of the sample monitoring with the continuous monitoring data, the stratified nature of the observed water body at the start of the trial period is well described by both data sets. While the contours of the sample data provide a picture of the geometry of isohalines, the continuous record supplies a matching series of salinity and temperature fluxes for the surface and deep water layer. The most interesting feature of the trial period is the actual break up of stratification at the monitoring sites 1-6 during the course of the 4th trial day (28/08/98), while the reference site 0 (Addis Footbridge), 200 m upstream of the first set of aerators, remains stratified. Since the downstream reference site 6 is comparably shallow (2.0 – 2.5 m) and in 50 m proximity to the last aerator set, other downstream sites had to be compared to the aerated river section. The regular monitoring programme provided additional data for the standard sites Landore, Foxhole, Steps and Barrage (Figure 2-11) from 27/08/98 onwards, which, in conjunction with the trial section data, delineates the impact of the diffusers.

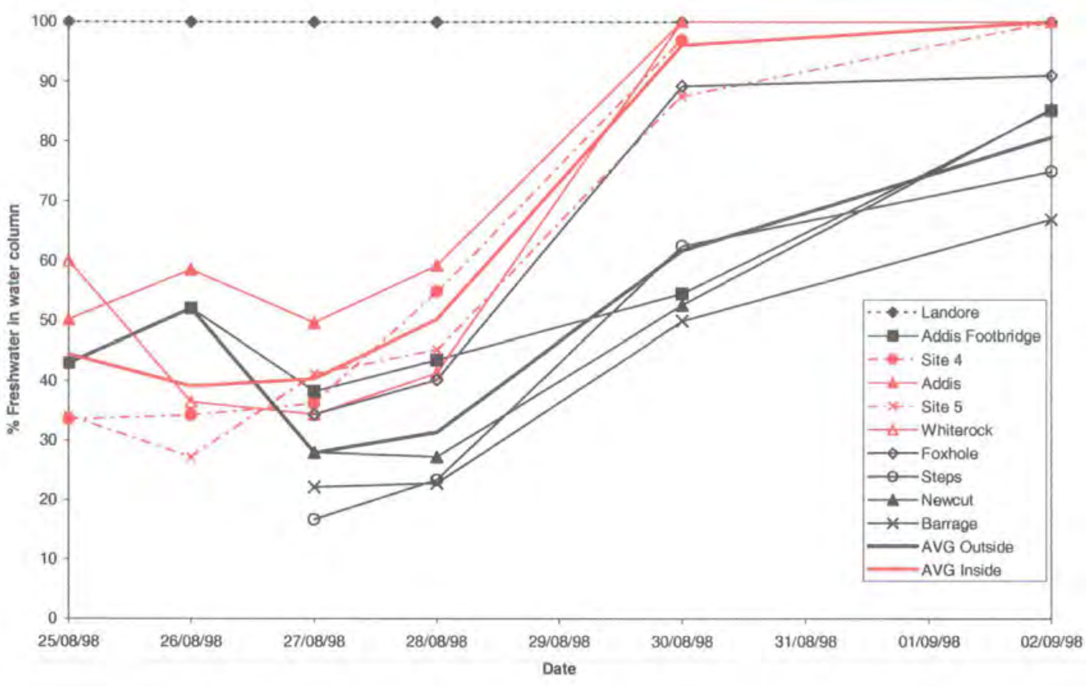

Figure 2-28 Freshwater percentage in the water column of aerator and reference sites during the aeration trial period 1998

The percentage of freshwater (defined as <5 ppt Salinity) in the measured water column of each site over the trial period is compared in Figure 2-28. Landore, as the furthest upstream site, was unaffected by tidal intrusion or aerator activity and remains at 100% freshwater level, while other profiles consisted of only 17% freshwater (Steps, 27/08/98) before mixing or erosion of saline layers was observed. Assuming that the upstream reference site Addis Footbridge and the downstream sites Foxhole, Steps and Barrage define those sites unaffected by the aerators, an average 53% reduction of saltwater was triggered by shear forces introduced through freshwater inflow, which left those sites at an average of 20%

trapped saltwater in their near bed layers. In comparison, an average 60% reduction of saltwater was achieved for those sites within the range of the aerators, leaving none with trapped saltwater in their deeper layer.

However, stratification was quickly re-established with the next overtopping tide (Figure 2-27) showing the limitations of the remediation equipment in introducing mixing during the initial phase of stratification build up.

The effect of different spacing of the aerators on the performance in each section is difficult to determine from the available data set, since the immediate downstream reference site Whiterock shows that mixing upstream also affects the respective downstream section. Therefore, an assessment of the spacing strategy during the trial seemed baseless and cannot be included in this study.

2.6 Aeration Period

2.6.1 Introduction

Following the successful completion of the aeration trials, CCS awarded EAC with the contract of supplying a full mixing/aeration system, which was installed by the EAC in two phases during the summer of 1999 and 2000. For the amount, schedule and location of equipment installed please refer to Figure 2-29. Phase 1 of the aeration scheme consisted of 44 diffusers, with 11 installed immediately upstream of the barrage (3 at the primary weir, 8 at the secondary weir), 5 in a hollow next to the marina, 13 at the Newcut site and 15 remaining installed after the trials between the sites Whiterock and Addis (Figure 2-29). Phase 2 involved an additional 43 of diffusers with an extra 7 at the Newcut site, 2 at the Steps site, 10 at the Foxhole site, 3 downstream of the Whiterock site, 1 at the Addis site and 20 installed near the furthest upstream site, Landore. To assess the effectiveness of the equipment in preventing low DO conditions, the water quality monitoring programme of the EA Wales and CCS was continued at their agreed sites during 1999-2002. The observed data sets were made available to this study, for an evaluation of the impact the two aeration stages had on the DO distribution within the impoundment under different conditions. Monitoring during the aeration period only covers flow conditions below 17 m^3/s, due to safety issues. However, stratification is less likely to occur at higher flows and can therefore be excluded from the assessment analysis.

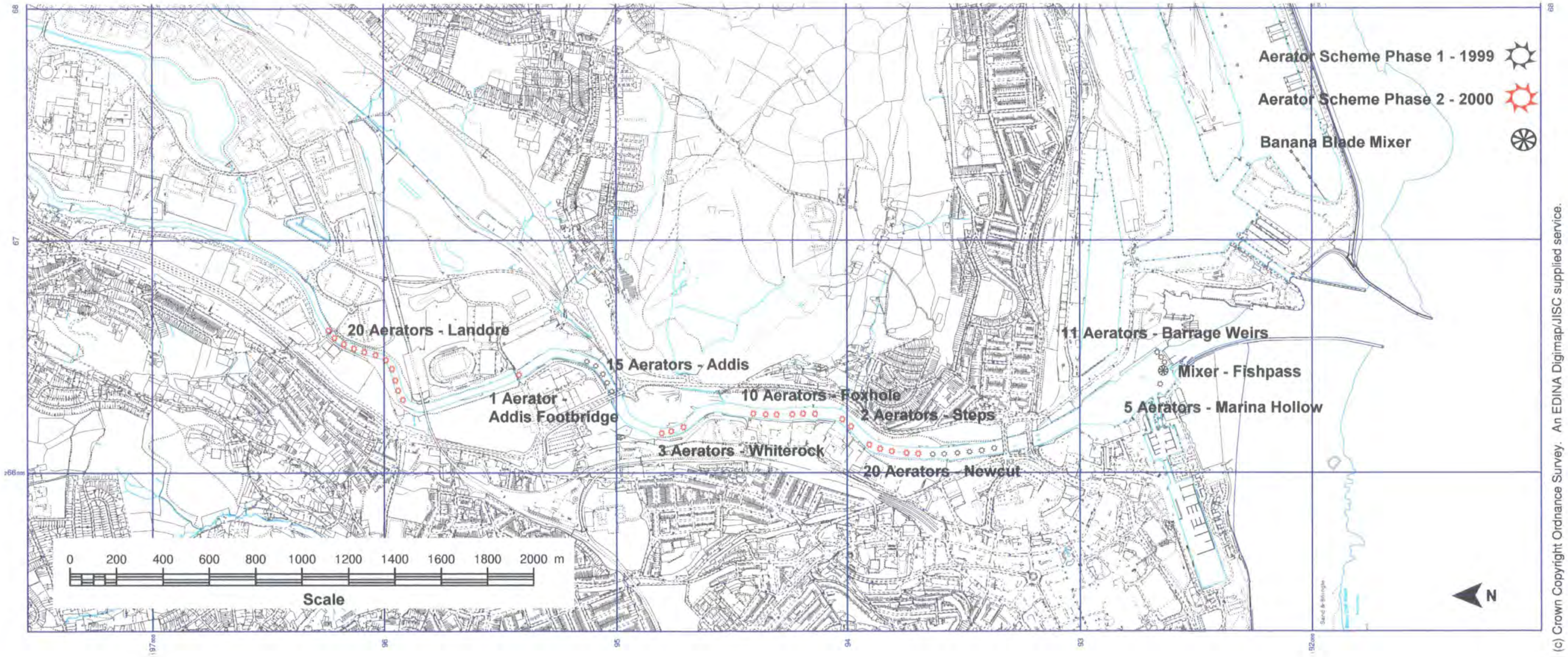

Figure 2-29 River Tawe aeration scheme 1999-2000 (overview)

2.6.2 Results – Numerical analysis - ANOVA

The ANOVA results of the GLM regarding the monitored DO conditions in the Tawe impoundment are presented in Table 2-4. It was possible to describe 49 % (ω^2 = 0.49) of the differences in DO means observed on 11 selected days by the included parameters: Site Code, Depth Code, Tides OT, Aerators, and the covariates Salinity and Flow at a significance level of $p<0.01$.

Source	Degrees of Freedom (df)	Mean Square (MS)	F	P	Variance of factor or interaction (σ^2)	Size of Effect (ω^2)
Salinity	Covariate	661.34	118.84	0.000	0.946	0.08
Flow Q_s	Covariate	172.83	31.06	0.000	0.241	0.02
Site Code	6	64.72	11.63	0.000	0.512	0.05
Depth Code	2	182.99	32.88	0.000	0.512	0.05
Tides OT	10	167.42	30.08	0.000	2.336	0.22
Aerators	2	186.19	33.46	0.000	0.512	0.05
Site Code*Depth Code	12	14.94	2.69	0.001	0.162	0.02
Error		5.57			5.565	0.51

Table 2-4 GLM ANOVA for DO mean levels monitored in the Tawe impoundmment

The results represent the best fit model achieved from using forward and backward procedures for the selection of factors and interaction between factors. The magnitude of the error term (ω^2 = 0.51), however, suggests that there are still factors not accounted for in the model, although all observed parameters (e.g., temperature, different flow measures) were included at some stage during the modelling process.

The largest effect amongst the factors is that of the tidal regime (Tide OT), which explained 22% (ω^2 = 0.22) of the variation in DO levels. The site factors, depth and site code, together described 10% (ω^2 = 0.10) of the total variance and improved the model slightly when included as an interaction (Depth Code *Site Code). Regarding the overall DO mean, the aerator phases provided a surprisingly small impact of 5 %, but the separate impact of each factor has to be considered as well. This is provided by in the main effects plot (Figure 2-30), which illustrates the variance of DO levels due to each factor included.

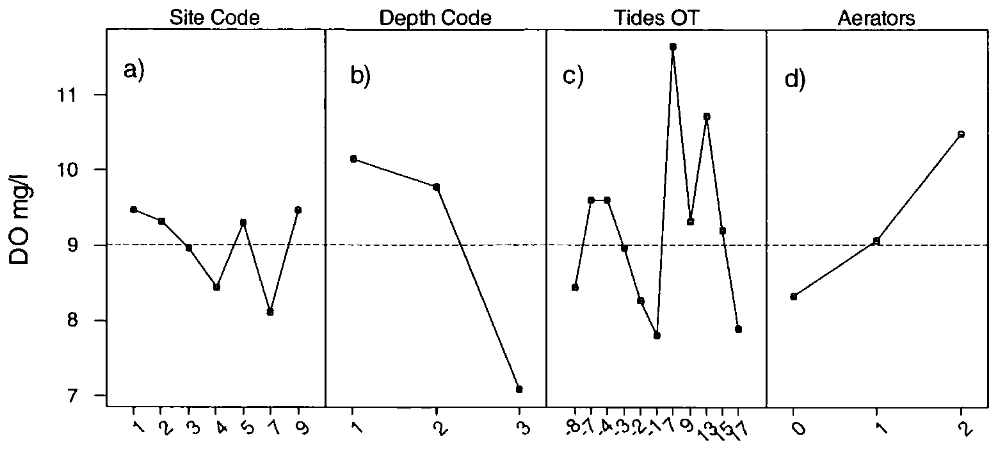

Figure 2-30 Main effects plot for DO mean levels in the Tawe, dotted line indicates overall mean

It is illustrated (Figure 2-30-a) that DO decreases with distance upstream, with exception of the shallower sites 5 (Whiterock) and 9 (Landore). The large influence of site depth is documented by the gap between DO means of epilimnial and hypolimnial layer (Figure 2-30-b). Significant variation is also indicated for the DO of different stages within the tidal cycle (Figure 2-30-c). During overtopping periods the DO is generally higher, with decreasing levels towards the end of the regime. For non-overtopping conditions, DO levels are lowest directly after the regime changed, while levels improve with prolonged non-overtopping. Levels at maximum periods (-8), however, were observed to drop below the overall mean again. The different treatment levels of aerators (Figure 2-30-d) induced a nearly linear increase in DO levels. While conditions were at 8.3 mg/l pre-aeration, the first phase of 44 aerators lifted the mean to 9.1 mg/l and the second phase of 87 aerators resulted in a shift to 10.53 mg/l. Although the main effects plot provides valuable information on factor specific impacts, it is equally important to discover interactions between the factors, e.g., sites where phase 2 aerators improved conditions most. Therefore, an interaction plot matrix (Figure 2-31) was created, in which the means for DO levels in relation to the factors site code, depth and aerators are given. Plot b) and d) represent the DO means for a two factor matrix of site code and depth code, while the effect of the third factor, aerators, is removed. In this, a pair-wise comparison of factors without the influence of the third factor is possible when analysing the graphs. Subsequently, the plots f) and h) describe the two factor matrix depth code and aerator phase, while plots c) and g) show the interaction between aerators and site code. In terms of expressiveness, it would have been sufficient to display only one of the pair-wise interaction plots, but the display of a full interaction plot matrix sometimes allows for better discrimination of significant factors.

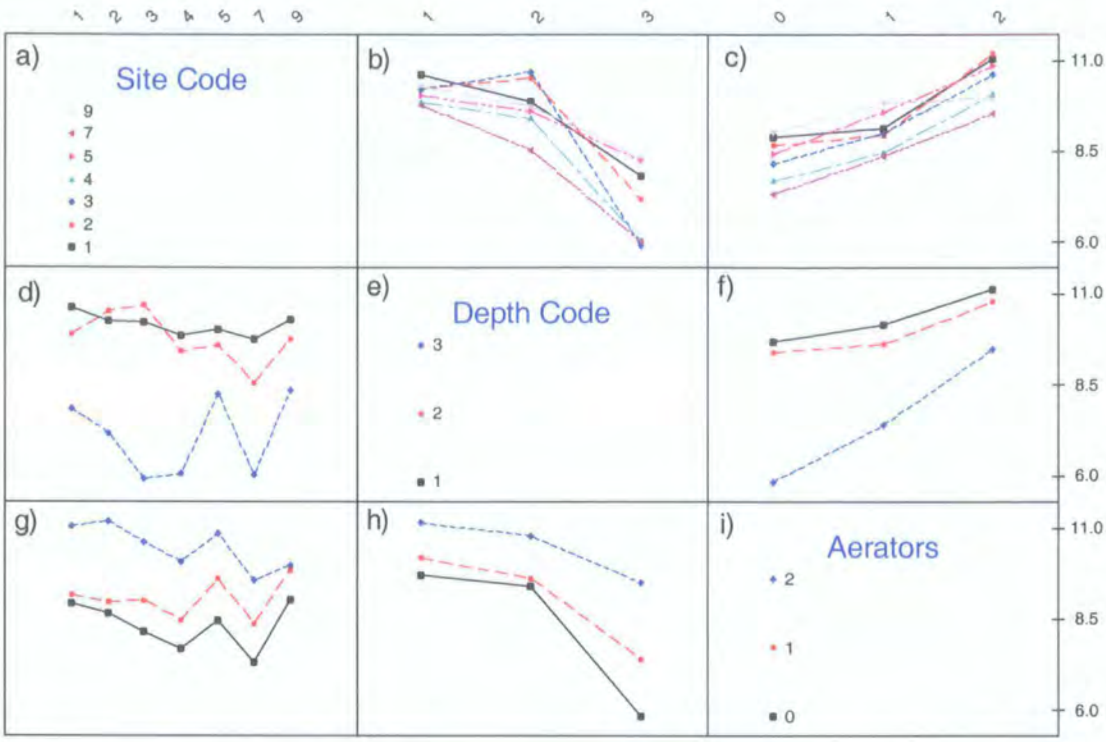

Figure 2-31 Interaction plot for DO mean levels in the Tawe, plot a), e) and i) indicate symbol definition for diffferent factor levels

Starting with plot d), it can be observed that the DO means for the near bed layers (Depth Code 3) are higher at sites closer to the barrage (Site Code 1 and 2) and for the shallow sites (Site Code 5 and 9), whereas low DO means are detected at the deeper upstream sites (Site Code 3,4 and 7). The large gap in DO levels between the hypolimnion (Depth Code 3) and the mid and surface layers (Depth Code 1 and 2) suggests stratified conditions throughout the profile. The DO at mid depth (Depth Code 2) distinguishes further from the surface water with increased distance from the barrage (Site Code 4-9), indicating enhanced stratification, while the downstream sites (Site Code 2 and 3) show reversed DO levels for mid and surface layers, suggesting mixed conditions for these layers.

The influence of the different aerator phases on the DO means at the three different depths (Figure 2-31 – f and h) suggests that their application provided an increase in DO means for all depths, with a maximum effect in the hypolimnetic layers (Depth Code 3). The first phase of aerators led to a 1.0 mg/l increase of the depth DO means. The mid depth mean levels were not affected by the introduction of the first phase (Aerators 1) of remedial measures, while the surface levels only showed a slight improvement with an increase of 0.3 mg/l DO. The second phase of aerators (Aerators 2), however, provided the main impact on the DO means at all depths and increased the near bed DO means by almost another 2.0 mg/l and the mid and surface DO levels by an extra 1.0-1.2 mg/l DO compared to aerator phase 1.

Considering the impact of distance and site depth on DO means, it is demonstrated in plot c) and g) that the aeration effect becomes more pronounced with distance upstream during aerator phase 1. Sites closer to the barrage (Site Code 1 and 2) only showed a slight improvement of DO conditions while the upstream sites (Site Code 3-9) indicated DO increases of about 1.0 mg/l as a result of the first 44 aerators installed. The addition of aerators in phase 2 provided a major jump in DO means throughout the longitudinal profile with average increase of 1.0-2.0 mg/l, compared to aerator phase 1. The only exception is the furthest upstream site 9, where DO levels were generally high, as indicated in plot d) and g).

Although the tidal regime (Tides OT) was mentioned as a factor to be included, the model was not able to calculate the interaction effect for three phases of aeration. Nevertheless, it was possible to include the tidal regime for modelling two aeration levels, e.g., 0 and 1 or 0 and 2, and the interaction plots for those models are given in Figure 2-32 and Figure 2-33.

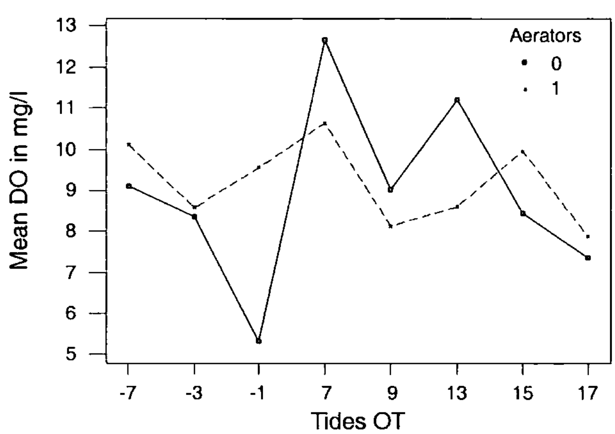

Figure 2-32 Interaction plot of DO means for tidal regime and aeration phase 0 and 1

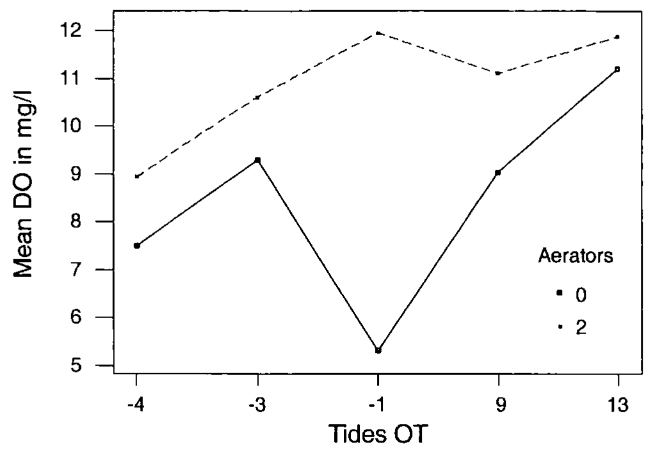

Figure 2-33 Interaction plot of DO means for tidal regime and aeration phase 0 and 2

The DO means in Figure 2-32 demonstrate that aeration phase 1 led to an increase of levels only during non-overtopping periods and towards the end of the overtopping cycle (15-17 Tides OT). The impact of phase 2 in Figure 2-33 is noticeable for all tidal conditions although the range of conditions here is smaller compared to Figure 2-32. Nevertheless, both aeration phases suggest a maximum effect on the DO means for the first non-overtopping tide (-1 Tides OT) and a minimum impact after long influx periods (13 Tides OT).

2.6.3 Results - Observational data analysis

To visualise the salinity and DO patterns observed during the course of the monitoring program 1994-2002, a selection of days is presented as contour plots below. Although the plots show a limited sample, they represent the main observed hydrodynamic patterns occurring under extreme flow, tidal range and tidal regime conditions. While Figure 2-34 and Figure 2-35 include salinity and DO contours of the period prior to aeration, the plots in Figure 2-36 and Figure 2-37 show conditions during aeration phase 1 and 2. Beginning with the pre-aeration period, the salinity and DO distributions during high freshwater flow conditions for two tidal conditions are given in Figure 2-34.

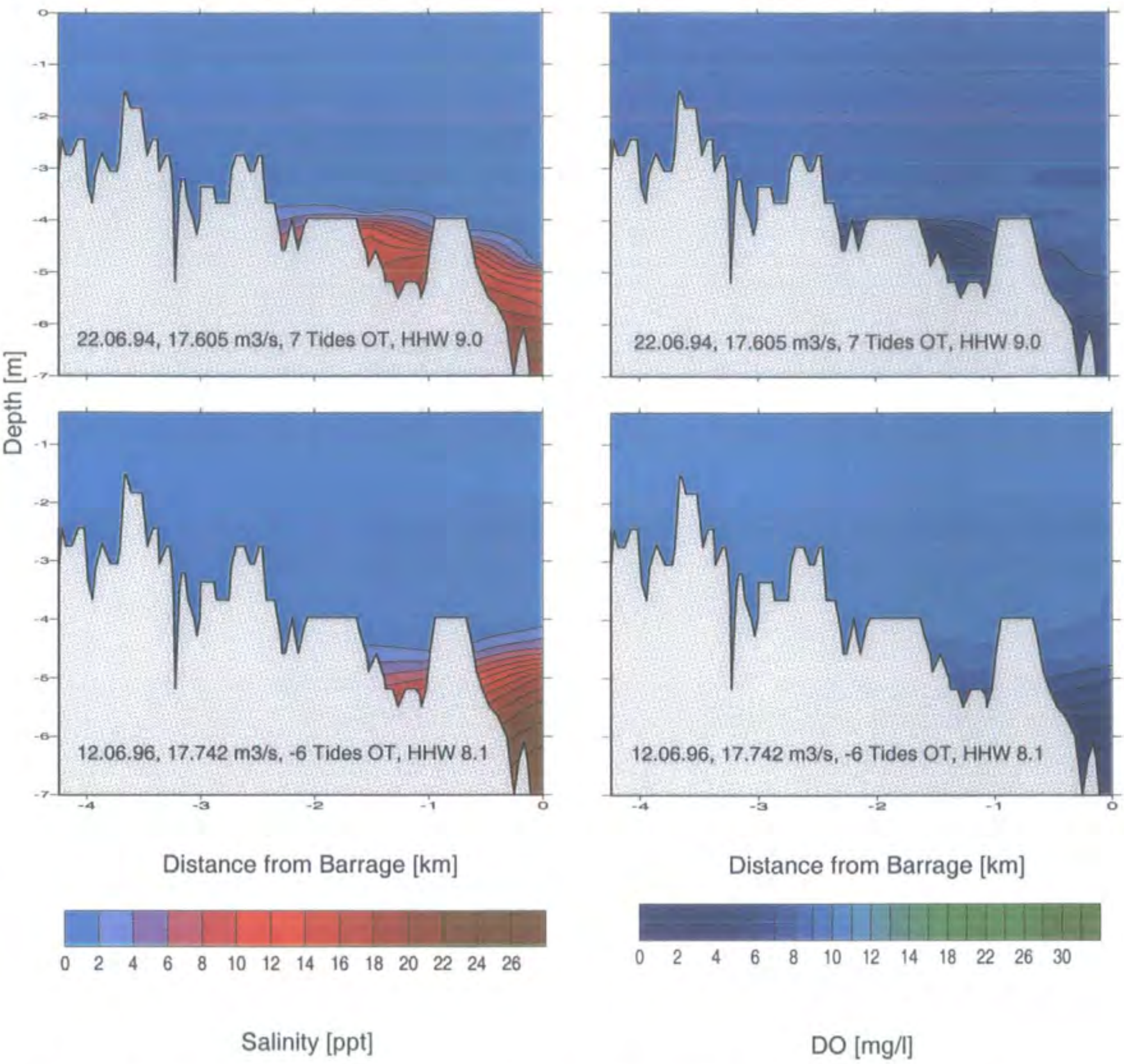

Figure 2-34 Longitudinal salinity and DO contours for high flows during pre-aeration period

The isohalines of 22/06/94 (Figure 2-34) show that, although seven tides overtopped the barrage at 9.0 m ACD, saline waters were restricted to deeper sections within a 2 km distance of the barrage, due to the high river discharge. The top 4 m of the profile is fresh, and only slightly reduced DO conditions were recorded at depths within the lower impoundment. The conditions look similar during non-overtopping periods of 12/06/96 (Figure 2-34) with stagnant saline waters and low oxygen levels only persisting in the vicinity of the barrage, due to the dominance of high freshwater flows.

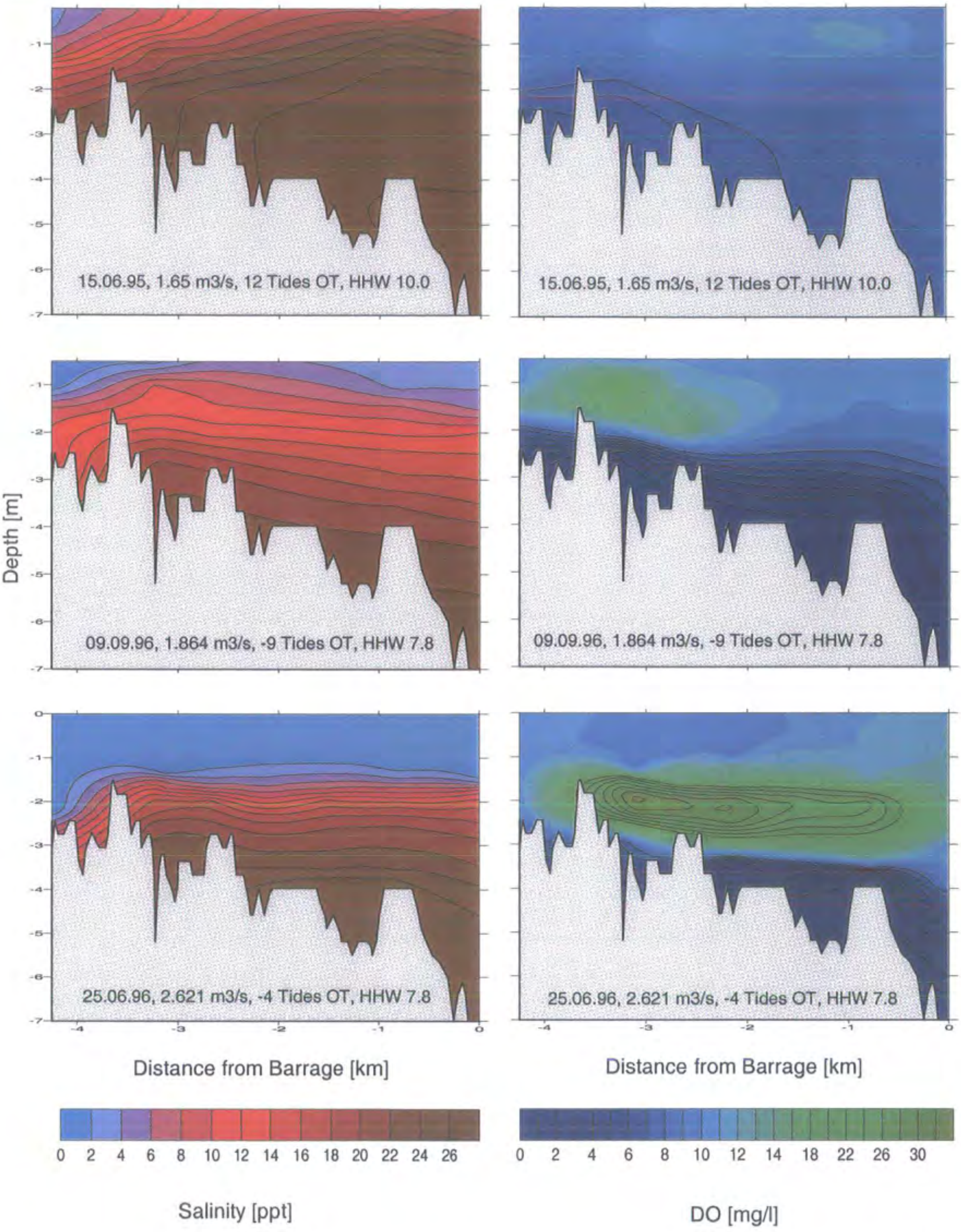

Figure 2-35 Longitudinal salinity and DO contours for low flows during pre-aeration period

The salinity and DO contours for low river discharges (Figure 2-35) present a completely different picture. Saline intrusion with levels >14 ppt dominate the profile on 15/06/95, after twelve tides overtopped the barrage at a maximum tide level of 10.0 m ACD. The extensive marine water intrusion provided saturated DO conditions for most of the impoundment, leaving only the upstream end (Landore, -4.25 km) slightly reduced. In contrast, stratified conditions are apparent after non-overtopping periods, as shown for 09/09/96 and 25/06/96 (Figure 2-35). Both profiles feature a stagnant saline wedge confined by a top layer of freshwater. The slight difference in freshwater flow resulted in a thicker freshwater layer on 25/06/96, narrowing the sharp salinity gradient to an approximate halocline depth of 2.0 m. The DO levels are reduced in the deeper sections, although adjacent to hypoxic conditions across the halocline. The oxygen profile on 25/06/96 presents a three layered system throughout the complete transect, with fresh water at the surface, hypoxic conditions at 2.00 m depth and a DO depleted layer near the river bed.

Since it was discovered that stratification is more likely to occur during low flow summer periods the following plots in Figure 2-36 and Figure 2-37 for the aeration phases focus on these conditions. Typical salinity and DO patterns are given for the first phase of aeration (with 28 units deployed within 1.2 km distance of Barrage and another 15 installed at Addis, 3.2 km upstream) in Figure 2-36. The contours of 16/07/99 indicate extensive salt intrusion after 15 tides overtopped at maximum level of 9.8 m ACD. Due to the low freshwater supply and the additional mixing of the aerators, a distinctive freshwater layer can only be identified at the furthest upstream end, leaving most of the profile at salinities >18 ppt. The DO levels also show a relatively homogenous distribution throughout the profile, as a result of fresh seawater ingress and introduced mixing. The contours of 23/06/99 during similar flow conditions, but after seven non-overtopping tides, present a different picture. The profile is stratified, with saline waters trapped in the deep hollows of the upstream sites (Foxhole 2.1 km, Addis 3.2 km), while the sites further downstream show a lower salinity at depth. This is also reflected in the DO profile, which indicates almost anoxic conditions for the stagnant upstream sections and reduced DO levels at downstream depths.

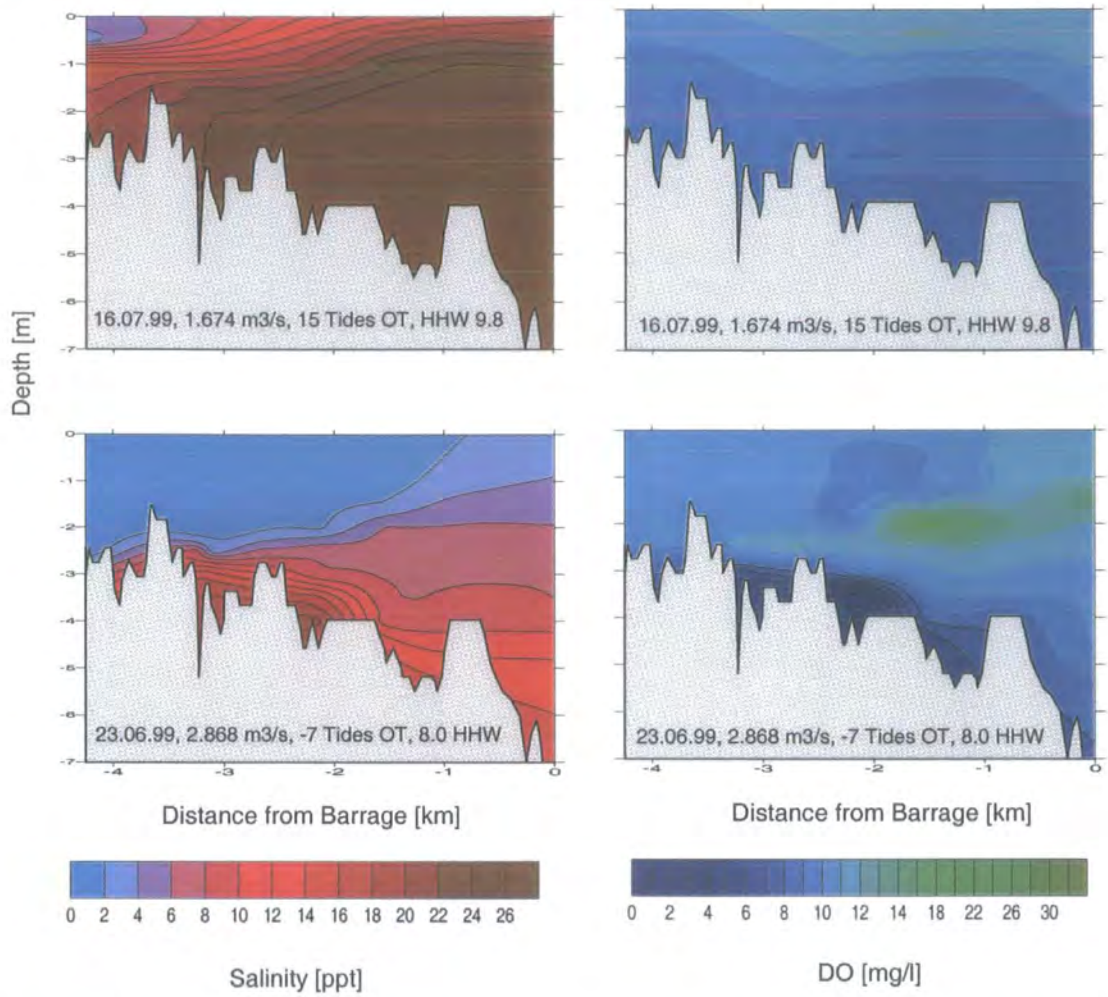

Figure 2-36 Longitudinal salinity and DO contours for aeration period phase 1 during low flows

The contours for aeration phase 2 (Figure 2-37) indicate that, for periods of overtopping, similar salinity contours can be observed when compared to the low flow overtopping profile during aeration phase 1.

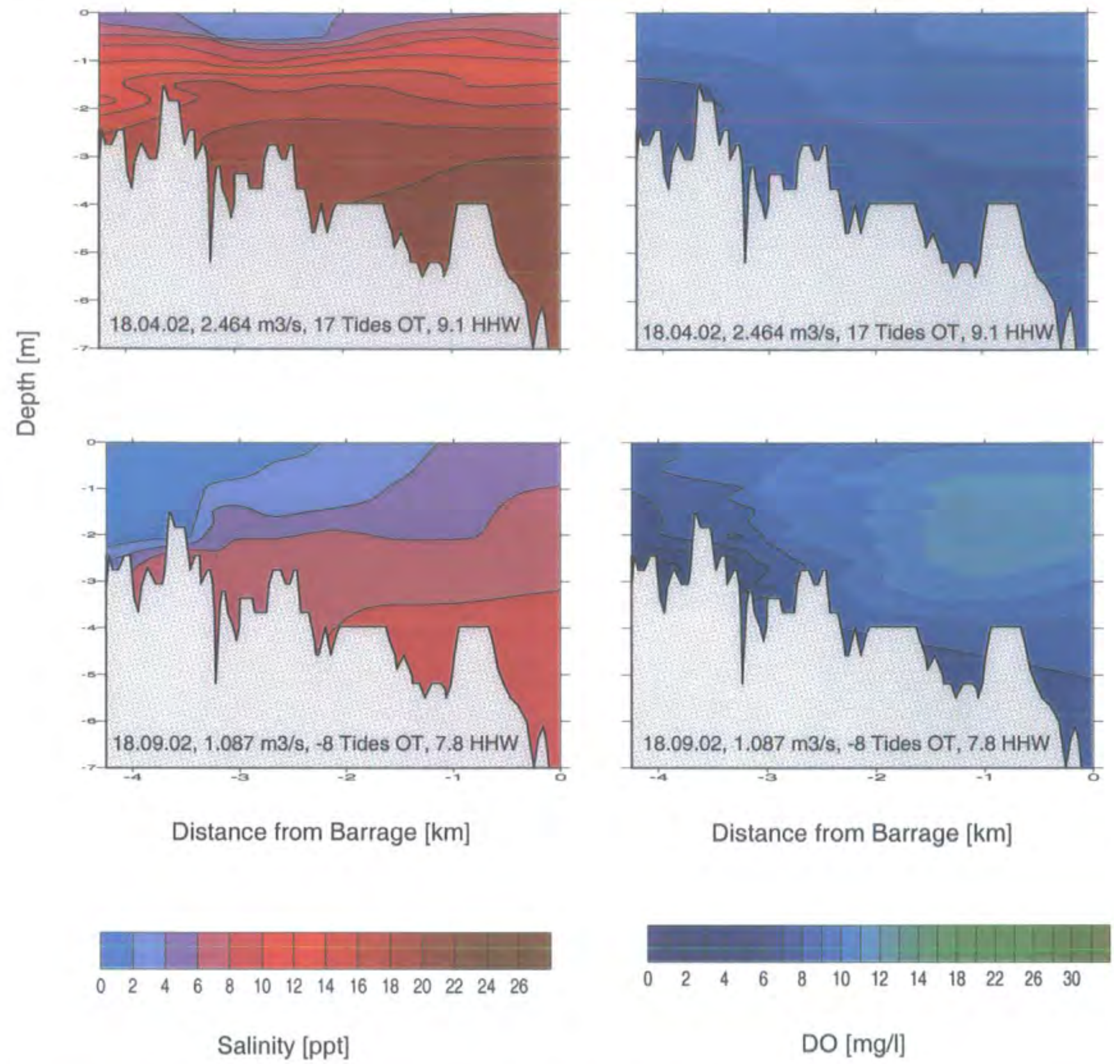

Figure 2-37 Longitudinal salinity and DO contours for aeration period phase 2 during low flows

The profile of 18/04/02 (Figure 2-37) after 17 tides overtopped at a HHW of 9.1 m ACD demonstrates that the saline wedge had moved well into the estuary, but also indicates a lower salinity for the metalimnetic layers (1-3 m Depth) when compared to the similar conditions during phase 1 (16/07/99-Figure 2-36). The DO profile shows well-mixed conditions with a slight decrease of levels at the upstream end (Landore, 4.25 km). The profile of 18/09/02 (Figure 2-37), during very low river discharges and after eight non-overtopping tides, does not appear to be as critically stratified as the comparable profile of the 23/06/99 (Figure 2-36) in aeration phase 1. Depth salinity ranges from 8-12 ppt, and arrested pockets of salt water cannot be observed in the rather homogenous profile. However, reduced DO conditions were recorded for the deeper layers with lowest concentrations at the upstream end.

2.6.4 Discussion

The findings from the ANOVA modelling in 2.6.2 in conjunction with the observed data given in 2.6.3, indicate that the introduction of mixing equipment significantly improved the DO conditions within the impoundment. The study also demonstrated a near doubling of improvement when the number of aerators was doubled in phase 2. An increase in DO levels was mainly observed at depth, which was the main target of the remediation campaign since the difference between mid-depth DO and surface means was relatively large. This is a result of the stratified conditions, which completely isolate the deep layers from surface and mid-depth layer re-oxygenation. The break-down of stratification by the aerators resulted in a dilution of the DO reduced bottom waters, while the mid-layer reacted as buffer zone, which led to a smaller increase of DO level improvement, compared to the surface layer. The initial gap to the DO means of the mid-depth layer and the effectiveness of the equipment in raising the DO levels became more pronounced with distance from the barrage. Furthermore, it was observed that the aerators were less effective during periods of overtopping tides, with a slight reversal towards the end of the tidal regime. These results suggest that the aerators perform de-stratification rather than re-aeration. The sites closer to the barrage are subject to longer periods of mixing, while extended periods of stratification occur at further upstream sites, increasing the likelihood of low DO conditions developing. Therefore, the aerators performed most effectively at deep sites further upstream, where pronounced stratification was commonly observed during the pre-aeration period. It was suggested previously, that the build up of stratification is further restricted to non-overtopping periods, which explains the increased effectiveness of the equipment during these tidal conditions. The question of whether an incoming tide actually overtops at the given construction level of 8.05 m ACD was discussed under 2.3.5.2 and could explain the reversal of effectiveness towards the end of the tidal regime. Given an overtopping level of 8.30 m ACD, would re-define the last tides of an overtopping period extending the non-overtopping periods.

The provision of freshwater within the impoundment is of major importance for the performance of the installed aerators. As it was suggested in section 2.4.4, freshwater flows of $Q_s \geq 12$ m^3/s introduce sufficient erosion at the FSI to break up stratification and flush out the saline wedge during non-overtopping periods. Therefore, the equipment is not required during these conditions and could be switched off. Nevertheless, a few aerators installed in the deepest hollows along the river bed (Site 3, 4 and 7-Figure 2-29) could prevent possible isolation of saline pockets and aid in clearing out arrested saltwater in the vicinity of the barrage (Figure 2-34). In addition to the impact on the breakdown of stratification during non-overtopping periods, the amount of freshwater also restricts the up-estuary marine water intrusion during overtopping periods, as seen in Figure 2-20 and Figure 2-21. Dependent on the flow conditions during the highest spring tide levels, certain aerator sections were not required and performed less effectively compared to deeper sites further downstream. Site number 9 is an example which showed only minor improvements of DO conditions although 20 aerators (Figure 2-29) were installed there. The area around this site is generally freshwater dominated and low DO conditions rarely occur as a result of salinity stratification. Furthermore, it was seen from the observational data that the equipment was successful in preventing the build up of a pronounced halocline within the impoundment. Whereas low flow, non-overtopping periods featured sharp salinity gradients at depths around 1.0-2.0 m (09.09.96 and 25.06.96-Figure 2-35) pre-aeration, this was not apparent during similar conditions for aeration phase 1 and 2. Subsequently, algal blooms that tend to be on top of the halocline, due to reduced velocities in that layer (Figure 2-35), disappeared with the introduction of the aeration scheme and are now restricted to sites where less mixing occurs.

2.6.4.1 Conclusions

The two phase installation of aeration equipment in the Tawe during 1999 and 2000 had a significant impact on the density stratification and oxygen conditions resulting after impoundment of the estuary in 1992. The performance of the system was dependent on the tidal conditions, site properties and freshwater flow. The following patterns were observed during numerical and observational data analysis, in which monitoring data of days prior to aeration were compared with data recorded during aeration phase 1 and 2.

- The complete aeration system led to an increase in mean DO levels of 3.0 mg/l for the near bed layer, 1.0 mg/l for the mid depth layer and 1.8 mg/l for the surface layer. Prior to aeration, the near bed layers showed significantly lower DO conditions, due to saline stratification which did not allow re-aeration by the fresh top layer. Therefore, the remediation potential was greatest at depth, while the mid layer acted as a buffer for the mixed up bottom layers.
- The equipment showed the highest efficiency during non-overtopping conditions, since stratification is likely to occur then, due to the hydrodynamic behaviour of the salt wedge. Tidal overtopping introduces a considerable amount of mixing and provides saturated DO conditions, depending on the amplitude and excursion

length of the incoming tide. Therefore, sites closer to the barrage, which are exposed to higher frequencies of tidal mixing, showed lower efficiencies within the aeration scheme.

- The application of the scheme should be adjustable to the freshwater inflow. High flows of $Q_s \geq 12$ m^3/s introduce sufficient shear to the FSI to provide flushing of the saline wedge. In addition, the up-estuary marine intrusion is related to the freshwater flow during the HHWL, so that certain upstream sites might not require the application of mixing devices during non-overtopping periods.
- A limited number of aerators situated in the deepest sections of the river bed should constantly be applied during summer conditions and at all flows to avoid trapping of saline pools by the falling halocline.

3 Baffles – The Boom Skirt System

3.1 Introduction

An alternative approach for managing marine water propagation within impoundments is the installation of screens or baffles. The idea was first outlined by Schijf and Schönfeld (1953) who suggested a fixed screen upstream of partial exclusion barrages to utilize the limited amounts of freshwater more effective during sluiced flushing of saline water (Figure 3-1).

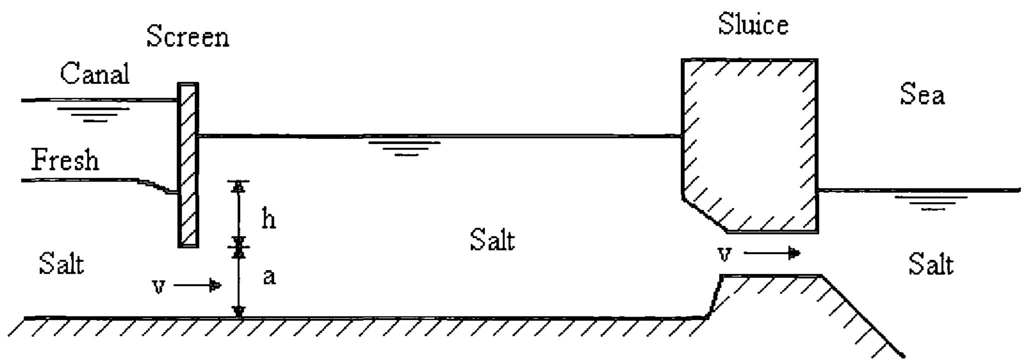

Figure 3-1 Sluice with screen for desalting (Schijf and Schönfeld, 1953)

It was suggested that the lower edge of the screen is situated below the freshwater-saltwater interface (FSI) to confine the opening (a-Figure 3-1) for the deep saline flow which results in increased velocities (v-Figure 3-1) and thus in higher expulsion rates. The authors already anticipated the sensitive task of regulating the flow velocity below a double critical value (v_{cc}) to avoid freshwater expulsion through depression of the FSI surface (h-Figure 3-1) and to prevent interfacial mixing.

Twenty-five years later, Karl Dunkers, a Swedish research engineer (Soderlund, 1988), developed a method for the storage and primary treatment of combined sewer overflow (CSO) discharges in a receiving water body by the means of suspended plastic curtains. The idea of density containments in stratified water bodies was followed by Cataldo et al. (1987), who recommended a baffle system for the storage of CSO discharges in a tidal creek. The authors modelled the prototype in a laboratory flume tank (1:36 scale) to examine the hydrodynamic behaviour of the baffles on the stratified flow and concluded that the buoyant CSO discharge can be retained by the baffle enclosure. However, the effectiveness of the so-called 'flow balance method' (FBM) was dependent upon the stability of the stratified layers, which were affected by the simulated tidal motions of the underlying saline layers and the discharge volume of the CSO. From 1987 to 1990, the system was implemented as a prototype when an arrangement of plastic curtains suspended from pontoons was installed at the outlet of four CSO barrels at Fresh Creek, New York City (Field et al, 1995). The containment, with a capacity of 1550 m^3, successfully proved that the initial idea of vertical displacement by density difference could work in a marine environment. The formation of a stable 'freshwater' (CSO) layer occurred and was retained for pumpback to the wastewater treatment plant. Specific conductance was chosen to distinguish between CSO and creek water during back-pumping while little mixing of the layers was monitored within the confinement due to the underdesign of the FBM. As a result of the small capacity, the design suffered from so called 'blow outs' during which excess CSO water was released underneath the billowed out curtains which decreased the overall storage efficiency substantially. Nevertheless, a CSO event (984 m^3) appropriate to the capacity of the FBM led to a storage efficiency of 77 % whereas the residual freshwater loss was explained by interfacial mixing.

The hydrodynamic behaviour of a series of interconnected enclosures within a marine environment, for the purpose of water treatment rather than storage, was investigated in a feasibility study by Burrows et al. (2001). The so-called 'Marine-based Waste Stabilisation Pond (MBWSP)' concept developed by Alpinconsult/LTI features booms with suspended flexible curtains that are weighted to resist the net force resulting from different hydrostatic pressures on both sides (Figure 3-2).

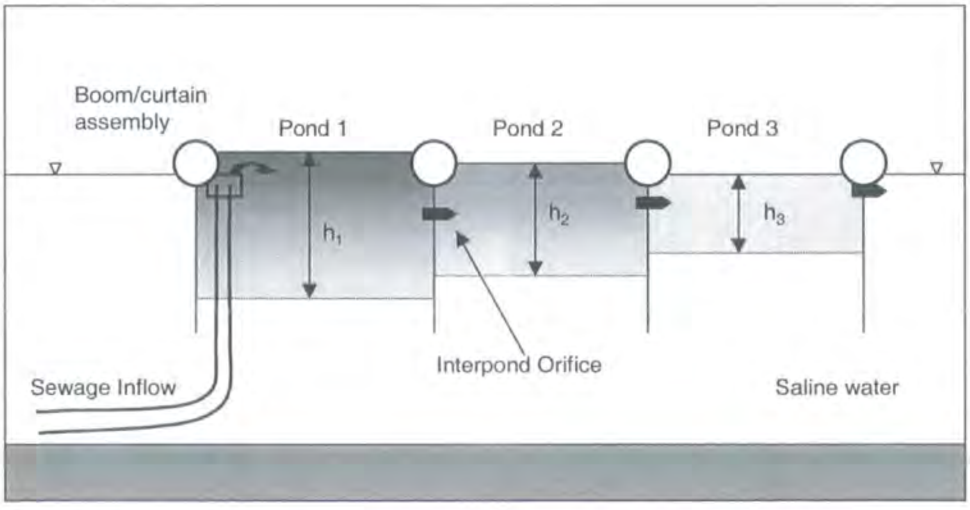

Figure 3-2 Schematic MBWSP concept with interpond orifices (after Burrows et al., 2001)

The system utilizes density stratification and positive replacement by applying an influent to a series of bottomless ponds floating in a denser saline environment. The water at different treatment stages enters and leaves the baffled ponds through orifices or under-skirt flow depending on the design. As with previous applications, the success of the system depends on the stability of the stratification and on the appropriate dimensional layout. Laboratory studies and numerical modelling with computational fluid dynamics (CFD) software revealed only minor disturbances of the FSI by wind induced currents as well as stability against overtopping seawater and an overall loss of 1 % of the influent for large under-flow currents. Therefore, the authors recommended the concept as a low-cost, self-stabilizing alternative when land availability is limited and low-tech maintenance is required.

The flow diverting function of vertical curtains installed at the inlet of an eutrophic reservoir was investigated by Onishi et al. (2000). The curtains created an underflow (interflow-Figure 3-3) of nutrient rich waters passing the euphotic epilimnetic layer, thus, reducing the amount of algal bloom in the reservoir. Creation of an interflow was assisted by the plunging of low temperature (high density) inflows and mid-level withdrawal at the dam.

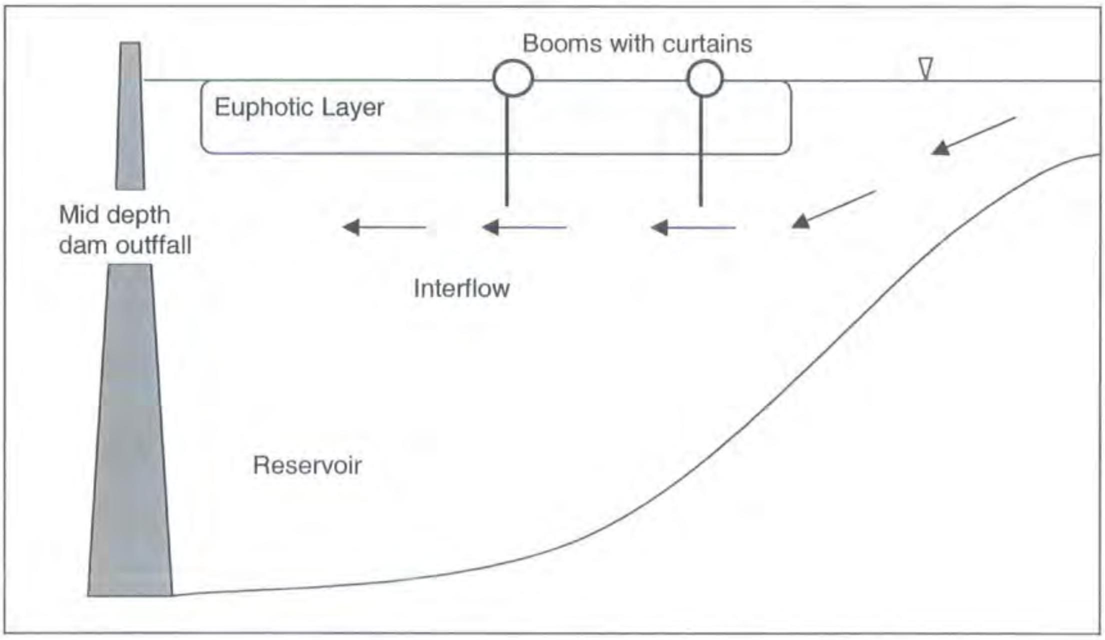

Figure 3-3 Eutrophic reservoir with curtains (after Onishi et al. 2000)

The application investigated in this study can be seen as a mixture of stratification enhancement and flow directing. Burrows and Ali (2001) revisited Schijf and Schönfeld's initial idea, utilising flexible materials to study the hydrodynamics of curtains in stratified estuarine impoundments. Consequently, they were approached by Durham University for a joint investigation on remedial applications of this concept.

3.2 Aim of study

Experiments were undertaken to determine the hydrodynamic behaviour of the baffle concept in a stratified impounded environment and to assess the feasibility and limits for future applications. This research was conducted on a laboratory scale since appropriate field sites were not available. The intention is to apply a boom/skirt pilot scheme within an impoundment subject to saline influx in order to investigate the hydrodynamic patterns and resilience of the arrangement under field conditions. The results described below offer crucial information on the principle design considerations for the field model regarding alignment and deflective behaviour.

3.3 Theoretical considerations on stratified flow

In order to describe the behaviour of stratified fluid flows, a number of parameters have been introduced in the relevant literature (Turner, 1973, Abraham et al., 1979; Yih, 1980). The most important of these for the characterisation of the observed hydrodynamic patterns in this chapter are:

- Reynolds number
- Froude number
- Richardson number
- Entrainment rate

3.3.1 Reynolds number

The *Reynolds number* describes the way a liquid travels and it allows the distinction between layered and turbulent motion. The dimensionless *Reynolds number* is derived from a comparison of inertia and viscous forces within a fluid and subsequently describes the resistance to flow forces defined as:

$$\mathrm{Re} = \frac{\rho U D}{\mu}$$

Equation 3-1

Where ρ is the density of a layer [kg/m^3], U the layer velocity [m/s], D its depth [m] and μ the molecular viscosity of the fluid [kg/ms], given as μ=0.00114 kg/ms for water (at 15°C) (Kay, 1998). Since it was discovered that an intermediate layer forms by molecular diffusion between two layers of different density which does not necessarily follow flow patterns of the adjacent layers, an *interlayer Reynolds number* was introduced by Schijf and Schönfeld (1953). This is defined as:

$$\mathrm{Re} = \frac{\Delta v \Delta z}{\upsilon}$$

Equation 3-2

Where Δv is the difference in velocities as measured just above and below the interlayer [m/s], Δz is the interlayer thickness [m] and υ is the kinematic viscosity of the fluid [m^2/s], given as the ratio of ρ and μ. For Reynolds numbers Re<2000 laminar flow is assumed, while Re>4000 indicate turbulent conditions. A value between these margins describes the transitional zone in which the conditions are unstable since the transition from turbulent to laminar and vice versa takes place randomly (Lewis, 1997; Kay, 1998).

3.3.2 Froude number

The dimensionless Froude number relates the longitudinal velocity of a flow to the maximum velocity of progressive wave propagating along a density surface. In the case of surface waves, the speed is defined with the gravitational acceleration as the restoring force, but for internal waves this force is a *reduced gravity g'* resulting from the density difference of a two layered system, given as:

$$g' = g\left(\frac{\rho_1 - \rho_2}{\rho_2}\right)[m/s]$$

Equation 3-3

Where ρ_1 and ρ_2 are the densities of the upper and lower layer respectively [kg/m^3] and g is gravitational acceleration [9.81 m/s^2]. The reduced or effective gravitational acceleration is used to determine the maximum internal wave speed, given as the celerity:

$$c = \sqrt{(g' h_1)}$$

Equation 3-4

Where h_1 is the thickness of the upper layer [m]. The ratio of the longitudinal velocity U [m/s] to the internal wave speed c [m/s] is defined as the internal or densimetric Froude number:

$$F_i = \frac{U}{\sqrt{(g'h_1)}}$$

Equation 3-5

The Froude number is therefore a measure for the conditions allowing movement of internal waves against the direction of flow. For $F_i > 1$ the longitudinal flow dominates the system and so-called 'super-critical' conditions exist. Wave motions at the interface travel only downstream with the flow and small disturbances, i.e. through obstacles in the flow, might be broken down depending on the value of F_i. As c approaches U, the propagation of internal waves becomes more dominant and for $F_i<1$ the conditions are defined as 'sub-critical', since the movement of internal waves is not restricted by the flow. Since the critical value for F_i is unity, the theoretical assumption is that salt wedges will not form in estuaries for $F_i>1$. In practice, however, it has been shown that saline wedge formation is prevented at F_i values as low as 0.6 (U.S. Army Corps of Engineers, 1993; Lewis, 1997).

The restriction of flow leads to an increase in velocities which can result in the occurrence of localised super-critical and sub-critical conditions (Figure 3-4).

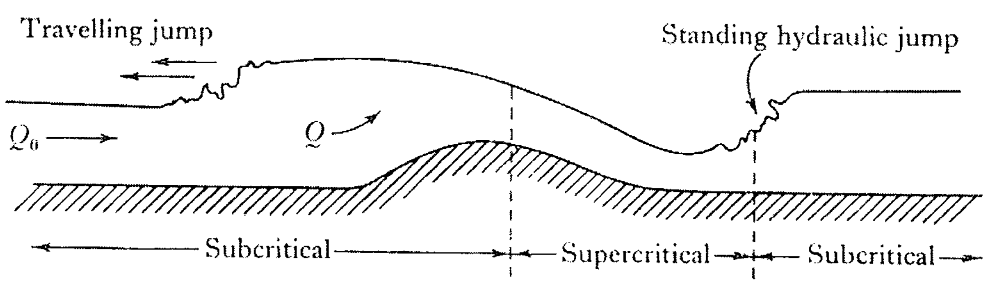

Figure 3-4 Travelling and standing hydraulic jumps associated with flow constriction (Turner, 1973)

Similar to surface jumps (i.e. surges, bores) internal jumps are created as a result of the different Froude numbers. If the flow increases from sub-critical to super-critical around an obstacle an upstream travelling jump can be created while the backwards transition downstream can lead to a standing hydraulic jump. Baddour and Abbink (1983) and Baddour (1987) investigated the behaviour of a super-critical underflow discharge of seawater into a confined channel of freshwater to determine maximum interfacial mixing conditions. Their experiments revealed the occurrence of a relatively short mixing zone featuring an internal hydraulic jump and a sub-critical counterflow region characterized by a stable interface (Figure 3-5).

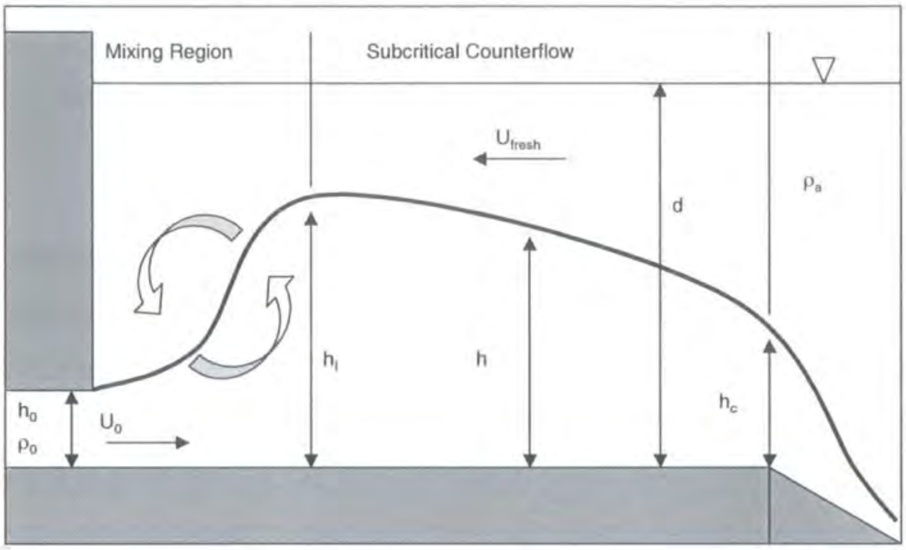

Figure 3-5 Definition sketch of underflow discharge channel (after Badour, 1987)

By varying the control parameters upstream (h_o) and downstream (d) of the mixing compartment, a total of three mixing modes (Figure 3-6) were observed which provided different dilution rates between the water bodies.

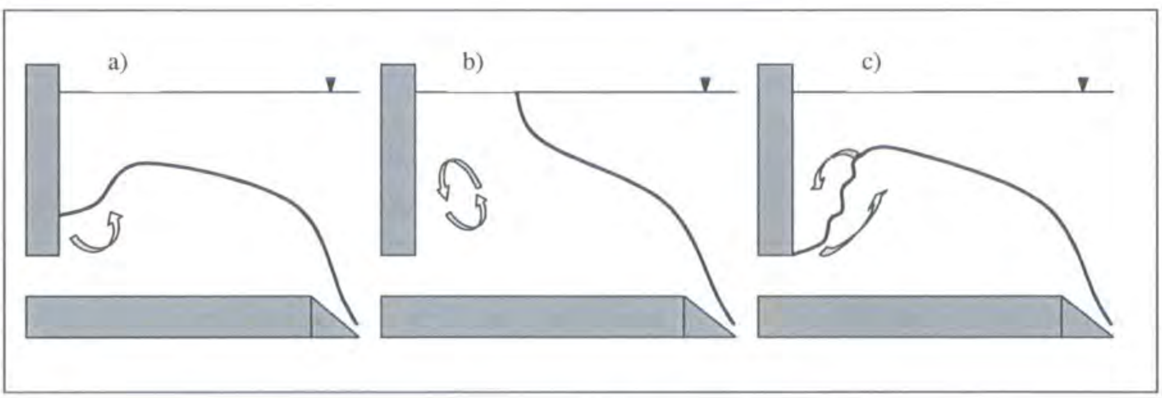

Figure 3-6 Mixing modes observed by Baddour (1987)

For a higher amount of downstream control, i.e. a decrease in h_c (Figure 3-5), it is anticipated that the hydraulic jump will travel upstream, resulting in a drowned discharge exit (Figure 3-6 a)). This *drowned internal jump* reduced the amount of dilution significantly. When the discharge Froude numbers exceeded $F_0 \cong 15$ through the variation of h_o

(upstream control), an *upstream control instability* developed, in which the internal jump reaches the surface boundary (Figure 3-6 b)). The same instability could be triggered by lowering the overall depth of the compartment, via downstream controls, so that the depth confinement would physically restrict the development of an internal or drowned jump referred to as *downstream control instability* (Figure 3-6 b)). The perfect balance between those interacting controls ensures the development of a *free internal jump* (Figure 3-6 c)), which provided maximum dilution rates of the water bodies through turbulent entrainment at the interface.

3.3.3 Richardson number

There are several applications of different Richardson numbers mentioned in the literature (Lewis, 1997; Kay, 1998). One of the main purposes of this dimensionless parameter is to quantify the degree of stability, i.e., stratification, and the subsequent ability to inhibit vertical transfer of momentum and matter.

The *gradient Richardson number* R_i describes the damping forces in a two layered system against velocity shear at an interface and is defined by:

$$Ri = \frac{g(\Delta\rho / \Delta z)}{\rho(\Delta u / \Delta z)}$$

Equation 3-6

Where the stability is described by the vertical density gradient ($\Delta\rho/\Delta z$) [kg/m^4] and the destabilizing forces of velocity shear are included by the vertical gradient of horizontal velocity ($\Delta u/\Delta z$) [1/s]. Because of the implied difficulties in measuring the local gradients of a two-layered system precisely, another Richardson number is more commonly applied which is defined as the *bulk* or *layer Richardson number*

$$Ri_L = \frac{(\Delta\rho / \rho_m) gD}{\Delta(u)^2}$$

Equation 3-7

Where the effective gravitational acceleration $g' = (\Delta\rho/\rho_m)g$ [m/s^2] of the moving layer of D=depth [m] is compared to the net velocity shear represented by Δu (relative velocity to stagnant layer) [m/s]. The *layer Richardson number* describes the characteristics of the whole flowing layer as opposed to the very localised *gradient Richardson number*. However, the Ri_L approaches Ri for decreasing layer thickness values.

In general, it can assumed that for Ri>0 the stratification is stable and damping of velocity shear occurs at the interface, such that mixing is reduced to molecular diffusion only. As Ri approaches 0, the conditions become neutral, where stratification is not significant for the mixing processes, and for Ri<0 stratification is unstable which refers to a homogeneous fluid.

Since the Richardson number is a measure of interfacial stability, it can be used to evaluate the development of instabilities, such as Kelvin-Helmholtz billows (Figure 3-7). These large billows form at conditions of Ri<0.25 induced by velocity shear on the interface. They feature a vortex that eventually breaks down into a turbulent core, resulting in entrainment of the two involved layers. For Ri>0.25 smaller scale mixing processes, in which current shear is introduced to a stable interface, are represented by interfacial Holmboe waves (Figure 3-8). These cusp-like progressive waves break down and eject elements from a denser layer into the lighter layer causing entrainment.

Figure 3-7 K-H-Billows (from Van Dyke, 1982)

Figure 3-8 Interfacial Holmboe waves (Dyer, 1997)

3.3.4 Entrainment rate

Entrainment describes the process of mass transport across an interface generated by velocity shear. The rate of entrainment is the interfacial net movement of particles, which is ultimately due to molecular diffusion induced by the thermal movement of molecules. Nevertheless, molecular diffusion is rarely of direct influence and rather a result of larger scale mixing processes as described in the previous section. The effect of mixing on the rate of entrainment in open-bottomed containments was studied extensively during flume tank experiments conducted by Moore and Long (1971) and Christodoulou (1986). The loss of particles into another layer is commonly measured in terms of an entrainment coefficient E (Foo et al.,1995), defined as:

$$E = \frac{W_e}{U}$$

Equation 3-8

Where W_e= is the entrainment velocity, usually described as the vertical displacement of the density interface [m/s], and U is the mean velocity of the moving layer [m/s]. E is a suitable measure for quiescent conditions but it does not account for the fact that mixing conditions rapidly change layer thickness and mean velocities across the boundary. The change of these conditions around and across the interface is a measure of Ri (Section 3.3.3) which lead to the proposal by Christodoulou (1986) and Shuy et al. (1998) to describe E as a function of the bulk Richardson number:

$$E = CRi^{-k}$$

Equation 3-9

Where *C* and *k* are empirical constants for different ranges of Ri representing the change of predominant mixing mechanisms.

3.3.5 Relevance of parameters

The relevance of the described parameters for the outlined study lies in their description of the predominant conditions that are present. Since the experiments were concerned with the stability of stratification and the nature of the flow conditions, it was necessary to express the observations in standard variables for comparison with existing research.

Although the following study is not completely based on these parameters, they were utilized to explain certain observed patterns. Thus, they have to be considered as a support for the results of the experiments, which were mainly based on the visual evidence gained.

3.4 Methods

For the purpose of this study, four flume tank experiments were conducted in the fluid laboratories of the Engineering Department at the University of Liverpool over the course of two weeks. Conditions of an impoundment, subject to tidal influx, with a screen arrangement fitted as a remedial measure were simulated. The design involved a rectangular flume tank (Figure 3-9) made out of transparent Perspex with a barrage construction fitted that divided the flow into an upstream freshwater reservoir and a downstream seawater compartment. The flume was slightly tilted to simulate a bed slope and included an overflow point at the seaward end. One tank at each end of the flume ensured the provision of the different inflows. A visible distinction between the freshwater (ρ_f=1000 kg/m^3) and the denser saltwater (ρ_s=1026 kg/m^3) was ensured by adding a florescence dye to the saltwater before each run. In order to observe flow velocities at the interface and mixing conditions within the layers, rhodamine B dye was injected with a fine needle during the experiment.

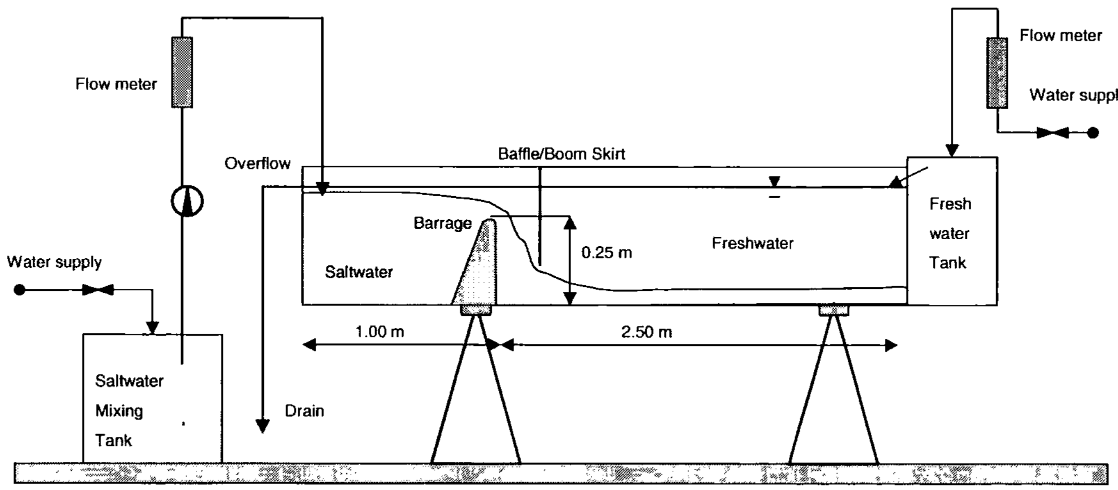

Figure 3-9 Schematic of the barrage flume tank

The findings of this study were based mainly on visual observations made during the experiments, supported by a time series of photographs taken from a fixed position near the flume. An ultrasonic velocity profile monitor was also installed above the barrage crest to observe the velocity distribution of the water column at this position. Although the study was based on the visible distinction between fresh and saltwater, the findings were backed up by vertical density measurements taken with a handheld meter 0.25 m upstream of the barrage.

The model design for the hydrodynamic simulation was based on the conditions found at the Tawe Barrage in Swansea (Section 2.3). Due to the difficulties in quantifying the amount of overtopping seawater under drowned weir conditions, a simplified approach was applied for modelling the saline influx. The influx of the prototype impoundment under spring tide conditions was calculated using the total volume rise over a tidal cycle. The maximum extra height over the impoundment area of approximately 24.67 hectares is 2.25 m (10.4 m HHWL – 8.15 average crest level) and it takes 2 hours for the tidal amplitude to reach this maximum. This results in an average tidal influx of 77 m^3/s, which is 1.078 m^3/s per m crest width, assuming a total overtopping length of 71.5 m (45 m Secondary Weir, 19 m Primary Weir and 7.5 m Fish Pass). This average saline influx was taken to simulate a tidal cycle in the model. The freshwater inflow was calculated from the mean daily flow value of 12.13 m^3/s and results in a flow per unit value of 0.169 m^3/s per m crest width for the prototype. The height difference of the prototype barrage and the model leads to a scale ratio of 0.25 m/8.15 m=0.0307 (≈1:33). Applying the scale ratio to the calculated prototype flows gives:

Q_{ms}=1.078 m^2/s*m width * 0.0307=0.033 m^3/s*m width
Q_{mf}=0.169 m^2/s*m width * 0.0307=0.005 m^3/s*m width

For a flume width of 0.1 m, the model's salt and freshwater inflows are $33.095*10^{-4}$ m^3/s (3.31 l/s) and $5.188*10^{-4}$ m^3/s (0.518 l/s) respectively. Considering the length of the prototype impoundment l_p=4.25 km, a flume tank of l_m=130 m would have had appropriate capacities to simulate the calculated inflows. However, the available flume was only 2.5 m in length, leading to reduction scale of 1:50 in order to avoid an immediate flooding of the model. Subsequently, the following model inflows were applied:

Q_{ms}=$6.619*10^{-5}$ m^3/s=0.0662 l/s
Q_{mf}=$1.019*10^{-5}$ m^3/s=0.0 103 l/s

Since the lower impoundment of the prototype was found to be completely saline at spring tide levels of 10.4 m ACD, the timing of the saltwater inflow in the model was determined by the maximum capacity of the upstream compartment of the flume. In order to fill the model impoundment with a volume of V_m=0.0625 m^3=62.5 l of saline water, the inflow of Q_{ms}=0.0662 l/s had to run for T_s=15.73 minutes. Considering the freshwater flow in the prototype is pushed upstream by the saltwater under average flow conditions, the freshwater flow entering the model impoundment had to be delayed. Following several flow test runs, the time scheme was agreed as follows:

0 minutes:	experiment starts with saline water overtopping the barrage
10 minutes:	freshwater flow starts
15 minutes:	seawater flow stops
15-60 minutes:	freshwater flow continues until freshwater expulsion becomes predominant which ends experiment

A similar density difference of the two competing water bodies was applied in the model utilising a saltwater density of ρ_s=1026 kg/m^3, ensuring comparable densimetric Froude and Richardson numbers with the prototype.

While the first experiment was carried out with a Plexiglas baffle installed in the impoundment, the other experiments utilized a boom/skirt assembly. The skirt was made out of nylon, coated with weldable polyether. The nominal weight of the material was 310 g/m^2. The boom was made out of Perspex with a diameter of 3.3 cm and length of 10 cm to match the width of the flume compartment. To form the assembly, the skirt was attached to the boom by a screwed on metal bar which also allowed for variable lengths. At the bottom end of the skirt, a sleeve was formed in which iron rods could be placed in order to vary the weight of the skirt. Three different weight categories of iron rods were available but findings of earlier hydrodynamic tests (Whyte, 2003) suggested the use of only the light (21 gram (g)) and the medium (27 g) weight for this configuration.

The four experiments were carried out with varying screen types and different weights which changed the underflow openings. The parameter changes in each experiment were as follows:

- Experiment 1: Plexiglas baffle installed at 10 cm distance from the barrage with an underflow opening of a=2.1 cm. Flow regime according to the calculated discharges for the model. Duration: 37 min.

- Experiment 2: Boom with flexible skirt of 25 cm length weighted by a 21 g iron rod creating an initial underflow opening of a=1.3 cm. Flow regime according to the calculated discharges for the model. Duration: 30 min.
- Experiment 3: Boom with flexible skirt of 25 cm length weighted by a 27 g iron rod creating no initial underflow opening (a=0 cm). Flow regime according to the calculated discharges for the model with doubling of flow after 60 min. Duration: 69 min.
- Experiment 4: Boom with flexible skirt of 23 cm length weighted by a 27 g iron rod creating an initial underflow opening of a=2.1 cm. Flow regime according to the calculated discharges for the model with doubling of flow after 60 min. Duration: 69 min.

3.5 Results

3.5.1 Results Experiment 1

The visual observations of the first experiment involving a baffle fitted upstream of the barrage are shown in a selection of photographs in Figure 3-10. The elapsed time of the experiment is given in minutes and seconds for each observation.

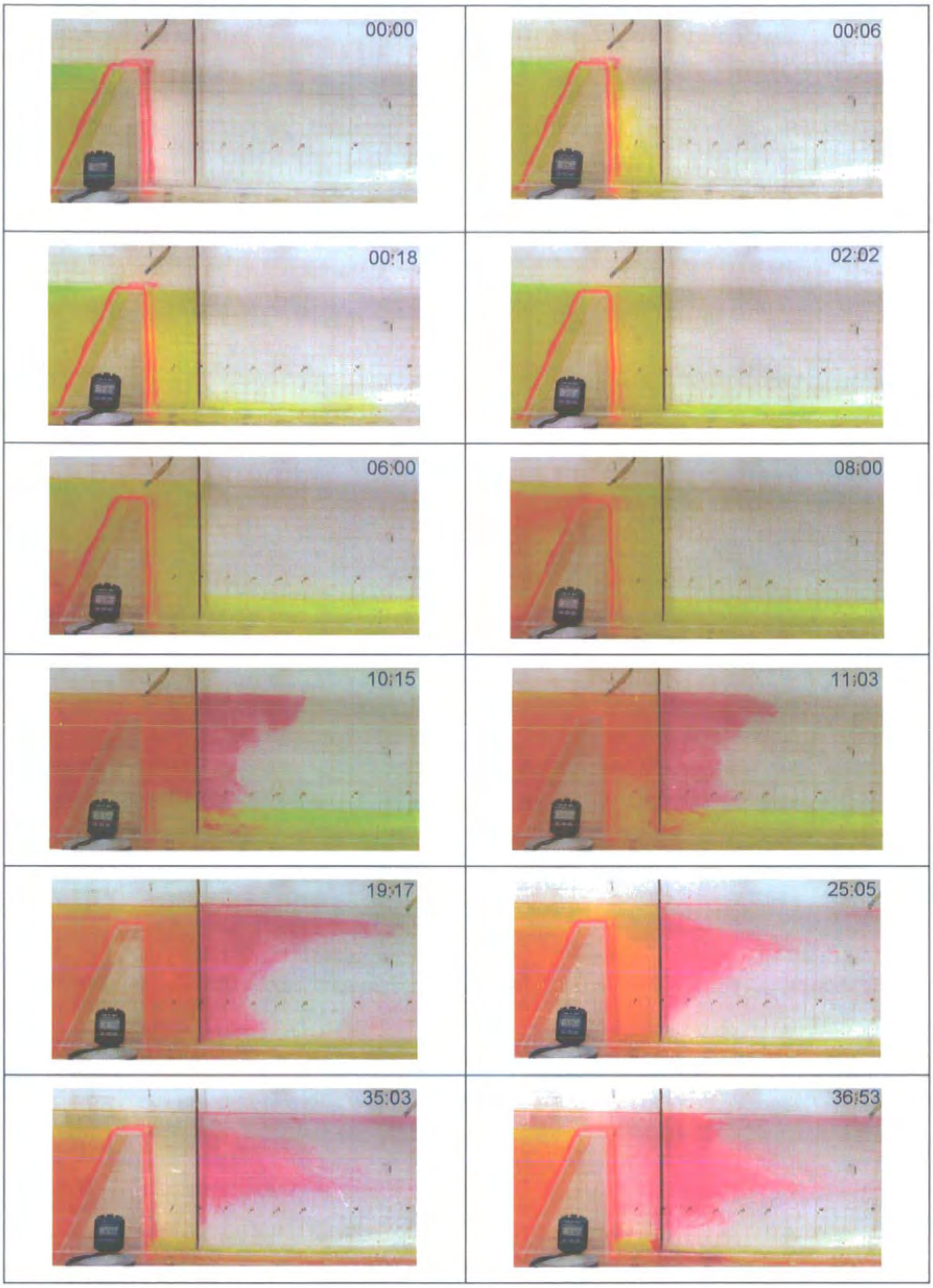

Figure 3-10 Flume experiment No. 1: Baffle, yellow dye indicates salt-water, red dye is injected to visualize hydrodynamic patterns within layers and at interfaces, first injection point is downstream of barrage in the lower layer (06:00 min.), second injection point is upstream of the baffle throughout the water column (10:15 min.)

As indicated in Figure 3-10 (00:18) the saltwater overtopped the barrage in a turbulent jet and plunged into the mixing zone of turbulent eddies downstream of the baffle. After an initial build up of hydrostatic energy within the mixing zone, saline water propagated upstream below the stagnant freshwater layer at the height of the baffle opening. With a velocity of approximately $U_{influx} \approx 21.71$ mm/s, the jet quickly reached the upstream end of the flume ($\Delta t=2$ min.) and the FSI started to rise. As the interface moved up a thinner top layer could be observed possibly resulting from the initial turbulence created by the propagating salt wedge. Maximum salt intrusion was reached after 10 min with a FSI height of z=8.3 cm at which point the freshwater flow was introduced while saltwater was still overtopping the barrage. The red rhodamine dye movement after 11:03 min clearly shows a reversal of the flow conditions below the baffle creating expulsion of the upstream saltwater layer. With the build up of hydrostatic pressure from the freshwater body, the salt layer was expelled from the upstream compartment. The subsequent increase in velocities at the baffle opening led to a depression of the interface just upstream of the baffle, which eventually resulted in freshwater expulsion, also promoted by the declining interface after 19:17 min. The behaviour of the rhodamine dye in the freshwater layer generally indicated stagnant conditions throughout the experiment with local turbulence at the baffle opening after freshwater expulsion started. The experiment was stopped after 37:00 min. when freshwater expulsion became predominant, well documented by the colours in the mixing zone from t>25 min. onwards, due to the minimum salt layer height of z=1 cm

3.5.2 Results Experiment 2

In experiment number 2, the fixed baffle was replaced by the flexible boom/skirt assembly and the results are shown in Figure 3-11.

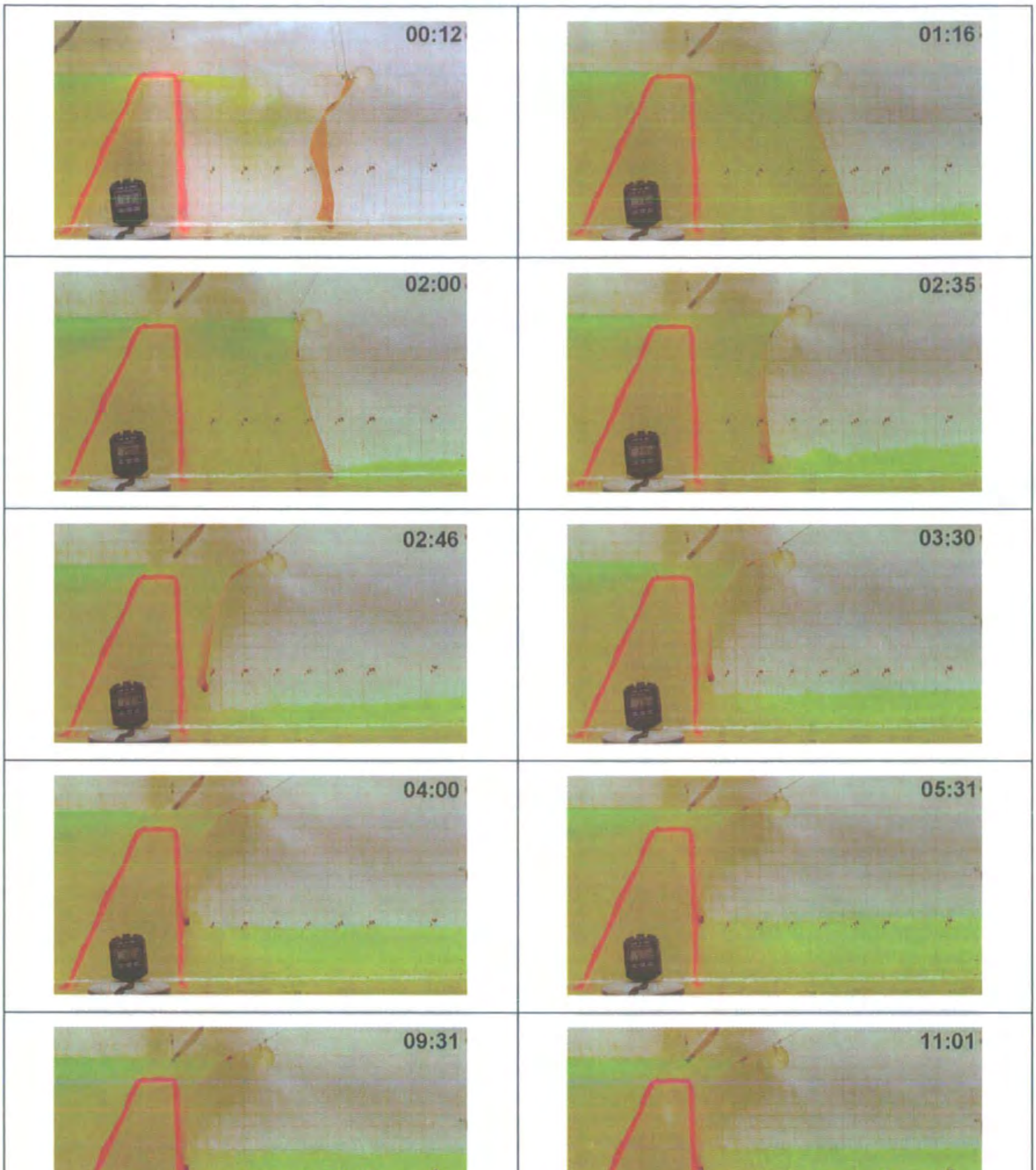

Figure 3-11 Flume experiment No. 2: Boom/Skirt at l=25 cm, 21 g weight, yellow dye indicates salt-water

At the start of the experiment, the skirt hung loosely in the flume due to the equal pressure distribution on both sides and its light weight of 21 g (Figure 3-11– 00:12 min.). The initial baffle opening of a≈1.3 cm allowed the propagation of a turbulent layer of saltwater underneath the upstream freshwater layer. When the overtopping saltwater in the mixing zone built up after 02:00 min., the skirt billowed out due to the hydrostatic loading imbalance. The rising surface level increased the baffle opening which resulted in more saline intrusion. The denser salt water lifted up the freshwater body which created a higher hydrostatic load on the upstream side, forcing the skirt to billow out in the opposite direction after 02:35 min. With the lift of the skirt ongoing, more saltwater shifted upstream during the next seconds bringing more hydrostatic load to the upstream side, which in return provided even more lift for the skirt. The physical boundaries for balancing the pressures were already reached after 04:00 min., when the skirt touched the barrage, thus closing the gap for flushing the system. Although, the experiment was practically over at this point the flow schedule was carried out according to the agreed regime. Salt water intrusion reached its maximum after 15:00 min. with an FSI at z=15.5 cm. The freshwater flow introduced at t=10:00 min. could not induce an expulsion of the salt layer since the opening was blocked, so that no movement of the interface was observed after the salt water was stopped. Instead, a side flow of freshwater between flume wall and skirt developed as identified by the top freshwater layer downstream of the barrage after 30:02 min.

3.5.3 Results Experiment 3

In response to the skirt behaviour of the preceding experiment, the skirt weight was increased to 27 g while the flows were held constant according to the agreed regime. A selection of the visual observations made during experiment 3 is documented in Figure 3-12.

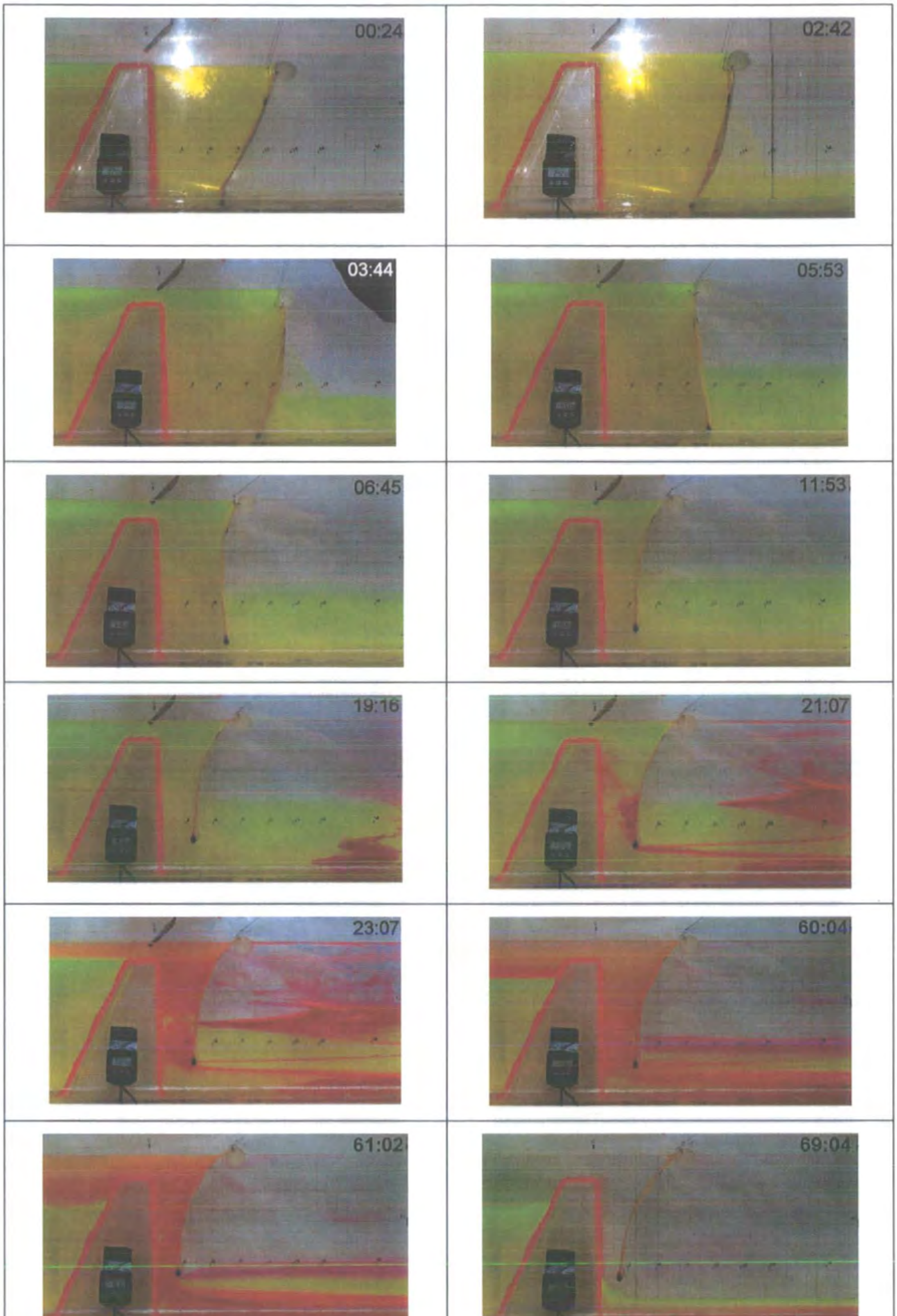

Figure 3-12 Flume experiment No. 3: Boom/Skirt at l=25 cm, 27 g weight, yellow dye indicates salt-water, red dye is injected to visualize hydrodynamic patterns within layers and at interfaces, first injection point is upstream into the lower salt-layer (19:16 min.), second injection point is upstream around FSI (21:07 min.)

Similar to the previous experiments, the jet of salt water overtopped the barrage and quickly filled up the mixing zone upstream (Figure 3-12 – 00:24). Due to the lack of an initial underflow opening, the salt water built up the hydrostatic pressure forcing the skirt to billow out in the upstream direction after 02:42 min., which was further aggravated by the greater weight of the skirt, preventing it from lifting off the flume bottom. Before the skirt lifted off the bottom, small amounts of salt water, forced by the hydrostatic pressure difference, escaped upstream through the gap between the flume wall and the skirt. The turbulent manner in which these considerably small amounts travelled upstream, created a relatively thick interlayer (z=10 cm) of low density (ρ_{int}=1005-1007 kg/m^3) after 03:44 min. When the skirt finally lifted off the flume bottom, after 5:53 min., as a result of the rising surface level, the hydrostatic load equalised and led to a jet of saltwater quickly propagating upstream which introduced considerable amounts of turbulence. The pressure equalisation is illustrated by the behaviour of the skirt swinging from one side to the other after 5:53 min. in order to adjust to the new hydrostatic conditions. The formation of the interlayer made the visual distinction between freshwater and saltwater difficult, while a second, more pronounced, interface developed below the interlayer after the saltwater was stopped (t>15 min.). The injection of rhodamine dye across the whole water column after 19:00 min. indicated the presence of an additional interface, at the height of the skirt opening, along which the downstream velocities were higher than the ambient. The dye movement in the visually distinctive freshwater layer on top, showed the reverse problem of side streams between the skirt and flume wall, which create a slow moving layer along the interface to the interlayer. The freshwater and the interlayer, however, showed more quiescent conditions than the fast moving top of the high density salt layer. A slow reduction of the FSI height from z=16.5 cm at 16:20 min. was observed after 41:21 min. with z=12.5 cm and after 60:04 min. with z=10 cm. After 61:00 min., it was decided to simulate flood conditions by doubling the amount of freshwater flow, which led to an increase in the skirt opening from a=8.3 cm to a=10 cm and also resulted in expulsion of freshwater below the skirt. Nonetheless, the increased velocity shear of the freshwater flow created rapid expulsion of the saline layer with a loss in thickness of Δz=3.4 cm in only 8 min.

3.5.4 Results Experiment 4

A reduced skirt length of 23 cm and a 27 g weight were applied in the last experiment as shown in Figure 3-13.

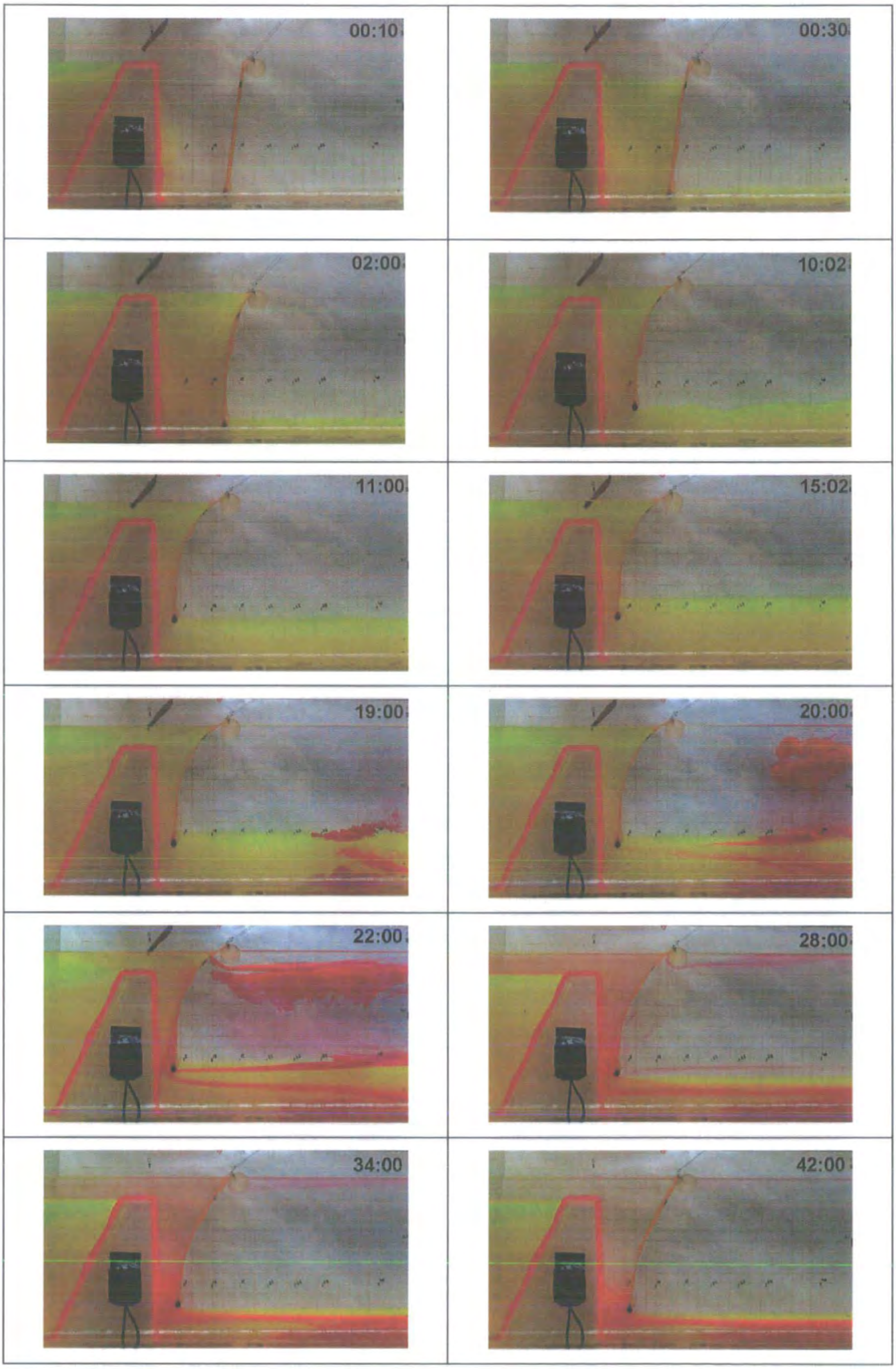

Figure 3-13 Flume experiment No. 4: Boom/Skirt at l=23 cm, 27 g weight, yellow dye indicates salt-water, red dye is injected to visualize hydrodynamic patterns within layers and at interfaces, first injection points are upstream into the lower salt-layer and at the FSI (19:00 min.), second injection point is upstream into the freshwater layer (20:00 min.)

The reduced skirt length provided an initial underflow opening of a=2.1 cm enabling salt water propagation right from the start of the experiment. This resulted in the development of a thinner saltwater layer (z=7.5 cm at 10:00 min.) with a sharper interface compared to the pervious experiment. The extra hydrodynamic load of the freshwater flow after 10:00 min. forced the skirt to lift which decreased the volume in the mixing zone and subsequently displaced this volume upstream. At this point, considerable wave movement was observed at the interface and its height quickly rose to a maximum of z=13.3 cm after 15:00 min. when the saltwater was stopped. Slow expulsion of the salt-layer started, which again was hindered by the relatively small opening between barrage and skirt (19:00 min.). The rhodamine movement after 20:00 min. revealed the main velocity distribution with the impoundment. The highest velocity existed just below the visible interface, causing shear on the saltwater layer and entrainment towards the skirt opening. Although considerable movement was recorded at the interface, the centre of the freshwater layer remained stagnant, while the top moved along the side of the skirt to equilibrate the remaining hydrodynamic load (22:00 min.). Since velocity shear was the dominant mechanism for expulsion, the interface only dropped slowly to a height of z=8.3 cm after 28:00 min. The freshwater flow was doubled after 30:00 min., increasing the shear on the FSI. However, the subsequent falling of the interface led to a decrease in the upstream hydrostatic load, which in turn lowered the lift on the skirt and widened the gap between the barrage and the skirt (42:00 min.).

3.6 Discussion

The laboratory experiments conducted, using a fixed and flexible baffle arrangement upstream of an estuarial barrage, indicated the general feasibility of this concept in restricting saline influx and supporting rapid expulsion of saline waters. During all the experiments, the overtopping turbulent saline influx jet was initially confined to the mixing zone from where it propagated upstream in relation to the underflow opening a. Once the upstream end of the flume was reached, the salt layer acquired a uniform depth and upward displacement of the freshwater layer started. Given the conditions in experiment 1 and 4, with a baffle opening of a=2.1 cm, the constriction generated a planar jet travelling underneath the freshwater layer at U_{influx}=21.71 mm/s which reached the end of the flume after t≈2:00 min. In considering the speed of the propagating wedge, it has to be assumed that the initial phase of influx generates dilution between the layers as the entry momentum dissipates. As a result, the formation of a diluted interlayer was observed during the first minutes of all the experiments, which was gradually strengthened by the continuous saline influx. Since the baffle opening dictated the velocity and nature of the propagating jet, the interface properties of experiment 2 and 3 were different to experiment 1 and 4. The initial underflow restriction in experiment 3 of a=0 cm created a turbulent salt plume from which a thick interlayer evolved. In experiment 2, the initial baffle opening of a=1.3 cm might have been sustainable, but the following widening due to the lift of the curtain displaced a large amount of saltwater upstream, causing substantial mixing. Subsequently, the interfacial gradient at t=15 min. was less pronounced for experiment 2 and 3 as documented by the density profiles for the upstream position l=0.25 m shown in Figure 3-14.

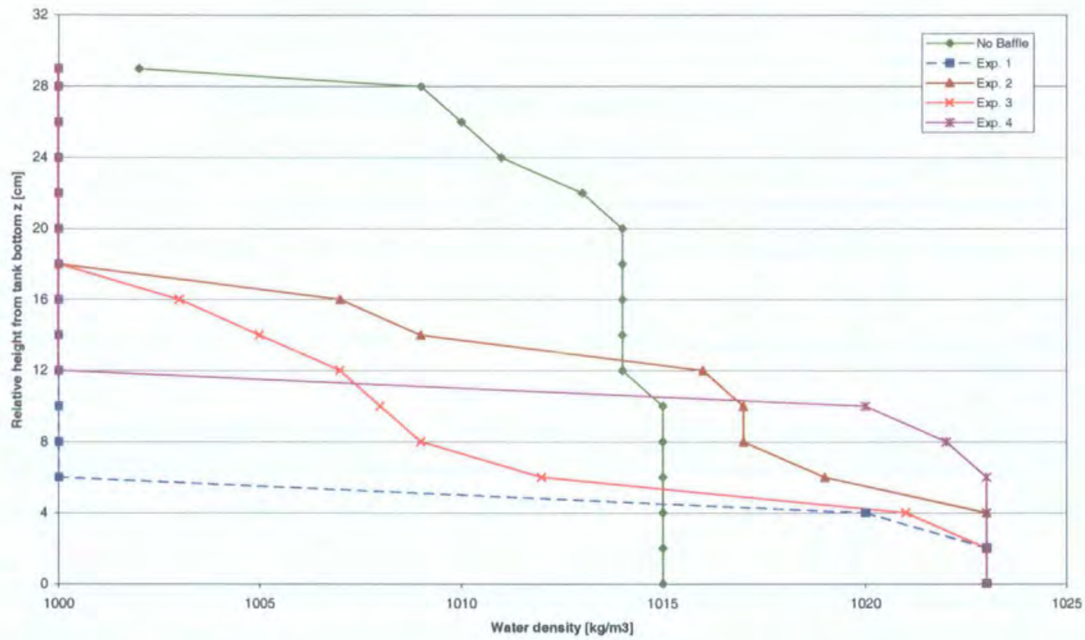

Figure 3-14 Density profiles for flume experiments at t=15 min., 25 cm u/s of the barrage

However, when compared to the saline influx studies conducted by Whyte (2003) without a baffle/skirt installed, all baffled systems provided less upstream accumulation of saline waters. The vertical density gradient of the same experiment without the baffle (Figure 3-14) indicated a high degree of mixing between the competing water bodies after 15 min. By contrast, the baffled systems, in particular those in experiment 1 and 4, developed a pronounced interface with a sharp salinity gradient at z=6 cm and z=12 cm respectively. As a result of the sharp gradient, these systems offered an increased stability of the interface which buffered against shear forces from movement within the layers (Abraham et al., 1979; Shuy et al., 1998). The layer velocities (U) in the three-layered water column in experiment 3 were calculated from the rhodamine dye movement (Table 3-1).

Time (min:sec)	Fresh water layer				Interlayer				Salt water layer			
	h_{Fresh} (m)	U_{Fresh} (m/s)	Re_{Fresh}	$F_{i\,Fresh}$	$h_{Interlayer}$ (m)	$U_{Interlayer}$ (m/s)	$Re_{Interlayer}$	$F_{i\,Interlayer}$	h_{Salt} (m)	U_{Salt} (m/s)	Re_{Salt}	$F_{i\,Salt}$
19:55	0.16	-	-	-	0.08	0.00	0.00	0.00	0.06	0.30	159.49	0.03
20:13	0.16	0.07	91.95	0.01	0.08	0.00	0.00	0.00	0.06	0.11	57.52	0.01
20:33	0.16	0.09	126.32	0.01	0.08	0.00	0.00	0.00	0.06	0.45	242.12	0.05
20:44	0.16	0.07	102.07	0.01	0.08	0.00	0.00	0.00	0.06	-	-	-

Table 3-1 Layer properties 0.25 m u/s of the barrage for experiment 3

All layers upstream of the baffle/skirt arrangement had Reynolds numbers below 2000, which suggests laminar flow conditions (Table 3-1). The observations support this assumption since the dye movement did not indicate any turbulence within the layers. The densimetric Froude numbers (Table 3-1) for the layers were calculated according to Equation 3-5. As expected from the low Reynolds numbers, the conditions were sub-critical with a maximum Froude number of $F_{i\ Salt}=0.05$ for the salt layer. The low Froude numbers resulted from the fact that once the hydrostatic equilibrium is reached in the upper impoundment, the freshwater layer remains relatively stagnant on top of the salt water. The process of positive displacement is then initiated by the influx of freshwater, increasing hydrostatic load at the upstream side of the baffle. Expulsion of the salt layer at a velocity U_{salt} depends on the hydrostatic difference and the baffle opening a. As shown in the density profiles (Figure 3-14), an 'unbaffled' impoundment promotes reversed hydraulic conditions with a thin moving fresh water layer on top of a strong stagnant salt layer. Thus, the saline expulsion due to hydrostatic displacement is almost negligible. Although the expulsion of the salt layer could be given in terms of its downstream velocity, it is common to display the movement of the FSI over the time, as defined by W_e (Eq.3.8), which is also known as the salt flux rate. The salt flux rates for the experiments 1 to 4 and, for comparison, for one 'unbaffled' experiment, are given in Figure 3-15.

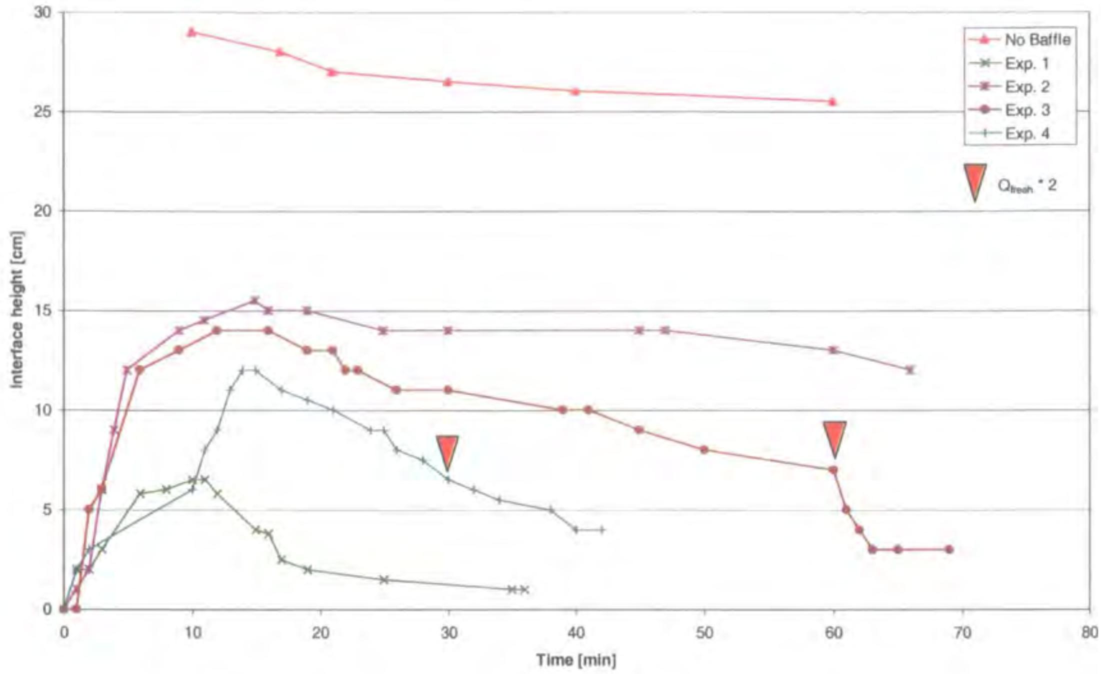

Figure 3-15 FSI height during flume tank experiments

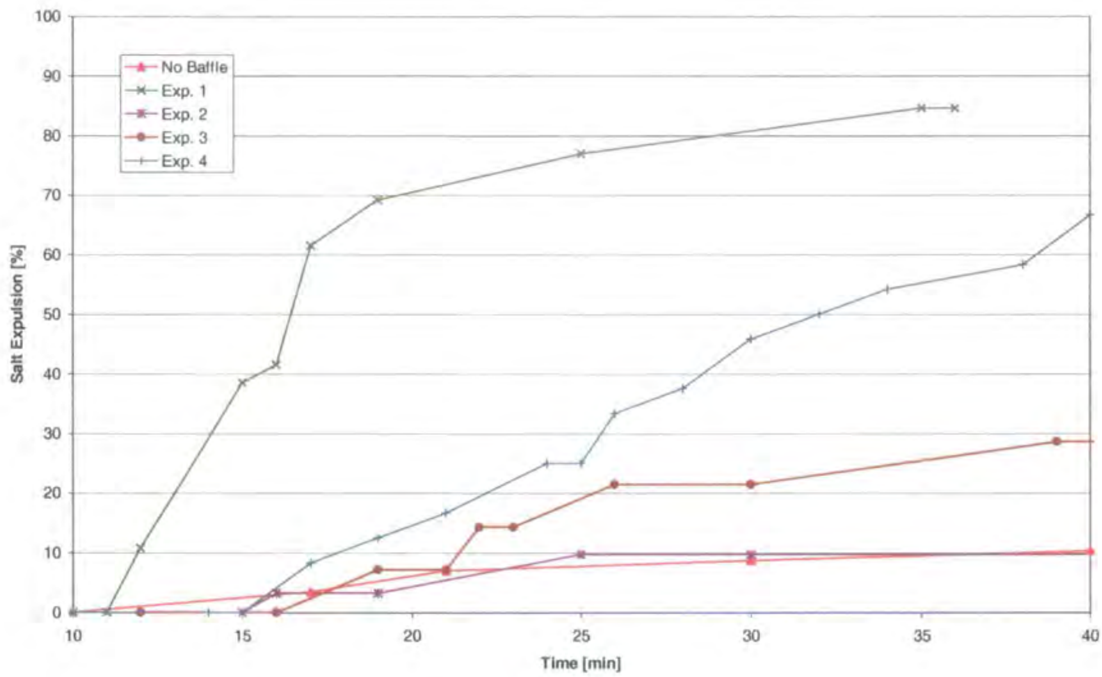

Figure 3-16 Expulsion rates in % for flume tank experiments

The flux rates (Figure 3-15) of the baffled impoundments, show a decrease over time, while most of the saline displacement is attained within the first 30 min. after the freshwater flow has been started. When compared to the 'unbaffled' system, expulsion rates of up to 75 % (Figure 3-16) higher were achieved with the baffles during the first 30 min. of freshwater flow. Nevertheless, during none of the experiments a complete removal of the saline layer was achieved within the given time. This is a result of the declining interface which eventually reaches the height of the baffle opening and thereby changes the expulsion mechanism from hydrostatic to hydrodynamic (Figure 3-17).

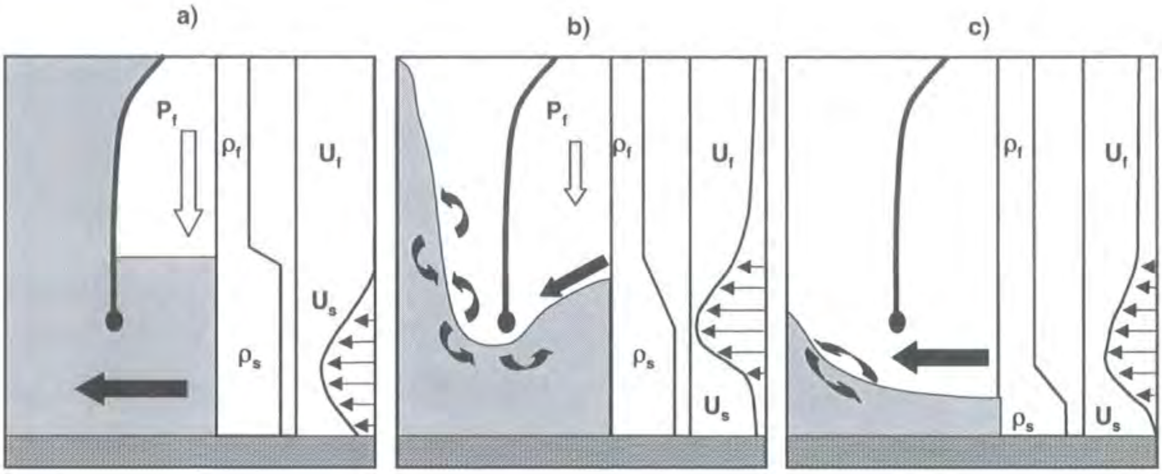

Figure 3-17 Interface at the baffle opening

The schematic in Figure 3-17 points out the main observed patterns regarding interface height and expulsion mechanism. If the interface drops below the baffle height, freshwater escapes the system due to the balancing of hydrostatic loads. The velocity shear at the interface is enhanced in the vicinity of the underflow opening and causes higher entrainment rates locally (Figure 3-17-b)). Furthermore, the loss of freshwater is accompanied by a reduction in the hydrostatic load (P_f) on the salt layer upstream. Eventually, the dominant expulsion mechanism changes from hydrostatic displacement to hydrodynamic entrainment (Figure 3-17-c)). As stated in section 3.3.4, the rate of entrainment depends on the dominant mixing mechanism at the interface which is defined by the Richardson number. The Richardson numbers for an experimental set-up similar to that in Experiment 1 were calculated by Whyte (2003) using an ultrasonic velocity profile monitor at a position 25 cm upstream of the barrage. The values ranged from 12 to 54 indicating stability with no turbulent mixing processes across the interface. The observations in experiment 1 and 4 comply with these findings, since turbulence was absent during the entrainment phase which is well documented by the laminar dye movement along the interface in Figure 3-10 (after 36:53 min.) and Figure 3-13 (after 26:00 min.). Therefore, entrainment was possibly induced by small scale mixing processes such as Holmboe waves (section 3.3.4) as a result of velocity shear from the moving freshwater layer. It is shown in Figure 3-15, that the rate of expulsion due to entrainment is very small compared to expulsion via positive displacement. In addition, entrainment involves the loss of freshwater from the impoundment which in many cases should be avoided to ensure water for extraction or navigation purposes. A priority for the management of a baffled system will therefore be the appropriate layout of the underflow opening a. The boom/skirt design includes the advantage of flexible adjustment to hydrodynamic and hydrostatic conditions via the skirt weight. However, experiments 1 to 4 also demonstrated that a weight change also needs to be accompanied by a skirt length adjustment to ensure sufficient influx opening (a) during lower surface levels. A more appropriate design could thus be a roll up skirt (Figure 3-18) adjustable to varying flow and surface level conditions.

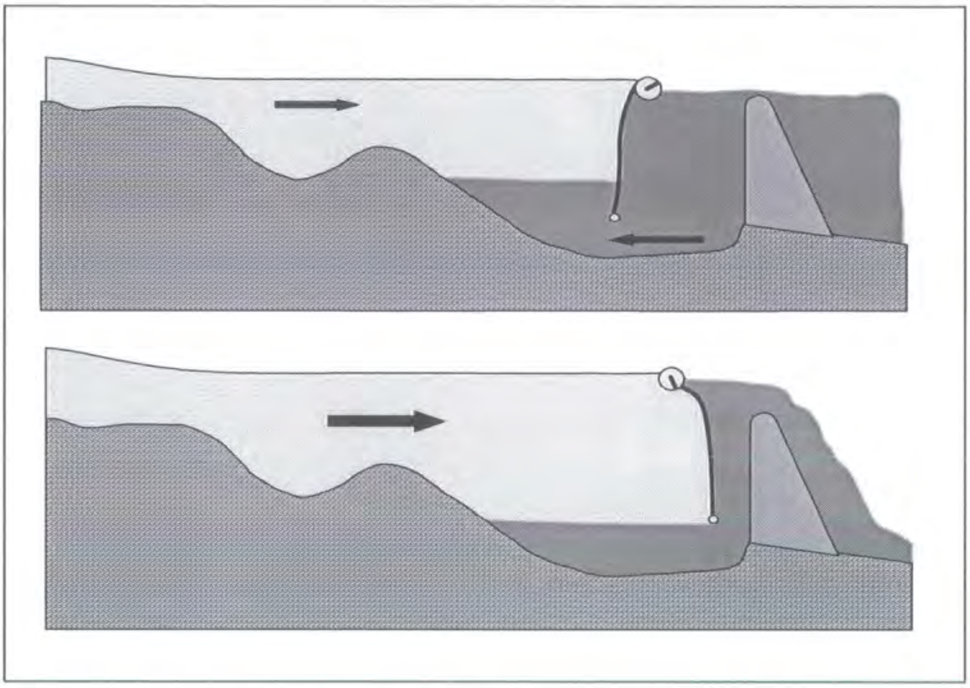

Figure 3-18 Boom/Skirt system under different hydrostatic and hydrodynamic conditions

In addition, this design allows for the possibility to remove the curtain completely from the flow during river spates when saline intrusion and stratification is less likely to occur.

One of the main experimental disadvantages of the boom/skirt design, as compared to the fixed baffle, was the development of side flows between the curtain and flume wall which significantly reduced hydrostatic loads upstream. Considering the prototype channel span of 105 m (Chapter 2), these side flows would be considerably smaller in reality, resulting in an increased efficiency of the system during seawater expulsion. This, however, also creates a considerable load on the construction anchorage and on the flexible curtains in particular. One option to reduce these loads, and also to remove a major criticism of the boom/skirt concept, is the introduction of orifices (Figure 3-19) into the curtains. Preferably located near the fish-pass, they would enable the free movement of fish, whilst ensuring a reduction of freshwater loads at the same time.

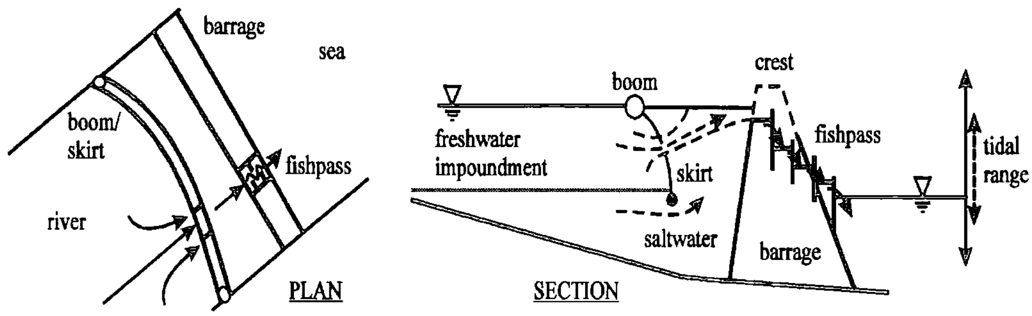

Figure 3-19 Boom/Skirt system with orifices (from Burrows and Ali, 2001)

During the experiments, the expansion of the influx jet was curtailed by the upstream end of the flume and more turbulence was added to the salt layer. In the prototype, the initial wedge propagation would be curtailed by the bed morphology which, i.e. for the Tawe, initially restricts the area of the rising interface to a distance of approximately 600 m, as the river bed quickly rises upstream of the barrage. This results in a smaller interface area to be disturbed by turbulence. It could also lead to a containment of the entire stratified water body to the lower impoundment, if the hydrostatic load upstream is sufficient and the system is adjusted to optimum influx reduction.

3.7 Conclusions

The flume tank experiments simulating an impoundment subject to tidal influx confirmed the general feasibility of the concept of utilising an upstream baffle for managing saline stratification. Saline intrusion was successfully reduced through containment of the turbulent saline influx jet to a mixing zone between the baffle and barrage. The entry momentum of the initial plunge dissipated into a planar jet that propagated upstream at the height of the baffle opening, which minimised the dilution with the ambient freshwater. After reaching the upstream boundary, the FSI rose against the hydrostatic forces of the overlying freshwater body induced by further salt intrusion. The salt influx phase was then characterized by the development of a stable interface towards the confined freshwater with only limited entrainment due to small scale mixing processes. Once hydrostatic equilibrium was obtained and freshwater flows started to dominate, positive displacement initiated expulsion of the salt water body. The limit for hydrostatic discharge was reached when the FSI levelled with the baffle opening, which started the outflow of freshwater and introduced velocity shear as the dominant expulsion mechanism. Expulsion rates, during hydrodynamic removal of the salt layer, decreased due to the stability of the interface and limited velocities created in the flume experiments.

Although hydrodynamic and hydrostatic forces will be considerably larger in a prototype environment, the flume tank experiments provided valuable information on the future design criteria required. As the most important variable, the underflow baffle opening has to be carefully designed for mean influx and flow conditions since it:

- determines the nature of saline intrusion by reducing the amount of initial turbulence created by the propagating jet
- reduces the amount of saline dilution by creating a stratified system in which a stable interface is maintained
- limits the upstream propagation of salt water through formation of a uniform FSI height that is curtailed by bed morphology
- determines the dominant expulsion mechanism and thus the rate of saline removal

Since a variation of the skirt weight is rather unlikely post installation, the length of the skirt should be adjustable to account for variable water surface levels and hydrodynamic conditions. However, the forces on the material and the anchorage could be considerably large and emergency measures such as orifices and complete removal from the flow should be incorporated in the design.

4 Flushing

4.1 Introduction

The purpose of this chapter is to investigate the classical remediation approach for impoundments that experience water quality problems - the release of deteriorated water or so called sluicing or flushing. The ability for a selective withdrawal of upstream waters is, mostly due to regulatory requirements, incorporated in every estuarine barrage design in the UK and can be realized via low level sluice gates, penstocks, culverts, hydroelectric power plant inlets, the navigation lock or a combination of all of these depending on the impoundment volume and depth. Controlled water releases from freshwater impoundments have been described by Churchill and Nicholas (1967), who showed that depth specific point withdrawal from a homogeneous fluid affects all depths of the storage volume (Figure 4-1). However, impoundments are rarely homogeneous since the ratio of dammed volume to baseflow conditions often leads to long residence times transforming the river into a lake environment. The stratified nature of impoundments and their hydrodynamic patterns with respect to water quality were first described by Wunderlich and Elder (1967) and Brooks and Koh (1969). They suggested that stratification restricts vertical movement and enhances the horizontal discharge from layers of constant density near the outlet elevation (Figure 4-1). Depending on the height of extraction, this considerably alters the properties of the discharged water and reduces the upstream mixing effects.

For summer conditions, the height of extraction could therefore either result in DO supersaturated, nutrient depleted, warm water or oxygen depleted, nutrient rich, cold water. However, these assumptions are based on observations made in freshwater reservoirs with density stratification occurring as a result of thermal stratification. Within estuarine impoundments, density stratification develops as a result of the interaction between fresh and salt water which creates a considerably larger vertical denisty gradient in the water column. Although estuarine barrage designers included the water release option for years (Wansbeck Barrage, completed 1975), surprisingly limited information is available on its impact in highly stratified impoundments.

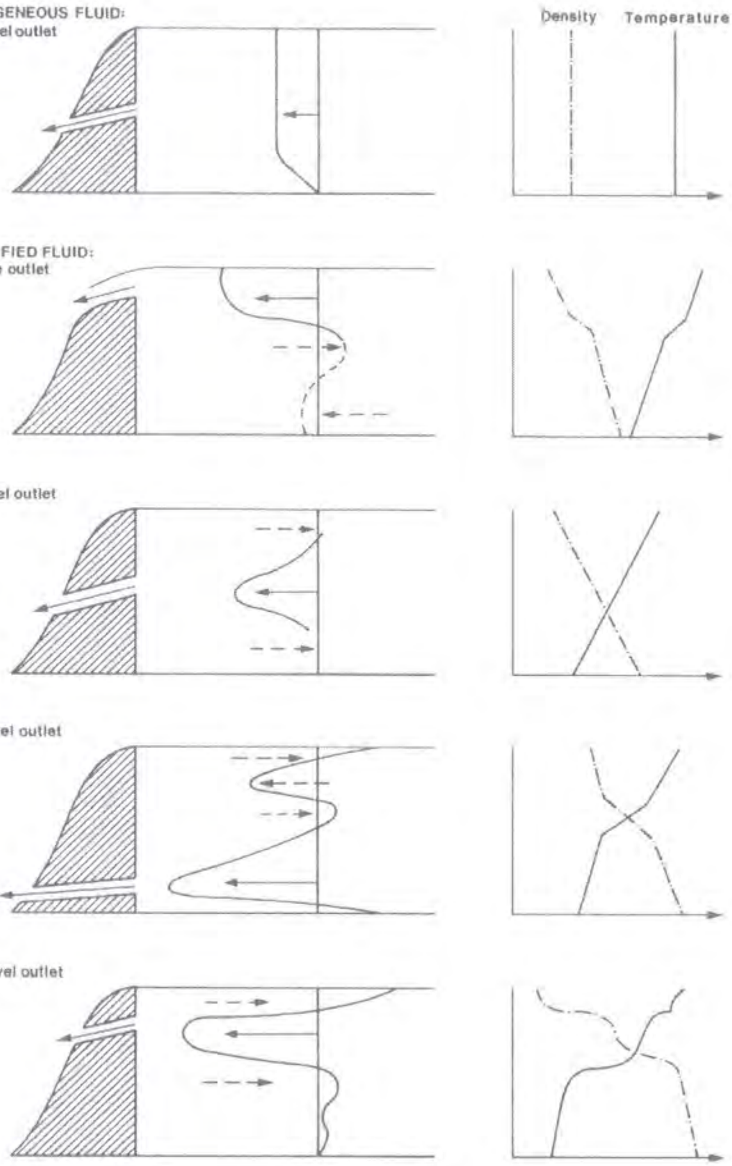

Figure 4-1 Effects of selective withdrawal from various depths of impoundments (Petts, 1984)

This chapter firstly outlines the results of a short-term sluicing survey on the River Tawe, Wales, and secondly describes a flushing survey conducted by Durham University on the River Wansbeck, Northumberland, in order to determine the effectiveness of selective water withdrawal as a remediation method for water quality of impoundments.

4.2 Results of the Tawe sluicing survey

In light of the deteriorating water quality observed soon after impoundment of the River Tawe in 1992, it was decided before the summer season in 1993 to investigate sluicing as a method to reduce volume and extent of the saline wedge. For this, a joint NRA/SCC Water Quality Survey (Rogers and Bryson, 1993) was conducted from the 30/03/93 to the

02/04/93, targeting neap tide conditions with low freshwater inputs (Figure 4-2) while SCC engineers operated one of the shipping lock penstocks. Incorporated in the barrage design are a total of two upstream culverts of 2 m^2 opening, which allow abstraction of water from the barrage base to level the lock. During the main survey of 31/03/93 SCC engineers operated one penstock at maximum allowed discharge (30 % open as defined in Technical Specification by W.S Atkins) between high water and low water (Figure 4-2) while intensive surveying was carried out in the impoundment on a 1.5 hour rolling programme.

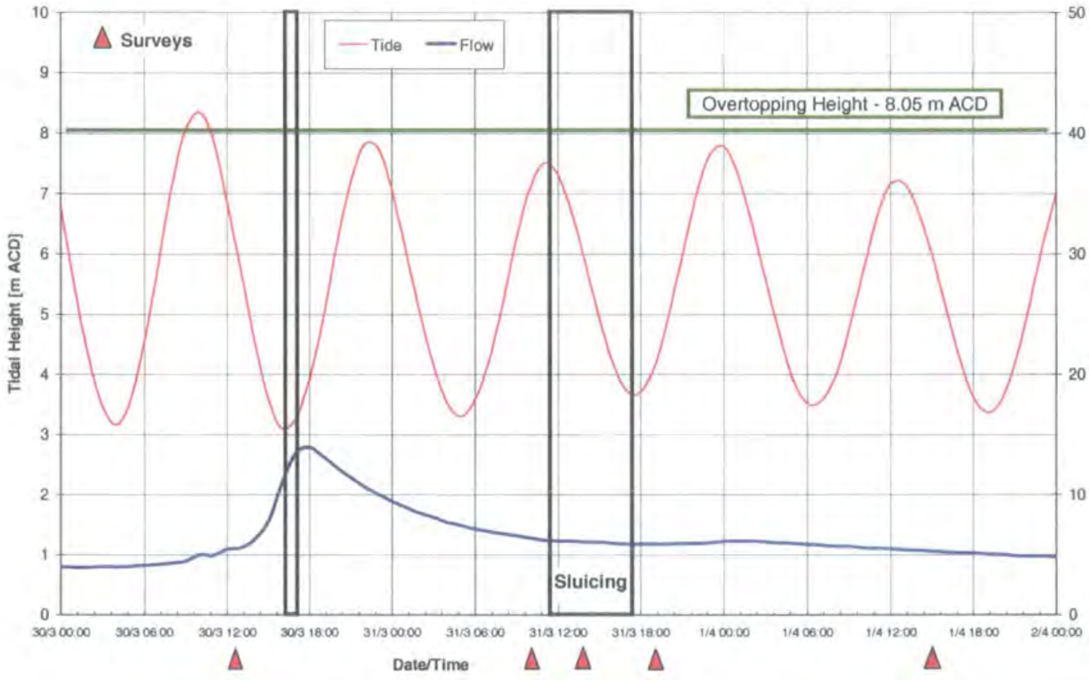

Figure 4-2 Environmental variables during sluicing survey on the Tawe in 1993

The monitored salinity records from the surveys (Figure 4-2) are displayed as contour plots in Figure 4-3. The preliminary survey on the 30/03/93 (Figure 4-3 a)) indicated that the impoundment was stratified with a pronounced halocline at 1.5 to 1.8 m depth. However, the flow spate of 14 m^3/s and a 50 min. penstock operation on the evening prior to the main survey (Figure 4-2) resulted in a slight lowering of the halocline, as indicated in the initial profile taken prior to the sluicing operation on the 31/03 (Figure 4-3 b)).

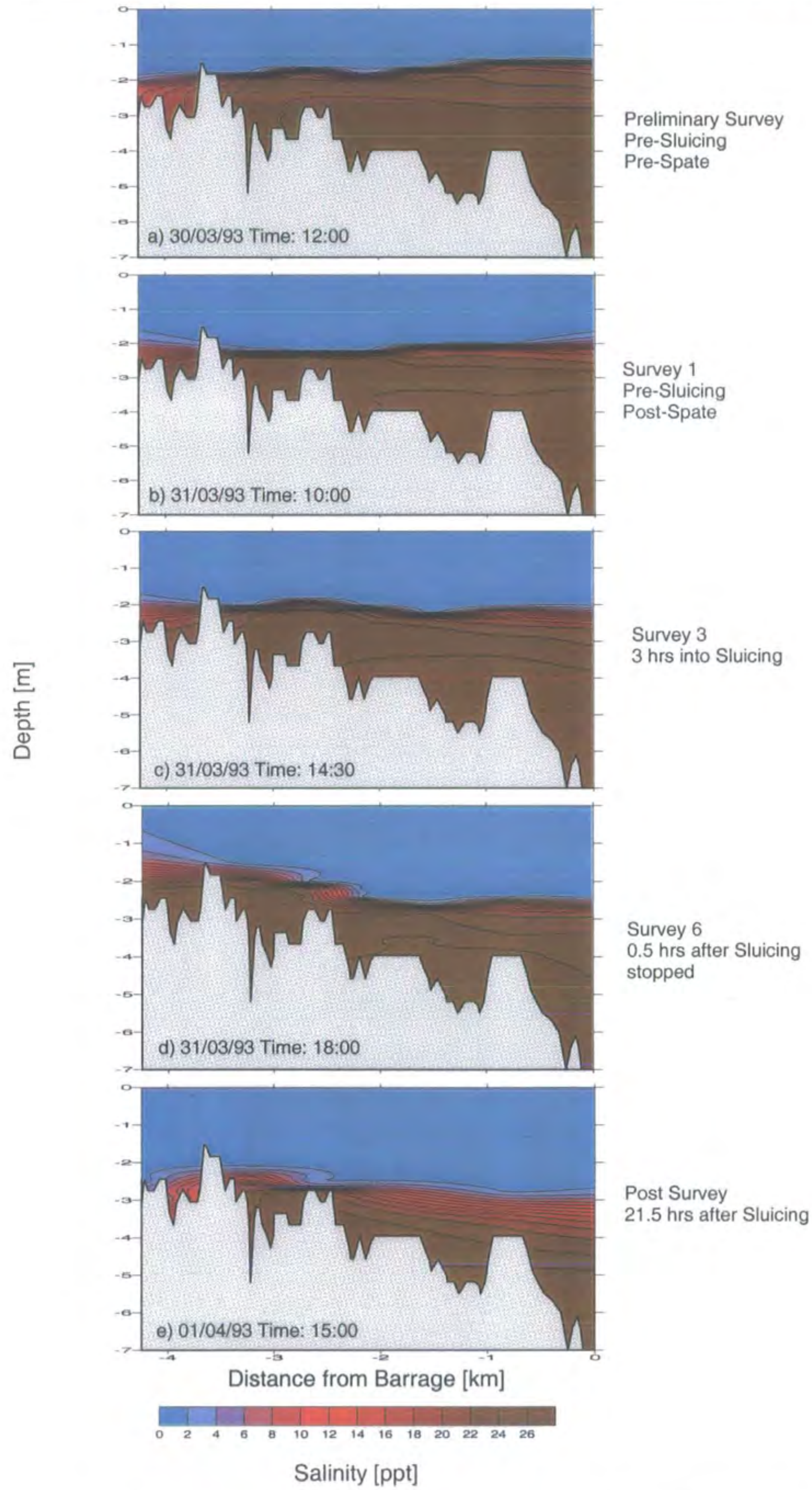

Figure 4-3 Salinity profiles for the sluicing survey on the River Tawe 1993

During the 6 hour penstock operation on the 31/03 the halocline dropped from an initial depth of around 2.0 m to a depth of 2.5 m within the lower impoundment up to a distance of 2.7 km from the barrage (Figure 4-3 d)). Initially, the upper impoundment remained relatively stagnant possibly due to the bed elevation at Whiterock Bridge (2.7 km upstream) and the distance from the barrage which prevented levelling of the halocline. The survey on the 02/04 demonstrated that the upstream level of the halocline was affected with a delay, bringing the halocline depth to around 3.0 m for the whole impoundment. For the shallow upstream site this resulted in a return to freshwater conditions while for other sites the falling halocline lead to trapped saltwater pools in the deep pockets.

The trial was partially successful since the operation reduced the volume and the extent of inland penetration of the saline wedge. However, it was unclear what size of effect the actual sluicing had, since a) the survey was accompanied by a river spate and b) the depression of the halocline depth continued with no discharge from the penstocks during flow recession. Clarity existed merely about the volumes and the origin of discharge since the released water showed a salinity of 24 ppt (Sample 31/03/93 16:30). On these grounds calculations were made for possible sluicing rates (Table 4-1) which had to consider the minimum discharge of 1 m^3/s for the fish pass as required by Section 13(2) Tawe Barrage Act (1986). It was estimated that a maximum of 12 hours operational ability during 2 ebb tides a day was possible.

Reach	**Volume**		**Volume Saline Wedge**		**Time required to discharge (hours, min – max)**		
	Min	Max	Min	Max	95%ile	75%ile	50%ile
Landor – Addis	45,000	90,000	5,000	30,000	3 – 17	0.6 – 4	0.3 – 1.5
Addis – Foxhole	60,000	120,000	6,333	38,000	4 – 21	0.8 – 5	0.3 – 1.9
Foxhole – Newcut	112,500	225,000	53,819	193,750	30 – 109	6 – 23	3 – 10
Newcut – Barrage	180,000	360,000	21,600	64,800	12 – 37	3 – 8	1.1 – 3
Newcut – Barrage (incl. Marina entrance)	510,000	102,000	113,333	340,000	64 – 192	14 - 41	6 – 17
Totals	**907,500**	**1,815,000**	**200,086**	**666,550**	**113 - 376**	**24 - 80**	**10 – 34**
Days (12 hours)					**9.4 – 31.3**	**2 – 6.6**	**0.8 – 2.8**

Table 4-1 Estimated sluicing rates to discharge saline wedge (reproduced from Rogers and Bryson, 1993)

The sluicing volumes were presented in the initial report (Rogers and Bryson, 1993) and were revised later during that year with the help of depth soundings and cross sectional data which lowered the total volume of the impoundment to 807,000 m^3 and the saline wedge volume to 431,000 m^3. On the basis of the new volumes, expulsion of the entire salt wedge volume would still require 10 days of no overtopping and sufficient freshwater flow to maintain fish-pass operation. Water quality conditions in the impoundment were observed to deteriorate within a much smaller time scale (Chapter 2) and, in addition, the tidal frame would not allow for a full expulsion cycle. It was therefore doubted that sluicing alone could effectively maintain acceptable water quality in the impoundment and that other options should be considered. Nevertheless, a routine sluicing operation was implemented in the daily lock operation of the barrage for the year 1994. This included a schedule calculated on the basis of required lock operation times, tidal frame and maximum penstock opening. Unfortunately, 1994 also proved to be a very dry year and the concerns expressed in the report (Rogers and Bryson, 1993) were substantiated, making it one of the worst years in terms of water quality which increased the pressure on SCC to develop alternative approaches in tackling their water quality problems.

4.3 The River Wansbeck flushing survey

4.3.1 Introduction

The River Wansbeck is located in South East Northumberland and drains a catchment area of 287.3 km^2 (Figure 4-4). It flows eastwards from its source 300 m above sea level in the hills and moorlands to the east of Bellingham and Otterburn until it reaches, some 37 km later, the North Sea at Cambois (NZ 318 855).

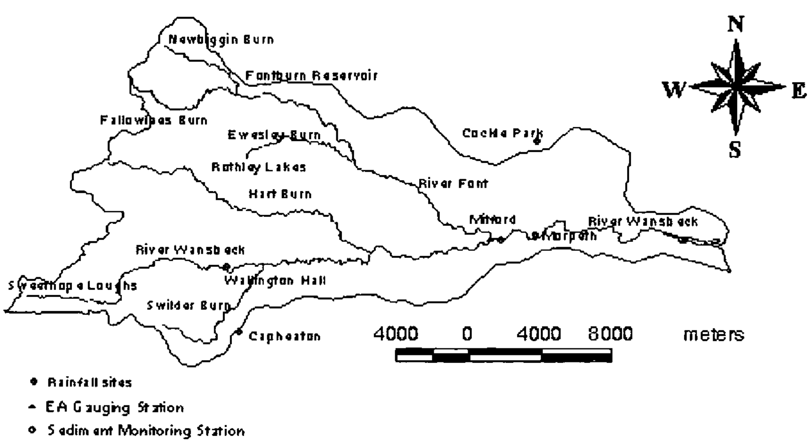

Figure 4-4 River Wansbeck catchment map

On its way, the river passes mainly through pasture and woodland before reaching the population centres at Morpeth, Stakeford/Guide Post and Ashington. It is also in these lower reaches where the main tributary, the River Font, joins the River Wansbeck and where the only EA Gauging Station at Mitford (NZ 175 858) is located. The station accounts for 84.6 % of the catchment and the average daily flow is given as 3.24 m^3/s since recording started in 1968 (National Flow Archive, 2003). The 95 % and 10 % exceedance flows for the River Wansbeck are 0.217 m^3/s (Q95) and 7.092 m^3/s (Q10) respectively (Figure 4-5).

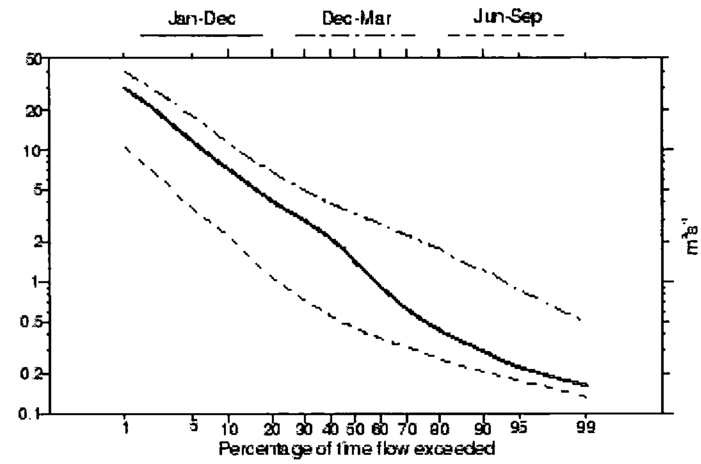

Figure 4-5 Flow duration curve for daily gauged flows on the River Wansbeck since 1968 (National Flow Archive, 2003)

Four treatment works discharge into the Wansbeck below the River Font confluence, with Morpeth having the largest plant (Q_{mean}=0.047 m^3/s). The impounded estuary reaches 4 km inland to the former tidal limit at Sheepwash weir (NZ 256 854). Several deep mines existed in this area and offshore coal crops provided a high proportion of coal dust carried in the sediment at high tide which resulted in an appearance known as 'black desolation' (Curtis and Dawson, 1978) during low tide. The unsightly tidal mudflats, eroded riverbanks and its general use as a tipping place made the lower estuary the focus of 'Operation Eyesore' in 1972. With financial assistance from central government industrial dereliction was to be removed and amenity value returned to the region. Central to the project was the construction of a weir within the estuary to flood the unsightly saltmarshes and create an amenity lake framed by a Riverside Park scheme. With the park scheme already underway, construction of the weir started in January 1974 after the site was elected from a choice of four possible locations, following the recommendations of the consulting engineers Babtie Shaw and Morton, Glasgow. The two main contractors, Harbour & General Works Ltd. (Civil Works) and Newton Chambers Engineering Ltd. (Lock and Sluice Gate), finished their work in May 1975 at a cost of £ 273,000, while the Riverside Park scheme, including bank gabion works, was finished in 1977 at a cost of £ 382,000. The height of the weir crest is fixed at 4.74 m ACD (2.14 m AOD), which was chosen primarily to provide sufficient depth for boats entering the impoundment upstream (1.6 m draught) and, secondly, to allow overtopping of the barrage in order to *"assist in maintaining water quality in the summer months ....by creating a limited but regular turn round of water stored in the lake"* (Curtis and Dawson, 1978).

Figure 4-6 View of the Wansbeck barrage from the south-bank

Height and location of the barrage reduced the tidal frame upstream of the barrage by 80 % allowing only tides >4.74 m ACD, as measured from dock sill height at Blyth, to enter the impoundment. This resulted in a transition away from a well-mixed salt-wedge estuary towards a highly-stratified lagoon-like impoundment, which relies upon major freshwater spates to experience mixing. Periods of 14-20 days without overtopping seawater reduce the oxygen conditions significantly within the lake and anoxia of deep layers occurs regularly during the summer time (Worrall and McIntyre, 1998). In addition, the major sediment build up, which developed over the last 28 years at an average sedimentation rate of 400 mm/year, has left most of the lower impoundment at less than 1 m depth. This created at least 3 new islands within the impoundment in addition to the biggest, Castle Island (Figure 4-7), which is now a recognized nature reserve for birds. However, the progressive shallowing of the lake also increased the entrapment of flood-derived debris, which accumulated in the area downstream of the railway bridge (Figure 4-7), making it unsafe for water users.

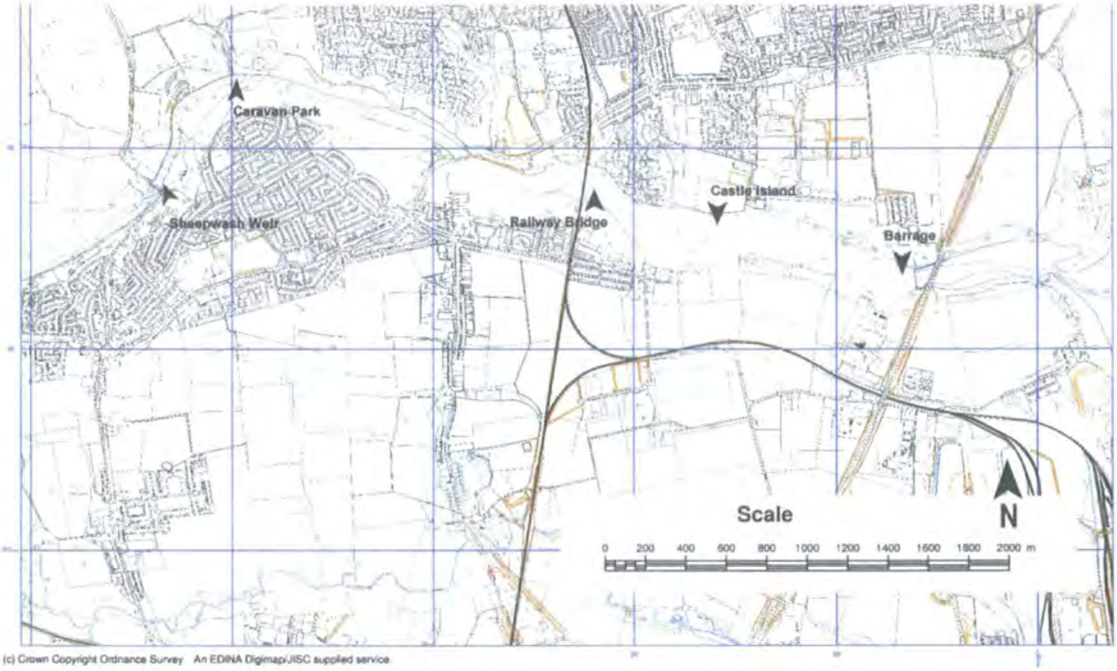

Figure 4-7 Location map Wansbeck impoundment

The main river channel meanders through the impoundment (Figure 4-7-purple line) and is characterized by several deep hollows that resulted from increased flow velocities in narrow stretches (Railway bridge piers) or geological faults (Caravan Park). These deeper parts of the river bed are particularly prone to pooling of waters which, due to stratification, can remain stagnant for complete non-overtopping cycles depending on the river flow. It is intended, to increase vertical shear stress in these parts via flushing of the impoundment and thus introduce mixing, or completely flush out these deteriorated pools.

4.3.2 Sampling locations

The river section studied extends 4 km upstream from the barrage to the former tidal limit at Sheepwash weir, which is considered as the section of river that has undergone impoundment.

Standard practice was applied in choosing sampling sites (ISO, 1990; Bartram and Balance, 1996), which were established approximately equidistantly along the section studied (Figure 4-7), at the deepest point of the river cross-section. Stations were chosen at clearly identifiable points to ensure repeatability of sampling at the same points. The tidal limit site was chosen with the objective of identifying the baseline conditions of water quality as it entered the system under study.

Site name	Site abbreviation	Max. Depth in m	Grid reference
Sheepwash Weir	SW	0.50	NZ 2565 8587
Caravan Park	CP	4.88	NZ 2610 8635
Railway Bridge	RB	1.83	NZ 2775 8585
Castle Island	CI	0.50	NZ 2835 8561
Wansbeck Barrage	B	3.66	NZ 2935 8536

Table 4-2 Sample sites on the River Wansbeck

As a check on random errors introduced in either sampling or analysis, duplicate samples were taken at all depths (shallow, mid and deep) during sampling.

4.3.3 Monitoring procedure

Monitoring was carried out as close as possible to midday during all of the surveys to avoid diurnal variation in water quality, e.g. changes in DO due to algal respiration with time of day (Lingeman et al., 1975), confounding results. The length of time taken for each date's run was on average circa 3 hours. The sequence of data collection was from the barrage upstream to the tidal limit.

All work was carried out from the research vessel (RV-Figure 4-8) except at site SW, where at low flows monitoring was carried out by wading. Care was taken to carry out field analysis and water sampling on the upstream side of the RV to minimise the risk of disturbance. At each site the time, water depth and the water quality parameters measured were recorded.

Figure 4-8 Research vessel used during the survey

Based on knowledge of limnological processes involving thermal or chemical stratification (Strøm, 1955; Wunderlich, 1971), and preliminary studies on the impoundment (Worrall and McIntyre, 1998; Worrall et al., 1998), it was decided to record at three depths at each site. These were: At the surface, just (circa 10cm) above the sediment water interface, and at the mid-depth between these points. The maximum depth at each site was measured using an echo sounder on the RV. At shallow sites, where it had been ensured that the water was well mixed to be homogenous (by taking DO, temperature and conductivity measurements), only surface recordings were taken. This was always the case at sites SW and CI, where weirs or riffles provided turbulence to mix the water body. On several occasions profiles of the parameters were taken at a higher resolution throughout the water column (0.3 m) to help define vertical variations in water quality on a finer scale.

4.3.4 Environmental variables

River discharges of the River Wansbeck are recorded by the EA Gauging Station at Mitford (NZ 175 858) 150 m downstream of the confluence with the River Font. Records for the survey period were available as 15 min. readings in m^3/s and represented summer baseflow conditions, with a major spate occurring around midday of 16/06/01 (Figure 4-9).

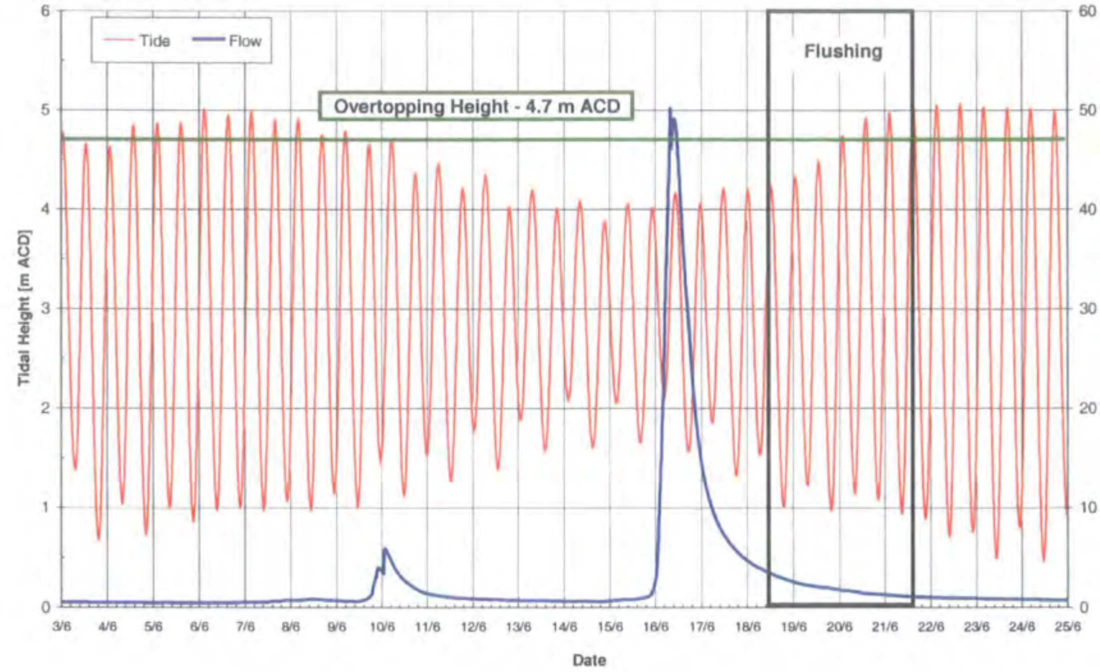

Figure 4-9 River flow and tidal regime during flushing survey

The river discharge parameter mainly referred to in this section is the measurement at the time of sampling, defined as Q_{sample}. Since the freshwater of the impoundment often represents flow conditions prior to the time of sampling, the maximum and mean discharges over the preceding 24 hour period, Q_{max24} and Q_{mean24} respectively, were also included in the statistical modelling.

The tidal regime for the Wansbeck estuary is recorded at the Sea Level Gauging Station North Shields (NZ 359 682), which is operated by the UK Hydrographic Office. Quality controlled 15 min. level observations of this station were obtained from the British Oceanographic Data Centre (BODC) and plotted against the flow records (Figure 4-9). The time and height differences for HW and LW applicable for the closest secondary port at the mouth of the River Blyth are negligible (< 10 min., 0.00 m HW, UK Hydrographic Office, 2001) within the measured time frame of the survey period, so that the observed levels from North Shields were directly applied in this section. The barrage height of 4.74 m ACD is also indicated (Figure 4-9) to illustrate overtopping and non-overtopping conditions in the impoundment.

4.3.5 Flushing

The Wansbeck barrage design includes a navigation lock and a bottom hinged leaf gate (Appendix C) for emptying purposes. The gate was introduced as an extra measure to, along with the lock, provide a maximum discharge of 87 % of the initial impoundment volume between two high tides. Unfortunately, the gate designers did not account for the hydraulic forces on both sides of the weir at high water which made it uncontrollable and prone to destruction. The gate has not been operational since 1990 (according to Wansbeck District Council) and was shut off with stoplocks (Figure 4-14). Subsequently, only the navigation lock was utilized to perform the flushing process. The lock consists of two pairs of manually operated sector gates with gear boxes which can be operated via hand-wheels or mechanical actuators. Years of torque led to the degradation of the gear boxes, so that only the two upstream gates were operational during the survey while the downstream ones were kept open at all times. Due to the lock condition, operation was limited to water release purposes only.

The flushing procedure is initiated by Wansbeck District Council personnel on a regular basis, approximately 7 times per year, as an operational requirement to maintain the channel downstream of the barrage. This activity, which is agreed in advance with the EA, is believed to create additional tidal scour and thereby reduce sedimentation occurring at the mouth of the estuary. However, the energy dissipating measures that are required downstream of the barrage to prevent detrimental effects to the barrage itself and the Spine Road bridge reduce the additional scour energy substantially. As a result, a sand bar, induced by long shore drift processes, is permanently blocking the entrance to the Wansbeck estuary these days. The campaign of the Wansbeck and North Seaton boat clubs for artificial management (dredging or groyne system) of a navigable entrance to the estuary is still ongoing.

4.3.6 Numerical analysis of data

In order to explore the difference between monitored DO and Conductivity readings of sampling days with and without flushing in operation, an unbalanced general linear model (GLM) ANOVA (Section 2.3) was applied to the data set. The model design in this study included the following factors:

- Site Code: Numerical Index (1-5) for the standard monitoring sites beginning with 1 for the barrage site and ending with 5 for the furthest upstream site at Sheepwash weir (Figure 4-7). The numerical order of the sites increases with distance from the barrage so that the site code can also regarded as an unscaled distance code.
- Depth Code: Numerical Index representing the relative depth for each measurement, beginning with 1 for the surface reading (approximately 0.1-0.5 m from surface), 2 for middle depth and 3 for the deepest reading that could be achieved without disturbance of the sediment. Although monitoring was carried out in smaller depth intervals of 0.2-0.5 m, the data set had to be limited to reduce the complexity of the model.
- Tides OT: Describes the tidal regime in number of tides overtopping the barrage, as defined by the observed tide levels relative to the construction levels (Figure 4-9). Negative values stand for non-overtopping and positive values for overtopping tides.
- Flushing: Numerical index describing the two different stages of remedial action applied, where 0 represents closed gates and 1 indicates conditions during and post flushing.

Furthermore, the freshwater flow (Q_s in m^3/s) and DO or Conductivity values were accounted for in the analysis as covariates, since it was established that the response variables change to a great extent due to these parameters. The final model compared 12 monitoring days of normal barrage operation against 6 days during and after which flushing was initiated.

4.4 Results

4.4.1 Dissolved Oxygen

The ANOVA results of the GLM, regarding the monitored DO conditions in the Wansbeck impoundment, are presented in Table 4-3. It was possible to describe 24.6 % (ω^2 =0.246) of the variation in DO means by the selected parameters Site Code, Depth Code, Tides OT and the covariates Salinity and Flow at a significance level (P) of 97 %. The parameter Flushing was left in the model to demonstrate the numerical insignificance.

Source	Degrees of Freedom (df)	Mean Square (MS)	F	P	Variance of factor or interaction (σ^2)	Size of Effect (ω^2)
Conductivity	Covariate	20.29	5.14	0.025	0.032	0.006
Flow Q_s	Covariate	21.54	5.46	0.002	0.034	0.007
Site Code	4	34.34	8.7	0.000	0.238	0.046
Depth Code	2	72.46	18.36	0.000	0.269	0.051
Tides OT	16	26.8	6.79	0.000	0.717	0.137
Flushing	1	0.14	0.03	0.853	-0.007	0.000
Error	157	3.95			3.948	0.754

Table 4-3 GLM ANOVA for DO mean levels monitored during flushing on the Wansbeck

The results in Table 4-3 represent the best fit model achieved from using forward and backward procedures for the selection of factors regarding the response variable DO. No significant interaction among the factors was indicated during modelling. The magnitude of the error term (ω^2=0.754) suggests that there are still factors not accounted for in the model although all observed parameters (e.g., Temperature, different Flow measures) have been included at some stage during the modelling process.

The tidal regime (Tides OT) produced by far the largest effect among the included factors explaining 13.7 % of DO variation during the survey period. The influence of distance (Site Code) and depth (Depth Code) on the measured DO variation is of similar size with 4.6 % and 5.1 % respectively. The flushing showed no significant impact on DO levels during the survey period (ω^2=0.00). The variance for each factor level is illustrated in the main effects plot (Figure 4-10).

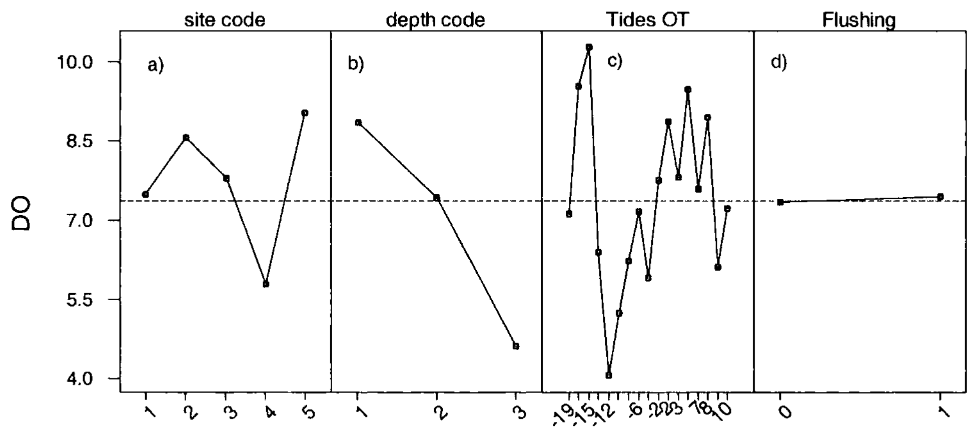

Figure 4-10 Main effects plot for DO means [mg/l] during flushing survey on the River Wansbeck, dotted line indicates overall mean

The two shallow sites (Figure 4-10-a) 2 and 5 of the impoundment indicate higher DO means than the remaining sites. Site number 4 showed by far the lowest DO conditions due to its distance and depth. The depth code (Figure 4-10-b) indicates an almost linear decrease in DO with increasing depth. Mean DO conditions at depth were below the 5 mg/l standard, which is commonly applied to salmonid rivers. The change of the tidal regime (Tides OT) to non-overtopping conditions introduces a general decrease in DO levels within the impoundment. Inconsistencies towards the end of each tidal period can be explained by the relatively large influence of the river spate flushing the system during the monitoring. In contrast, the external factor of flushing via lock operation (Figure 4-10-d) had no effect on the DO means of the whole impoundment.

Since the shallow sites showed higher DO means, indicating smaller likelihood for stratification, it was decided to examine the main effects at the deep sites separately. The Barrage site (1) and the Caravan Park site (4) were chosen to represent the distance range in which deep sites occur within the impoundment and which are in reach of tidal influx (Figure 4-11).

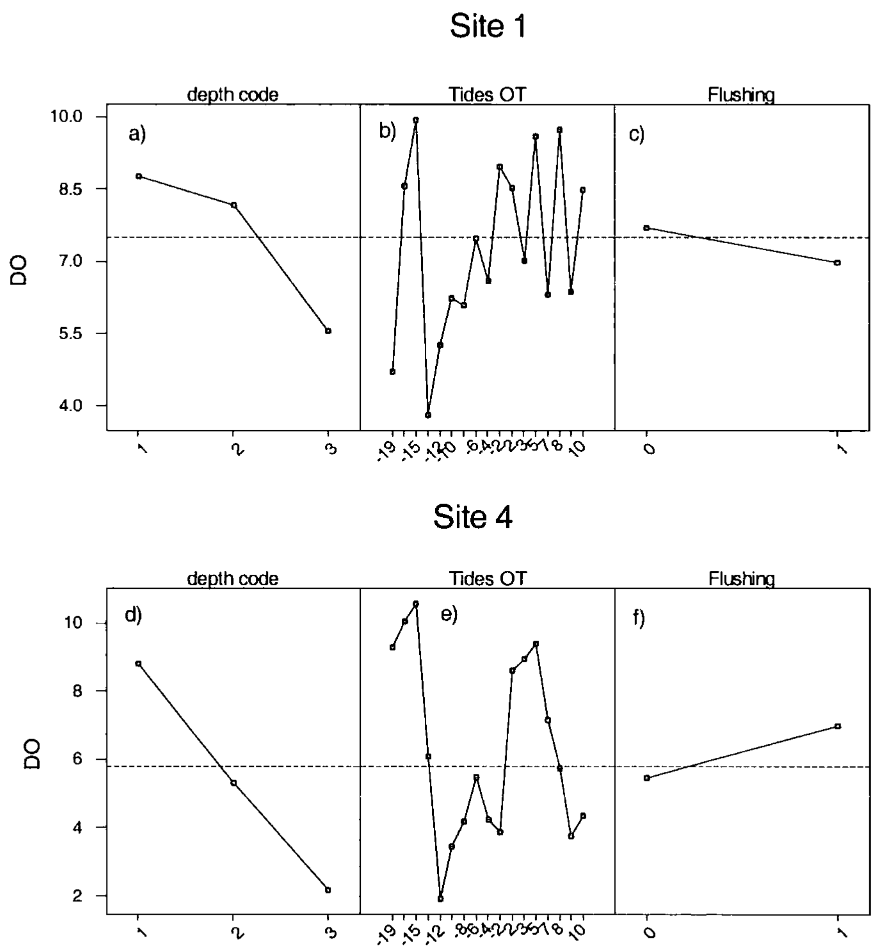

Figure 4-11 Main effects plot for DO means [mg/l] at the Barrage (Site 1) and Caravan Park (Site 4) during flushing survey on the Wansbeck, dotted line indicates overall mean

The upstream site at Caravan Park (Depth 4.88 m) shows a considerably larger influence of depth on the DO means (Figure 4-11-a, d) than the barrage site (Depth 3.66 m). Whereas surface conditions are similar, the mean DO level linearly drops to values <3 mg/l at depth for CP compared to deep layer means of 5.5 mg/l at the barrage. As expected, the impact of the tidal regime is easier to follow at the barrage (Figure 4-11-b), where DO conditions during overtopping are higher when compared to the Caravan Park site where DO levels quickly decrease towards the end of the overtopping cycle. The effect of external flushing is very low at both sites especially when compared to the possible mixing initiated by the river spate. The barrage site even indicates lower DO values during and after flushing was initiated.

4.4.2 Conductivity

The results of the GLM ANOVA regarding the response variable Conductivity are given in Table 4-4. As with the previous model, the best fit resulted from a forward and backward selection of factors included during modelling. Significance level of this model is at 98 % with the factor Flushing remaining in the model to demonstrate its numerical insignificance.

Source	Degrees of Freedom (df)	Mean Square (MS)	F	P	Variance of factor or interaction (σ^2)	Size of Effect (ω^2)
DO	Covariate	449.5	5.81	0.002	729.8	0.008
Site Code	4	431.2	5.51	0.000	277.5	0.029
Depth Code	2	2863.9	37.04	0.000	109.2	0.113
Tides OT	16	234.8	3.04	0.000	49.4	0.051
Flushing	1	143.4	1.85	0.175	1.2	0.001
Error	157	77.3			77.3	0.799

Table 4-4 GLM ANOVA for conductivity mean levels monitored during flushing on the Wansbeck

When compared to the previous model it can be observed that the covariate term river flow (Q_s) dropped from the model due to its large P value (P=0.632). Of the remaining factors the depth code has the largest effect on conductivity means, explaining ω^2=11.3 % of the variation. The tidal regime follows with ω^2=5.1 % while the location of sampling only had an impact of 2.9 % on the variance in conductivity levels. Again, the magnitude of the error term is very high (ω^2=79.9 %), which indicates that the model is far from complete. However, the main effects plot of the included factors (Figure 4-12) helps in understanding the principle dynamics in this estuary. No significant interaction of factors was found during modelling.

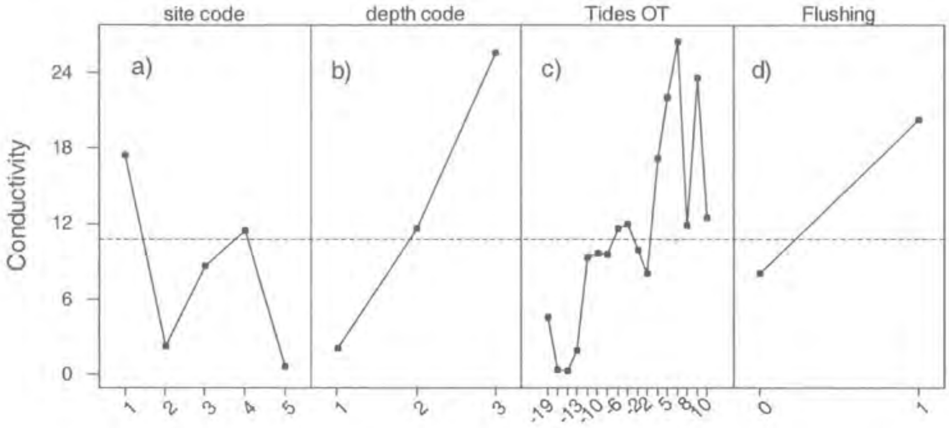

Figure 4-12 Main effects plot for conductivity means [mS/cm] during flushing survey on the Wansbeck, dotted line indicates overall mean

The saline water with high conductivity values accumulates mainly at the deep sites within the impoundment (Figure 4-12-a. The barrage site showed the highest conductivity mean of about 18 mS/cm. The overall mean is comparably high indicating large accumulation of seawater during overtopping. The stratified conditions are well represented by an almost linear transition from surface freshwater (Figure 4-12-b) to saline deep waters of very high conductivities (depth code 3>24 mS/cm). As expected, the change of conductivity levels follows the tidal regime with high levels during OT and decreasing values during Non-OT. However, during 8 and 12 Non-OT tides, the mean level for conductivity remained stagnant and then suddenly plunged into lower values. Flushing shows a positive response with increasing conductivity levels during opening of the lock gates (Figure 4-12-d).

Since the deeper sites showed higher conductivity means as result of accumulation of denser seawater, it was decided to examine the main effects at these sites separately. Again, the Barrage site (1) and the CP site (4) were chosen to represent the distance range in which deep sites occur within the impoundment that are in reach of tidal influence (Figure 4-13).

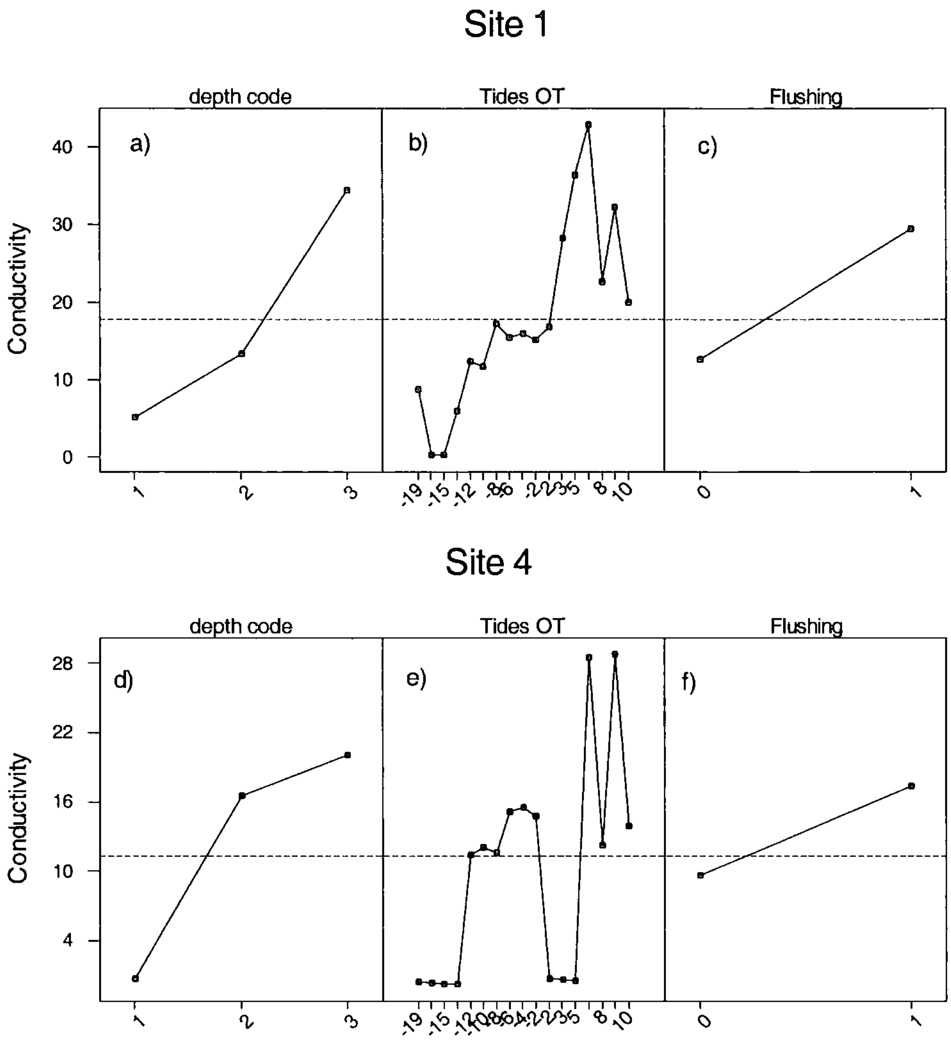

Figure 4-13 Main effects plot for conductivity means [mS/cm] at the Barrage (Site 1) and Caravan Park (Site 4) during flushing survey on the Wansbeck, dotted line indicates overall mean

At the CP site the influence of depth on conductivity is larger between surface and mid depth layer in comparison to the Barrage site. Conductivity means at CP change from freshwater levels at the surface to 16 mS/cm for the mid layer and continue to increase only slightly to approximately 20 mS/cm for the deep layer. In contrast, levels continue to increase between mid and deep layer to means of 35 mS/cm at depth for the barrage site. The tidal regime is again well reflected by the conductivity means at the barrage showing increasing levels with more overtopping, while stagnancy and dilution is occurring during Non-OT. Both sites indicate higher means during and after the flushing operation with barrage levels rising from 10 to 30 mS/cm.

With regard to the low DO and high Conductivity conditions in the epilimnetic and hypolimnetic layers of the deeper sites, the Barrage and CP site were also further examined for variation of the major nutrients that were sampled during the survey. GLM ANOVA was applied including the mentioned factors (Depth Code, Tides OT) and covariates (Q_s, Conductivity, DO) with Nitrate, Ammonium and Phosphate as response variables. Particular interest was given to the Flushing parameter but no significant impact on the variance of the nutrient models was indicated.

4.5 Discussion

For both impounded estuaries discussed in this chapter, the withdrawal of water is an emergency procedure that is initiated to remediate upstream water quality problems which occur as a result of density stratification in the impoundment. In both cases, the primary objective is to introduce additional shear stress via low level discharges, which in turn should increase mixing processes to break up stratification and reduce the extent of the salt wedge via secondary currents at low level. However, the procedure is associated with several problems due to the hydrodynamic behaviour of stratified water bodies. Firstly, the flow velocities required to successfully introduce large scale mixing processes at the density interface are larger than those experienced during the flushing trials. Both estuaries did not experience any stratification break up during the trial although water was released at maximum discharge capacities. This is a consequence resulting from two parameters that reduce the possibility to increase the longitudinal flow pattern - the discharge capacities at the barrage and the availability of freshwater within the impoundment. The volume of water that can be extracted per time is confined by the design features of the barrage, which, in the Tawe case, is limited to the sluices that feed the navigation lock, while the Wansbeck barrage is restricted to the navigation lock itself. Furthermore, an effective withdrawal is only achievable during ebb tide periods and at times when the tide level is below the impoundment level which restricts the operation further to a maximum of 12 hours a day. The fact that the Wansbeck lock was left open over a cycle of 6 tides, minimized the success of this operation and only allowed more ingress of seawater to the lower impoundment, which was demonstrated by an increase in conductivity means during and post flushing (Figure 4-12). As seen on the Tawe and anticipated by the designers of the Wansbeck barrage, an intrusion of fresh seawater could provide a replenishment of the degraded low layers but the results from the Wansbeck survey did not indicate this positive effect (Figure 4-10).

As a further restriction for successful flushing, the availability of freshwater for diluting the saline wedge is generally not sufficient during the summer season, when water quality is most likely to degrade and flushing is required most. The low level sluicing in the Tawe promoted retention of the upstream freshwater resources but the withdrawal across the whole water column through the Wansbeck lock depleted the already limited freshwater capacities quickly.

An additional major problem associated with employment of flushing as a remedial measure is the lowering of the water level within the impoundment. Due to the elevated bed levels in the Wansbeck impoundment, the flushing operation quickly drained most of the former lake which exposed large areas of sediment to the atmosphere (Figure 4-14).

Figure 4-14 Image of the Wansbeck impoundment during flushing, looking upstream

The remaining baseflow ran as a narrow stream utilizing the main channel bed. Falling surface levels lead to the pooling of water in the deeper hollows upstream reducing the chances here for water exchange via tidal intrusion of fresh seawater. The only chance for survival of the aquatic communities, assembling in these quickly degrading pools, is a flush out via a river spate. Therefore, a complete drainage of the impoundment should be avoided during flushing operation if the bathymetry indicates the likelihood for stagnant water pools. For the Tawe impoundment a complete drainage can only be initiated by the EA in order to counteract worst case conditions of water quality. This also involves an intensive logistic operation in which the moored boats have to be removed from the marina in advance. The partial drainage option, as applied during the trial, has to ensure a required minimum residual flow of 1 m^3/s through the fishpass, which guarantees sufficient surface levels upstream and avoids pooling. However, these requirements further limit the withdrawal capacities at the barrage which reduces the overall efficiency of the sluicing technique.

The surveys suggest that a low level extraction via penstocks or sluices is advantageous over a gate solution discharging from the whole water column. The procedure utilizes the fact that vertical mixing is restricted under stratification and horizontal surge can create backwater currents within lower layers upstream. Under ideal bathymetric conditions, with a steady slope and no depressions in the river bed, the low level sluicing technique could be effective in managing water quality of an impoundment. However, the morphology of the introduced estuaries lead to hydrodynamic exclusion of further upstream reaches where denser water tends to pool in deep hollows of the river bed (Figure 4-15).

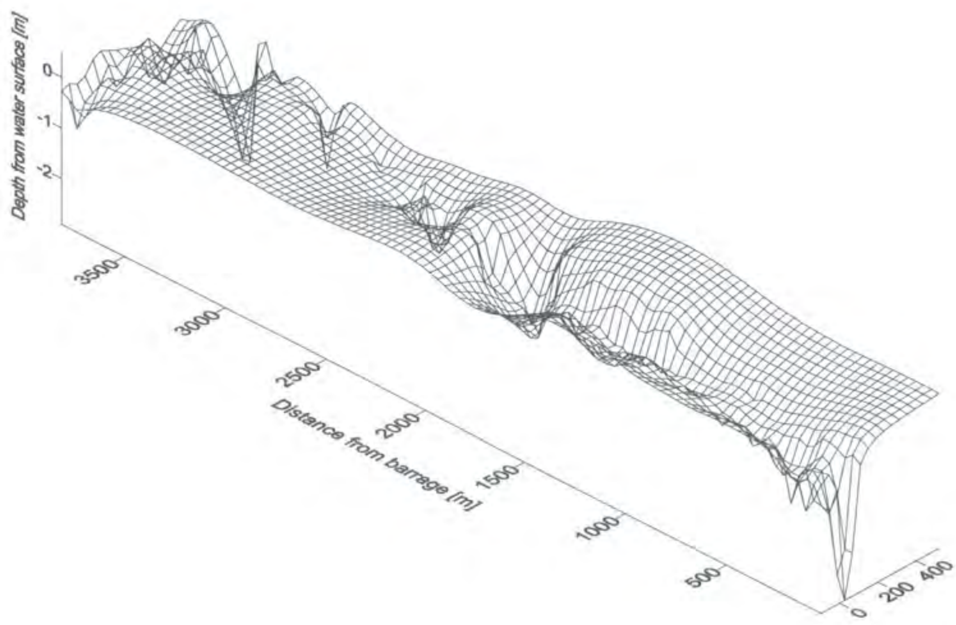

Figure 4-15 Bathymetry grid of the Wansbeck impoundment from a survey in 1999

For the Wansbeck, the density pools were identified by the accumulation of high conductivity waters at the deeper upstream sites Caravan Park and Railway Bridge (Figure 4-12-a). Currents introduced at the barrage through the discharge of water were unable to overcome the morphological barriers due to restriction of vertical movement within stratified estuaries (Figure 4-1). Thus, withdrawal of saline waters can be achieved in the vicinity of the inlet (navigation lock) but will not affect pooled waters beyond the first major upstream bed elevation where they are trapped by a falling halocline. This was one of the major findings during Tawe flushing survey, which in the following years of operational sluicing (1994-1997) proved to be the main difficulty to overcome and eventually led to the implementation of the aerator scheme.

Lastly, sluicing requires large volumes of saline water to be extracted from the impoundment over a small period of time. Whereas this problem can be neglected for the operational procedure on the Wansbeck, it is practically impossible on the Tawe to evacuate the complete volume of the saline wedge, considering the restrictions of residual flows, maximum penstock openings, tidal frame and navigational requirements (Table 4-1).

Although the ANOVA results identified the general hydrodynamic behaviour of the Wansbeck impoundment during flushing, the large error term has to be considered for the interpretation of the results. Sources of error can be identified mainly in the unbalanced nature of the data set and the subsequent modelling. It is therefore advisable to extend the monitoring of post flushing conditions in future surveys in order to produce a balanced data set and improve the description of long term flushing impacts and water quality restoration processes. In addition, it would be useful to carry out monitoring under steady flow conditions in order to improve assessment of the relative contribution of sluicing on water quality changes. Although the GLM modelling accounted for flow as a covariate, the selection of an appropriate representative for the flow conditions can be a problem. Whereas time averaged flow measurements (Q_{mean24}) lower the impact of flow on the overall model, maximum or instant flow measurements (Q_{max}, Q_s) can exaggerate it. It is therefore doubtful that the quick nature of flow spates is accurately accounted for in statistical modelling applying daily measurements. The introduction of continuous monitoring equipment (Chapter 5) would be a preferred option for the identified hot spots within the Wansbeck impoundment.

4.6 Conclusions

The data presented herein suggests that the main purpose of a flushing operation, the management of acceptable water quality within a density stratified impoundment, cannot be fulfilled by this technique due to the following reasons:

- The additional shear stress introduced by the withdrawal of water at the barrage was not sufficient to break up stratification within the studied impoundments. Mixing processes were limited to the inlet point and upstream sites showed persistent stratification throughout the flushing as a result of withdrawal restrictions at the barrage and bed elevations limiting vertical currents.
- The falling water levels (Wansbeck) and the lowering of the halocline (Tawe) lead to pooling of saline waters in upstream river bed depressions. The trapped waters

cannot be drawn by the induced currents at the barrage and are likely to deteriorate until sea water replenishment is provided or higher freshwater flows induce flushing.

- The availability of freshwater within the impoundment is significantly reduced at full level discharge as a result of the enhanced vertical dynamics under stratified conditions. Freshwater tends to overflow saline pools, which further limits dilution capacities under strained summer flow conditions.
- The complete evacuation of the deteriorated water volume is often not achievable where design and regulatory restrictions limit the discharge of water at the barrage. In addition, the free flow withdrawal is dependent on the tidal regime which puts a further constrain on the periods of discharge.

5 Total Exclusion Scheme – Temporal variations in water quality

5.1 Introduction

The barrage designs previously described in this study allowed the ingress of denser saline water and as a result experienced density stratification with the associated detrimental effects on water quality. A different approach for future barrage projects could therefore lie in separating the tidal water from the freshwater river via construction of a tidal or total exclusion barrage. Protection from tidal inundation can be achieved through appropriate combination of site selection and final crest level. However, the tidal frame as well as geological site characteristics and economic interests do not necessarily support the implementation of a tidal exclusion scheme at the chosen location. A UK example which was driven by economic and amenity interests as well as the aim to protect the freshwater river from the contaminated tidal reaches of the lower estuary is the River Tees Barrage (Hall et al., 1995a/b). The upstream impoundment created by the barrage was continuously monitored over a period of three years as part of this study to:

- Investigate temporal variation of water quality on an annual and event basis.
- Identify periods of low water quality and controls upon them.
- Determine the effects implementation of the barrage scheme had on water quality.

5.1.1 The River Tees

The River Tees has its source at Tees Head in Cumbria at 893 m above sea level, only a distance of 1.6 km from where the River Tyne rises and 10 km from the River Wear source (Archer, 1992). From its source, the Tees flows eastwards for some 160 km towards its mouth at Middlesborough, draining a catchment of 1,930 km^2 while being joined by the major tributaries River Lune, Balder, Greta, Skerne and Leven (Figure 5-1).

Figure 5-1 River Tees catchment map (reproduced from Environment Agency, 1996)

In the upper reaches the River Tees passes through sparsely populated moorland dominated by farming until it reaches the first population centres at Barnard Castle and Darlington. From there the flow widens over an extensive plain towards the heavily industrialized estuary, which was developed on large areas of reclaimed salt marshes and mudflats. The uplands of the Tees are marked by steep channel slopes and a sharp elevation drop of 630 m within the first 51 km towards Barnard Castle. These topographic features and the dominant wet peat soils lead to a 'flashy' nature in the headwaters, which are regulated by the five major reservoirs Cow Green, Selset, Grassholme, Balderhead and Blackton & Hury. Cow Green (40 million m^3) is the main regulatory reservoir which also compensates low river flows while the others are mainly used for drinking water abstraction (Environment Agency, 1996).

5.1.2 The Tees Barrage

Teeside Development Corporation (TDC) was established in 1987 to secure economic regeneration of derelict former industrial land adjacent to the River Tees and to attract investment while improving the quality of living in this area (Hall, 1996). One of the eleven flagship schemes incorporated in the TDC programme was the construction of a barrage across the River Tees to increase visual attractiveness and development potential by covering unsightly tidal mud banks with a permanent upstream water level. Since TDC´s regeneration strategy was centred around the Teesdale site, the chosen location of the barrage was approximately 17 km inland at Blue House Point (NZ 4616 1905) in Stockton (Figure 5-1), where construction started in November 1991 following the granting of the

Tees Barrage and Crossing Act in 1990. The site was not only chosen to bridge Stockton with Teeside Park (Middlesborough) but also to exclude the heavily polluted tidal reaches of the lower estuary at this point and create an upstream freshwater river of improved water quality (Hall et al., 1995b). The Tees Barrage (Figure 5-2) became operational in December 1994 and incorporates four hinged, fish belly gates, a navigation lock (6 x 25 m), a fish/elver pass and a 450 m white water course as an additional recreational and flow control feature. The hydraulically variable fish belly gates maintain an upstream water level at MHWS height of 2.65 m AOD with a regulatory operating band between 2.35 m and 2.85 m AOD depending on river discharge and tide level. Due to the tidal exclusion design the 22 km freshwater lake created by the barrage is completely excluded from the pollutant receiving tidal water of the estuary, which was anticipated to improve the water quality upstream significantly (W.S. Atkins (Northern), 1989). Downstream an increase in saline stratification and a subsequent improvement of gravitational circulation supporting a better flushing of the polluted surface layers was predicted following two-dimensional modelling (HR Wallingford, 1989). Although, the barrage maintains a raised river level upstream it was designed to minimize the increase of river surface levels during flood conditions. Lowering of the gates within the bandwidth was predicted to increase levels up to 260 mm for a 1-in-100 year flood event with a progressive reduction to the former tidal limit at Low Moor Weir 42 km inland (Figure 5-1). However, major flood defence improvements have been undertaken since impoundment to protect the former flood struck urban areas of Yarm, Darlington on the River Skerne and Stokesley on the River Leven.

Figure 5-2 Tees Barrage viewed from upstream

5.2 Methods

5.2.1 Monitoring locations and equipment

5.2.1.1 Tees impoundment

In order to observe temporal water quality changes of the impoundment on a seasonal basis as well as during fluvial events a water quality sonde was installed which recorded parameters every 30 min. The sonde was fixed at a depth of 1 m under normal barrage operation (2.65 m AOD) in 100 m upstream distance from the gate crests on the southern bank of the impoundment (Figure 5-3 - YSI Surface).

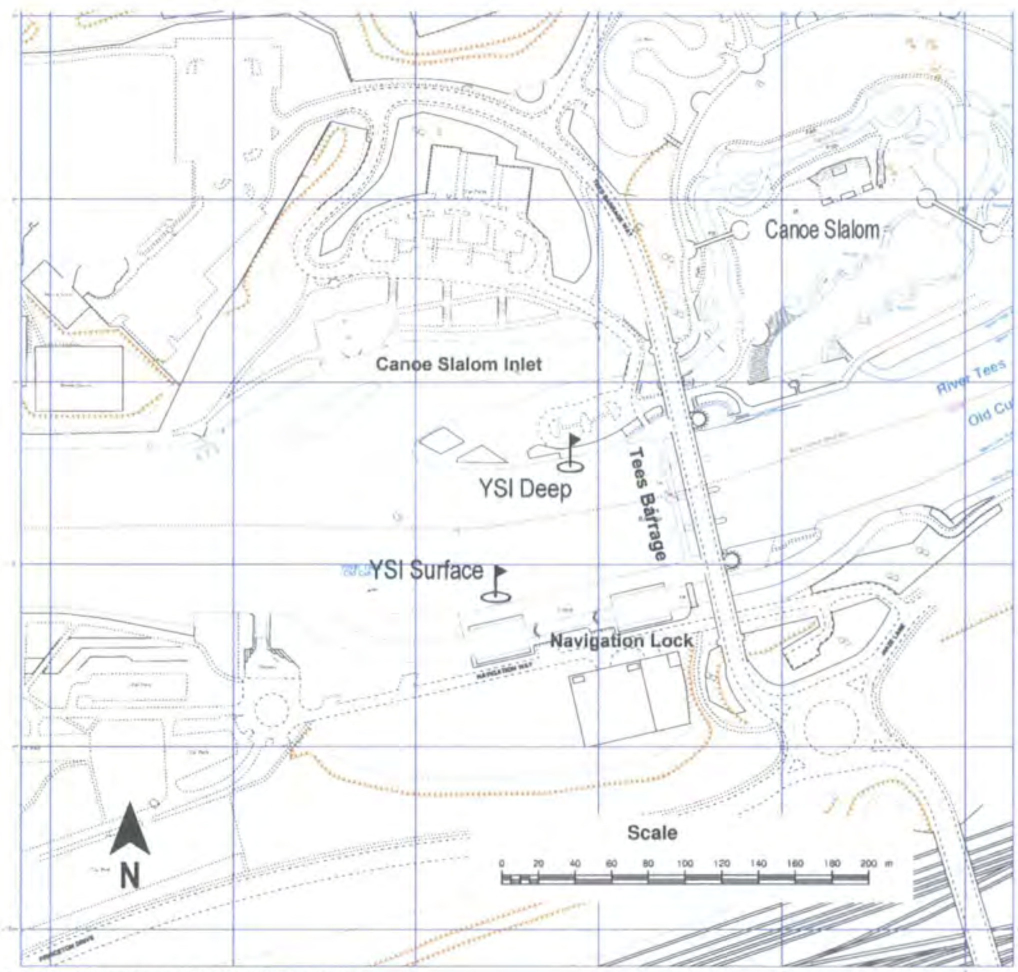

Figure 5-3 Water quality sonde locations in the Tees impoundment

The equipment was positioned within a ladder hatch to protect it from large amounts of debris that are associated with high flows entering the impoundment. Despite its sheltered position, sufficient exchange with the main flow was possible due to the large openings on the ladder guarding the sonde. The type of sonde deployed was a YSI 6920 multi parameter sonde (Figure 5-4) equipped with a customized sensor set up to record (for operation principles of these parameters please refer to Appendix A):

- Temperature
- pH
- Dissolved Oxygen (mg/l & %)
- Conductivity/Specific Conductance
- Salinity

Recordings were taken every 30 min. and the sonde was brought back to Durham University Laboratories for cleaning and recalibration every 4 to 6 weeks. The sensors were calibrated with standard calibration solutions and re-evaluation of readings was performed on the day of re-deployment. Manual readings were taken with Hanna handheld meters for data verification and recalibration of the recorded values.

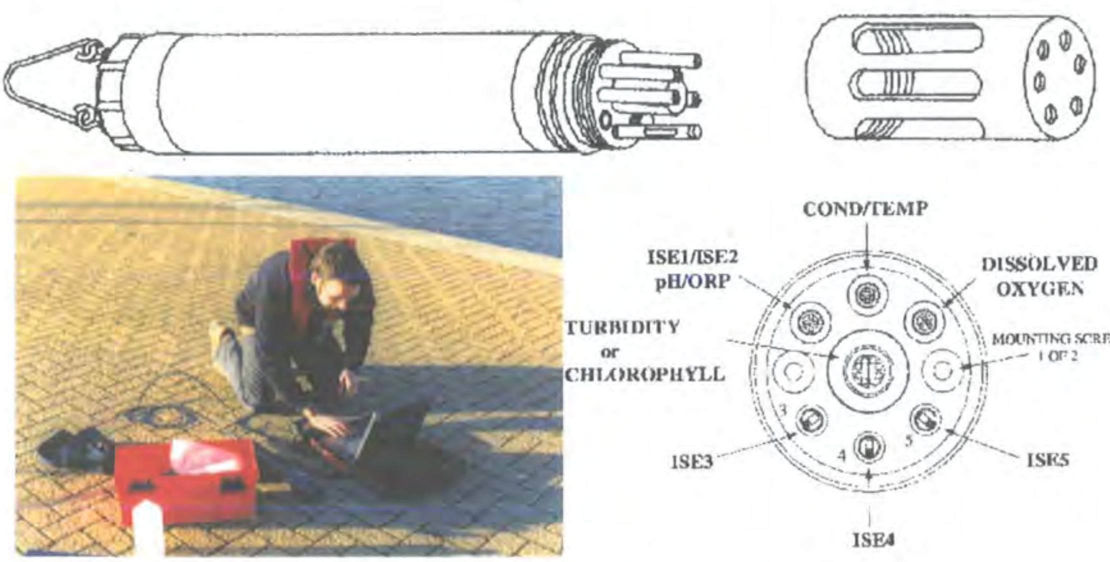

Figure 5-4 YSI Multi parameter sonde Model 6920 (YSI, 1999)

Over the course of the three-year monitoring programme, recording gaps developed as a result of probe maintenance, power failures or replacement orders. However, considering the amount of data gained from this appliance these data gaps are negligible.

An additional time series of the same major water quality parameters, but at 9 m depth, was made available from the EA Tees Region. The EA started continuous water quality monitoring in July 2002 also applying a YSI 6920 sonde fixed in a tube on the north bank of the impoundment (Figure 5-3). This probe also recorded at 30 min. intervals giving data directly comparable with that collected by the Durham University sonde.

5.2.1.2 Low Moor gauging station

A sediment monitoring station was maintained at the EA gauging station at Low Moor (NZ 365 105), 25 km upstream of the barrage (Figure 5-1), as part of the overall SiMBa project which the research of this chapter is associated with. The station setup (Figure 5-5) evolved from experience gained during the LOIS monitoring programme (Evans et al., 1997) and consisted of a Campbell Scientific CR10x data logger that was initially connected to a turbidity sensor, a pressure transducer for river level measurements and an automatic sampler unit which was externally controlled by the data logger. In order to tie in water quality recordings via the existing data logger, the setup was extended with the sensors given in Table 5-1.

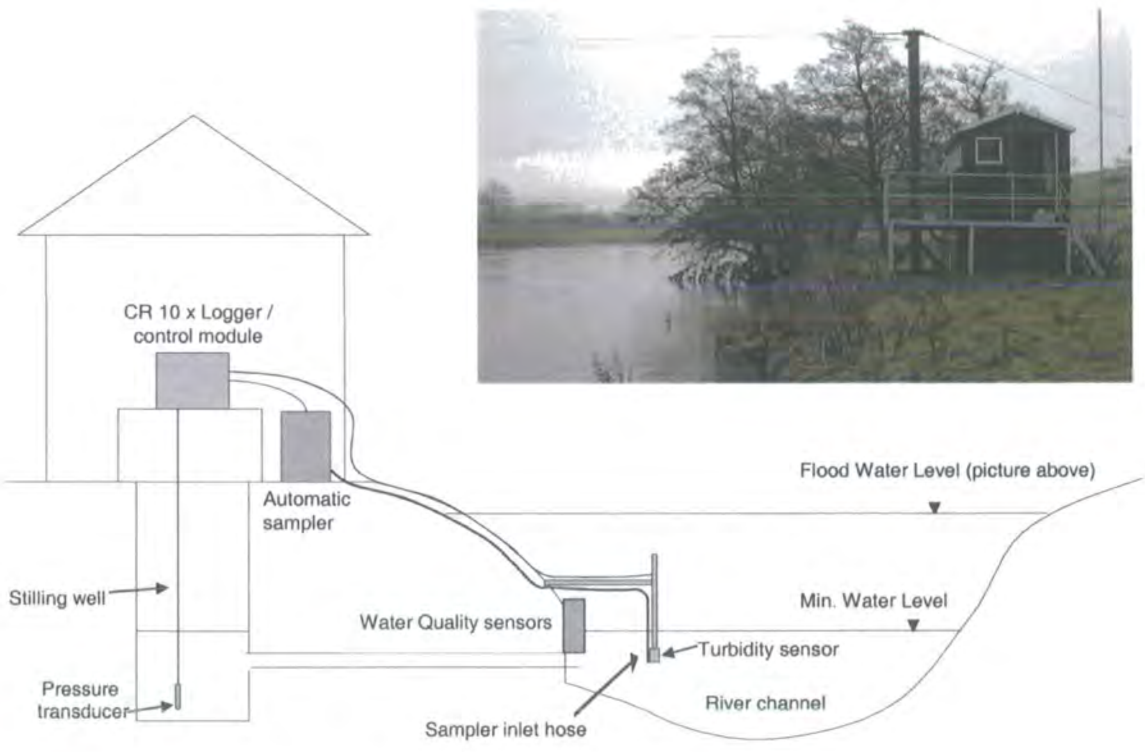

Figure 5-5 Low Moor gauging station, equipment set up

Instrument	Make and Model	Comments
Datalogger	Campbell CR10X	
Dissolved Oxygen Probe	Oxyguard 420	Oxygen Probe with built in transmitter and 20 m cable, output 4-20 mA, calibrated to 0-200 % DO
Conductivity and Temperature Probe	Campbell Scientific CS547 with Logger Interface A547	Stainless steel probe with 20 m cable, calibrated to 0.005 to 1.3 mS/cm, 0° - 50°C
pH Probe	AmpHell HI 1911B	Refillable electrode with built in amplifier to transmit -400 mV to + 400 mV signal over long distance
NO_3 Probe	DirectION Nitrate Probe	Ion selective NO_3^- Probe

Table 5-1 Instrumentation at Low Moor gauging station, technical details

Due to its susceptible position in the river, the water quality sensors required protection from the hydrodynamic forces and the debris associated with flood events. The novelty of this approach, using separate sensors in conjunction with a central data logger for continuous river water quality monitoring, required a new installation solution. A sensor guard was designed and built at Durham University to securely house the sensors as well as to allow quick maintenance access (Figure 5-6).

Figure 5-6 Sensor guard for Low Moor

The program for executing the new water quality measurements and storing the calculated results was re-written using the Campbell compiling software. Every four to six weeks the data was downloaded and the sensors were recalibrated using the same standard solutions that were applied to the YSI sonde. Manual readings were taken with handheld meters during site visits to verify the readings. The programmed reading interval of 15 min. was retained but water quality data was filtered during processing to match the 30 min. readings of the YSI sondes at the barrage.

5.2.2 Environmental variables

5.2.2.1 River discharge

River flow was continuously monitored by Durham University and the EA at Low Moor Gauging Station. The Durham University stage measurements are duplicates of the EA´s recordings and the water level is converted into discharge using official EA rating curves. The 15 min. discharge readings were obtained from the EA to verify Durham University´s measurements as well as to close record gaps resulting from station access restriction during the outbreak of Foot and Mouth Disease (February to December 2001). An additional contribution to the overall flow reaching the impoundment is given by the River Leven joining the Tees 12 km upstream of the barrage (Figure 5-1). The 15 min. records from the EA Gauging station at Leven Bridge (NZ 445 122) were also obtained and added to the Low Moor readings in order to account for the total fluvial discharge entering the impoundment. To provide consistency throughout this study it was decided to apply only the EA data sets during the analysis. The data was filtered to match the 30 min. recording interval of the YSI sonde. The flow duration curve for daily gauged flows at Low Moor (since 1969) and Leven Bridge (since 1959) is given in Figure 5-7.

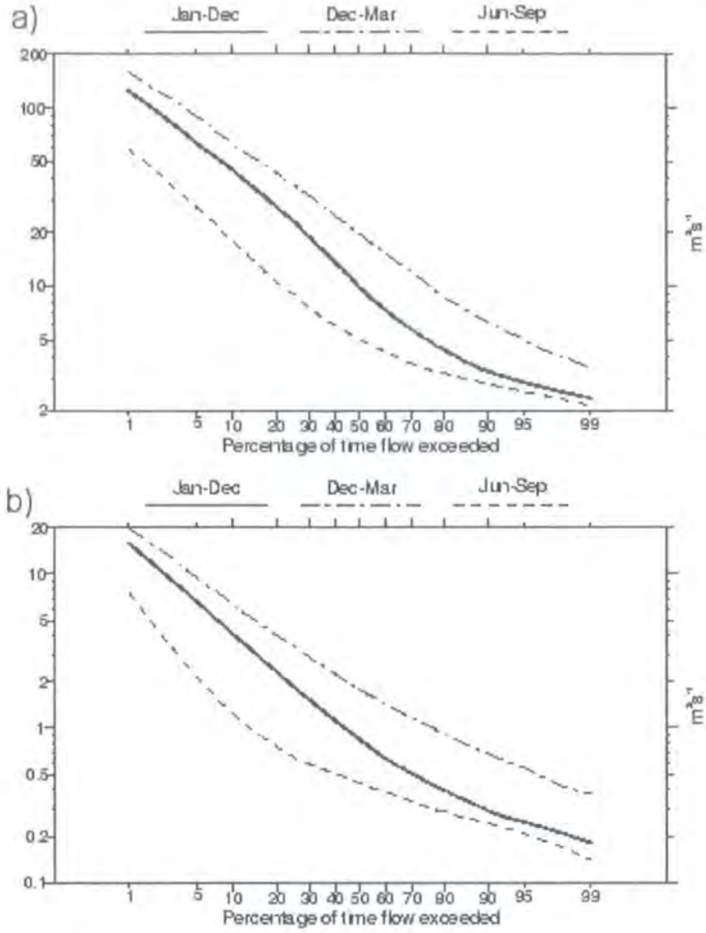

Figure 5-7 Flow duration curve for daily gauged flows at a) Low Moor and b) Leven Bridge (National Flow Archive, 2003)

5.2.2.2 Meteorological data

Meteorological data from the UK Meteorological Office Land Surface Data Set was made available by the British Atmospheric Data Centre (BADC). Teeside (NZ 422 211) was identified as the closest land surface weather station to the Tees impoundment providing hourly weather information of the area. Solar radiation levels are only recorded at a limited number of stations throughout the UK (103) of which Sunderland University (NZ 389 567) was identified as the nearest station to provide hourly interval readings for the period of the study. The solar radiation data received included large data gaps which are identified in the respective annual time series plots.

5.2.2.3 Barrage operation

The Tees barrage design includes an integrated system to record gate heights, water levels, lock operation, canoe slalom discharge and fish pass monitoring. By law, the barrage management is obliged to provide this information to the EA and other third parties to assess the operational performance of the barrage. However, technical problems with the sensors and a major hard disk crash in 2001 lead to a loss of data and a new operating system was installed after the period of study. Subsequently, no information about gate height levels and lock or canoe slalom operation was available for the study period.

5.2.2.4 Tidal Frame

Although the Tees Barrage is designed to exclude the upstream freshwater impoundment from the tidal water of the estuary, the tidal frame was considered as a parameter in this study to determine if the total exclusion strategy was effective or whether lock operation allowed for intrusion of saline water. For this, hourly tide levels were obtained from the BODC of the Sea level Gauging station at North Shields (NZ 359 682), which produces 15 min. sea level records that are quality checked by the BODC. The obtained readings were filtered to 30 min. intervals and compared with the continuous conductivity readings obtained in the upstream impoundment.

5.2.3 Impoundment volume

British Waterways conducted a high-resolution (2.5 m grid) bathymetric survey in November 2002 within the Tees impoundment. A 16 km river stretch from the barrage to Yarm was covered by the survey to identify possible siltation and bank collapse spots. The survey data was made available in a CAD format to Durham University by the EA and a volume calculation of the impoundment was attempted, creating a 3-D model of the survey grid with the GIS software package ArcMap©. The upstream boundary of the model system was defined by the Leven confluence and water level was set to 2.65 m AOD according to the main operational crest level. Hydrodynamic modelling conducted by Glasgow University (Beavers, 2003), as part of the SimBa project, confirmed a constant impoundment water level up to a 20 km distance from the barrage.

5.3 Results

The results are reported in three sections. Section one presents the results from the basic statistical analysis of the data in order to introduce the magnitude of variables, variability and temporal trends. The second section displays the observed annual variations of the parameters and identifies recurrent patterns between the monitored years while section three focuses on certain events within the recorded time series. The third section also includes water quality monitoring conducted at the upstream end and depth data from the impoundment to evaluate the effect the barrage has on water quality variations.

5.3.1 Statistical Analysis

Results from the basic statistical analysis of the 30 min. water quality records from the surface YSI sonde in the Tees impoundment are given in Table 5-2. Temperature means showed similar seasonal distribution for the three years ranging below 10°C for the first 3-4 months of each year and progressing to means of 16-19°C during the months June, July and August. Mean levels in September are already on the decline with a rapid cooling towards the winter months leaving December values around 5°C.

Naturally, flow showed high mean values in spring-time with an elevated variability for February 2001 and 2002. Although summer flows indicate lower means <10 m^3/s with small monthly variability for May, June, July and August, the occurrence of larger summer events up to 606 m^3/s raised standard deviation values for June 2000 and mean values for August 2002. Flows generally resumed to mean levels >30 m^3/s by October and stayed constantly high during the winter months with the wettest month recorded in November 2000 at a mean flow of 103 m^3/s.

DO demonstrates consistent conditions throughout the wet months of January till March and October till December with mean levels >9 mg/l. Spring conditions are characterized by a large variability in DO levels with standard deviation values of 1.95 (June 2000) to 2.5 (May 2001) from mean levels between 8.86 and 11.81 mg/l DO. Summer levels in DO are naturally lower reaching low means <6 mg/l in August and September 2002 while mean values for 2000 and 2001 stayed >6 mg/l. Nevertheless, minimum values indicate the occurrence of conditions below 5 mg/l for all three years during the summer period. The frequency of DO levels (Figure 5-8) recorded during the monitoring period shows a near normal distribution which is centred around 10-11 mg/l DO.

2000	Temperature in C				DO in mg/l				Spec Conductance in mS/cm				pH				Flow in m^3/s			
Month	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV
January	3.68	2.51	5.85	0.86	12.40	11.75	13.92	0.49	0.512	0.216	0.803	0.127	7.86	7.42	7.96	0.07	33.54	10.07	208.38	30.36
February	4.68	2.66	6.78	1.18	13.52	12.47	14.69	0.60	0.298	0.137	0.529	0.080	7.75	7.40	7.97	0.10	33.86	15.10	148.28	20.21
March	7.18	3.99	9.12	1.79	11.26	9.61	13.26	1.27	0.316	0.106	0.562	0.127	7.71	7.49	7.93	0.09	22.90	5.41	273.11	31.57
April	8.19	5.31	11.20	1.68					0.430	0.213	0.619	0.112	7.76	7.58	8.07	0.08	48.29	5.51	249.63	47.54
May	13.71	10.95	18.64	1.81	8.86	5.44	13.69	1.40	0.575	0.409	0.814	0.071	8.27	7.77	9.25	0.41	11.43	4.92	67.11	8.82
June	15.52	9.32	21.68	3.08	9.73	6.76	20.67	1.95	0.551	0.146	0.797	0.160	7.80	6.85	8.98	0.36	32.91	4.78	606.55	84.92
July	16.76	15.00	19.64	1.13					0.602	0.458	0.791	0.076	7.71	7.38	8.28	0.17	6.81	3.83	38.61	5.21
August	18.70	16.68	20.97	0.93	6.62	3.49	10.18	0.85	0.660	0.488	0.786	0.061					6.72	3.47	19.42	2.39
September	14.70	12.21	17.61	1.38	8.65	6.29	10.06	0.86	0.420	0.157	0.630	0.115					24.25	4.45	308.14	32.40
October	9.96	7.57	13.45	1.18	10.40	7.65	11.65	1.77	0.342	0.132	0.597	0.125					43.18	9.31	298.03	48.71
November	6.17	5.04	8.24	0.83	11.66	10.72	12.31	0.31	0.350	0.158	0.552	0.102	7.84	7.59	8.01	0.10	103.53	23.88	637.13	122.35
December	5.53	1.12	8.33	1.94	11.79	10.61	12.86	0.61	0.400	0.186	0.664	0.124	7.95	7.77	8.13	0.08	56.90	11.61	261.87	48.80
Grand	10.66	1.12	21.68	5.19	10.30	3.49	20.67	2.19	0.448	0.106	0.814	0.161	7.86	6.85	9.25	0.27	35.18	3.47	637.13	57.86

2001	Temperature in C				DO in mg/l				Spec Conductance in mS/cm				pH				Flow in m^3/s			
Month	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV
January	2.71	0.82	3.75	0.76	12.05	11.23	13.24	0.39	0.510	0.339	0.822	0.112	7.95	7.54	8.16	0.08	33.41	10.04	198.70	29.14
February																	58.26	11.57	593.16	94.44
March	4.56	1.84	6.71	1.30					0.629	0.377	0.871	0.120	8.02	7.81	8.41	0.15	26.93	11.50	174.56	20.68
April	7.70	5.87	10.51	1.10	9.67	8.45	10.67	0.51	0.452	0.261	0.620	0.099	7.95	7.64	8.37	0.19	33.03	12.99	214.48	27.92
May	13.73	9.17	18.28	2.90	10.21	6.12	21.43	2.50	0.544	0.335	0.647	0.097	8.20	7.59	9.13	0.41	9.69	5.37	34.46	4.66
June	15.72	13.13	20.15	1.80	7.40	4.06	12.41	1.30	0.622	0.461	0.885	0.087	7.84	7.39	8.96	0.37	9.39	4.44	26.49	5.07
July																	7.12	3.81	41.84	4.80
August	18.44	17.31	20.21	0.62	7.07	2.61	9.40	1.10	0.550	0.363	0.910	0.135	7.64	7.28	8.61	0.26	9.27	4.17	41.35	5.87
September	14.56	11.31	18.59	2.00	7.65	3.73	12.18	2.29	0.480	0.163	0.743	0.146	7.56	7.08	8.36	0.28	15.99	4.01	165.49	25.48
October	12.03	9.97	13.62	0.86	9.19	7.63	10.30	0.53	0.302	0.144	0.508	0.081	7.60	7.22	7.86	0.14	37.07	10.69	163.45	29.76
November	7.26	4.57	10.07	1.49	11.26	9.21	13.20	0.94	0.347	0.136	0.519	0.110	7.79	7.43	7.98	0.14	24.29	9.27	144.62	19.07
December	4.54	1.89	8.04	1.81	12.97	11.07	14.61	0.90	0.358	0.183	0.679	0.130	7.91	7.65	8.18	0.15	31.42	11.07	309.49	33.85
Grand	10.49	0.82	20.21	5.49	9.77	2.61	21.43	2.59	0.488	0.136	0.910	0.157	7.84	7.08	9.13	0.31	24.42	3.81	593.16	36.43

2002	Temperature in C				DO in mg/l				Spec Conductance in mS/cm				pH				Flow in m^3/s			
Month	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV	Mean	Min	Max	STDEV
January	2.86	0.09	6.97	2.28	13.93	10.45	15.43	1.08	0.449	0.20	0.71	0.146	7.95	7.73	8.07	0.07	30.53	7.11	131.80	23.30
February	5.44	3.17	7.70	1.03	12.72	10.95	14.37	5.50	0.254	0.11	0.78	0.094	7.75	7.56	8.01	0.08	86.24	18.06	443.34	73.17
March	6.28	3.64	10.38	1.83	12.55	10.08	14.27	1.21	0.373	0.16	0.56	0.109	7.79	7.50	8.05	0.12	29.88	7.83	259.48	27.41
April	11.12	9.32	14.06	1.18	11.81	9.40	16.93	1.41	0.620	0.51	0.79	0.067	8.42	8.01	9.09	0.26	7.34	3.76	43.65	4.71
May	12.90	10.82	15.70	1.44	8.95	7.14	11.04	0.69	0.359	0.15	0.70	0.136	7.65	7.13	8.35	0.26	11.96	3.01	53.31	11.44
June	16.02	13.75	18.48	1.16	6.87	5.62	7.92	0.48	0.241	0.13	0.49	0.063	7.33	7.05	7.65	0.12	14.61	3.05	142.30	16.23
July	17.40	15.18	20.47	1.21	6.13	2.56	9.68	1.12	0.390	0.27	0.66	0.115	7.43	7.02	8.20	0.24	8.86	3.00	75.66	10.14
August	18.07	14.70	20.52	1.07	5.83	3.63	8.31	0.85	0.438	0.16	0.67	0.104	7.55	7.27	7.80	0.12	21.03	3.72	375.11	48.99
September	15.76	14.23	18.68	1.18	5.64	2.97	9.21	1.27	0.438	0.26	0.71	0.149	7.51	7.19	8.12	0.22	8.71	3.31	81.81	9.19
October	10.94	6.74	15.93	2.93	9.36	6.61	11.99	1.73	0.442	0.20	0.91	0.153	7.48	7.29	7.69	0.08	31.20	3.06	284.55	47.06
November	7.63	6.45	9.52	0.80	10.94	9.52	11.81	0.43	0.358	0.19	0.54	0.078	7.52	7.34	7.68	0.08	49.17	15.88	304.25	45.01
December	5.25	2.70	7.25	1.24	9.84	8.60	10.97	0.45	0.310	0.17	0.49	0.074	7.76	7.54	7.93	0.09	48.50	12.56	311.58	42.05
Grand	10.87	0.09	20.52	5.25	9.52	2.56	16.93	2.95	0.393	0.11	0.91	0.148	7.68	7.02	9.09	0.33	28.63	3.00	443.34	41.84

Table 5.2 Monthly means and ranges of water variables from the Tees impoundment monitoring for the years 2000- 2002

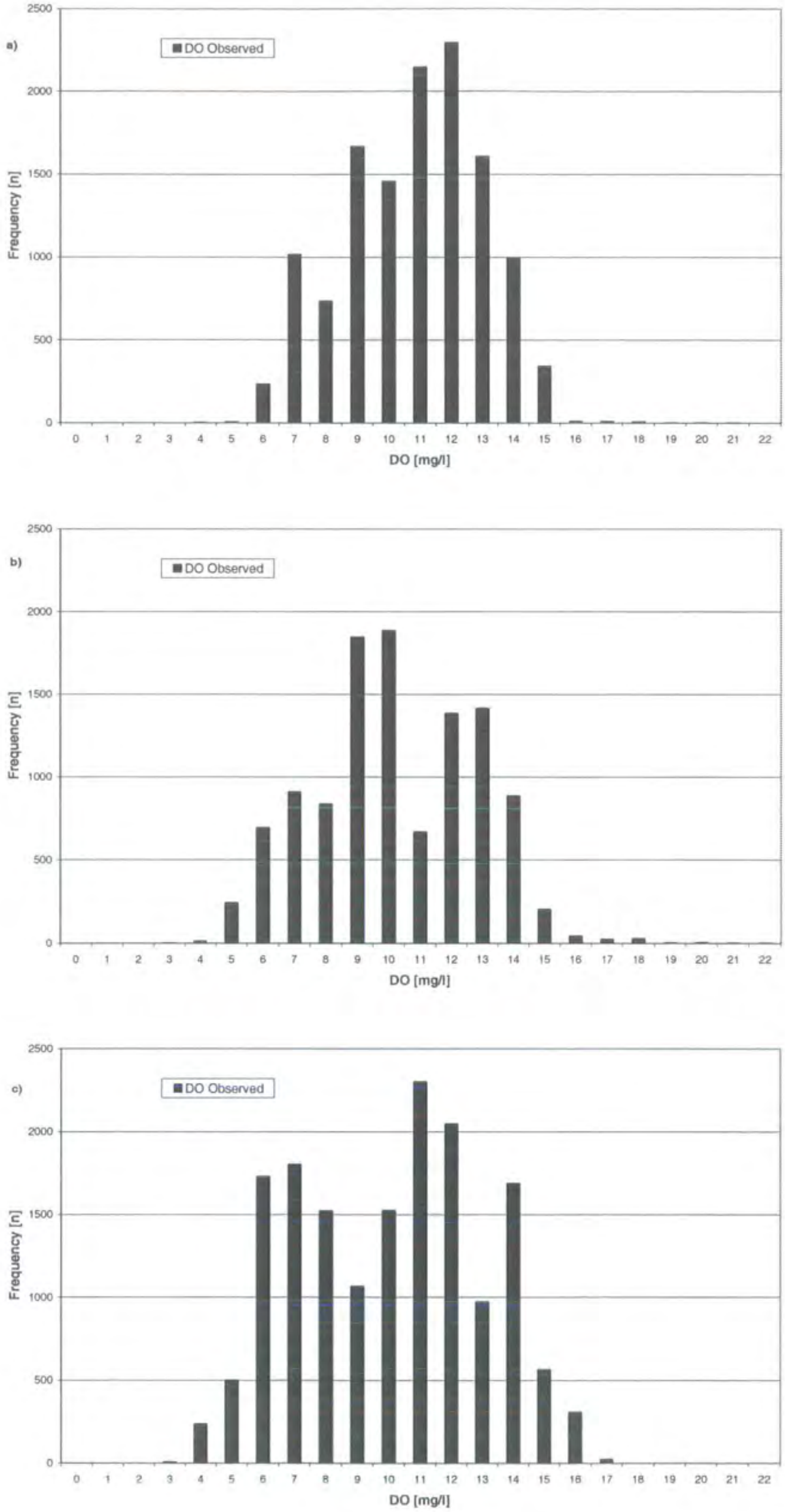

Figure 5-8 Histogram of DO concentrations measured in the Tees impoundment during the years a) 2000, b) 2001 and c) 2002

The 2002 DO histogram (Figure 5-8-c) shows an increased spread towards the lower intervals as already indicated by the mean summer values. However, levels below 5 mg/l only contribute a small portion to the DO conditions occurring in any one year with a maximum frequency of 1.4 % in 2002.

Conductivity records in Table 5-2 show a slight increase during the summer period of low flows with mean levels >0.500 mS/cm from May till August in 2000 and 2001. Mean levels in 2002 were evenly distributed with only one exceptional maximum mean of 0.620 mS/cm in April.

Records for pH demonstrate only minor annual variation with elevated mean levels during spring/early summer period as a result of episodic peaks reaching values up to 9.25 (May 2000). Monthly standard deviation values range below 0.5 pH units and values never dropped below 6.85 during the monitoring period.

5.3.2 Annual variations

5.3.2.1 Temperature

The temperature conditions within the impoundment show a strong seasonal variability following the annual cycle of temperature (Figure 5-9 a-c)).

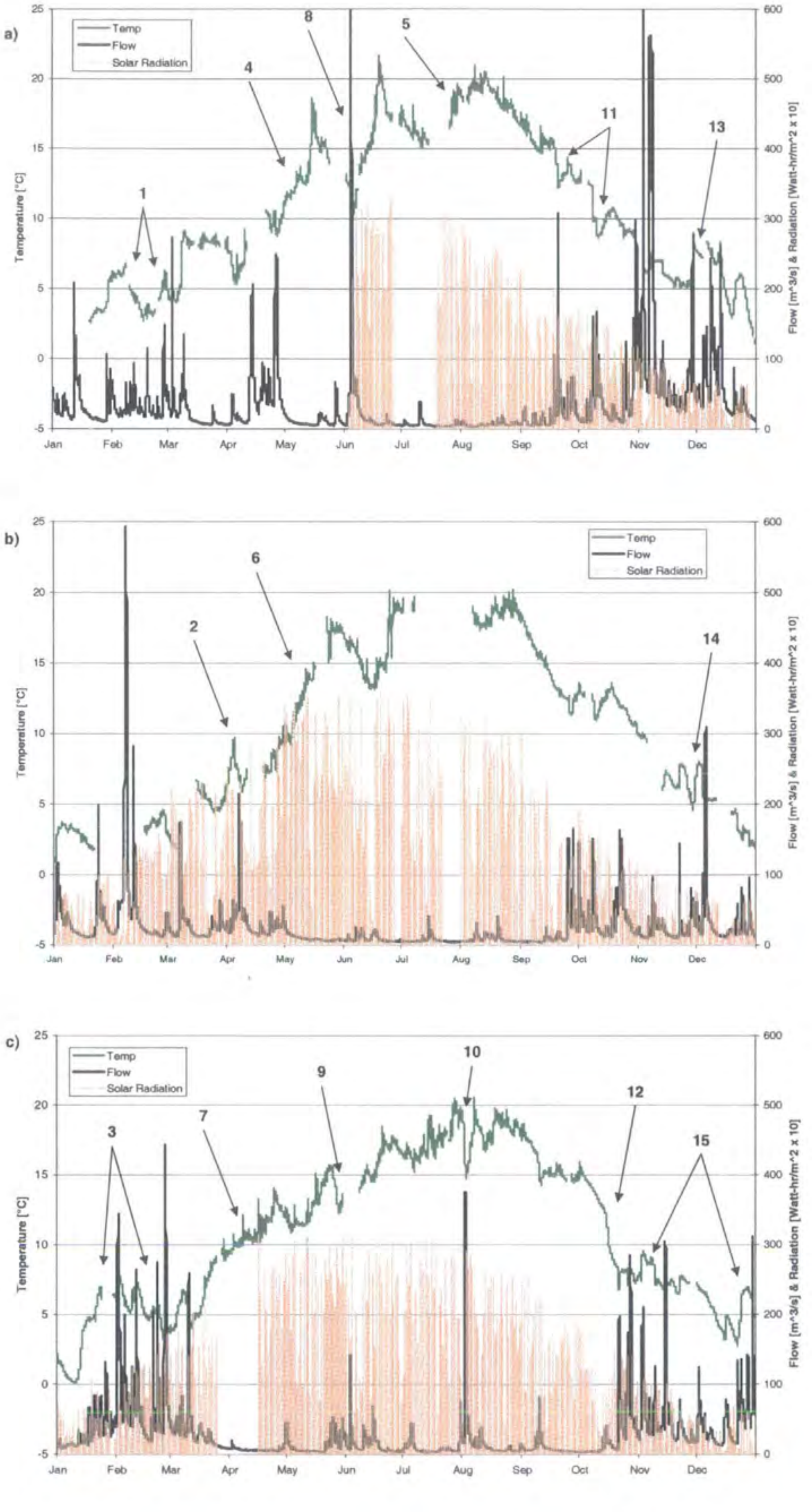

Figure 5-9 Annual time series of temperature within the Tees impoundment for the years a) 2000, b) 2001 and c) 2002, solar radiation gaps evolved from data gaps, not as a result of cloud cover, e.g. July 2000

Although slight temperature increases were introduced by flow events at the beginning of each year (Figure 5-9 Point 1-3), the main temperature rise correlated directly to the observed radiation records (Figure 5-9). Solar insolation warm-up was especially effective during periods of low flow (Figure 5-9 Point 4-7), while larger flow events lead to temperature drops during the summer months (Figure 5-9 Point 8-10). The higher frequency of larger events and declining radiation levels towards the end of each year resulted in a decline of water temperatures (Figure 5-9 Point 11-12). However, similar to the first months of the years, flow events at the end of the year also tend to warm up the impoundment at temperatures below 10°C (Figure 5-9 Point 13-15).

5.3.2.2 Dissolved Oxygen

As demonstrated in Table 5-2 consistently high concentrations of DO were found in the early months (January-March) of each year, which were dominated by high river discharges and low temperatures. Minor DO variations, occurring during these months, were mainly related to freshwater flow inputs (Figure 5-10 a-c). The months April and May showed the first major fluctuations in DO levels including super-saturation peaks up to 21.43 mg/l (227%) (Figure 5-10 Point 2), which were not associated with flow events (Figure 5-10 Point 1-3). The non-conservative behaviour (with flow) and the extreme levels reached indicate the arrival of algal 'blooms' as a result of increasing solar radiation (Figure 5-9). Following the blooms, low flow conditions and rising temperatures led to substantial decreases in DO concentrations (Figure 5-10 Point 4-5) that were only curtailed by flow events (Figure 5-10 Point 6). At the peak of summer temperatures and during long periods of low river discharges DO conditions fell below the 5 mg/l mark for several days (Figure 5-10 Point 7-8) until elevated discharges provided cooling and mixing within the water body resulting in enhanced DO levels. A general stabilisation of DO conditions occurs towards the end of each summer season as a result of the higher frequency of larger flow events. In 2000 and 2001 stabilisation occurred at the end of September while conditions remained dry and slightly reduced up until mid October in 2002.

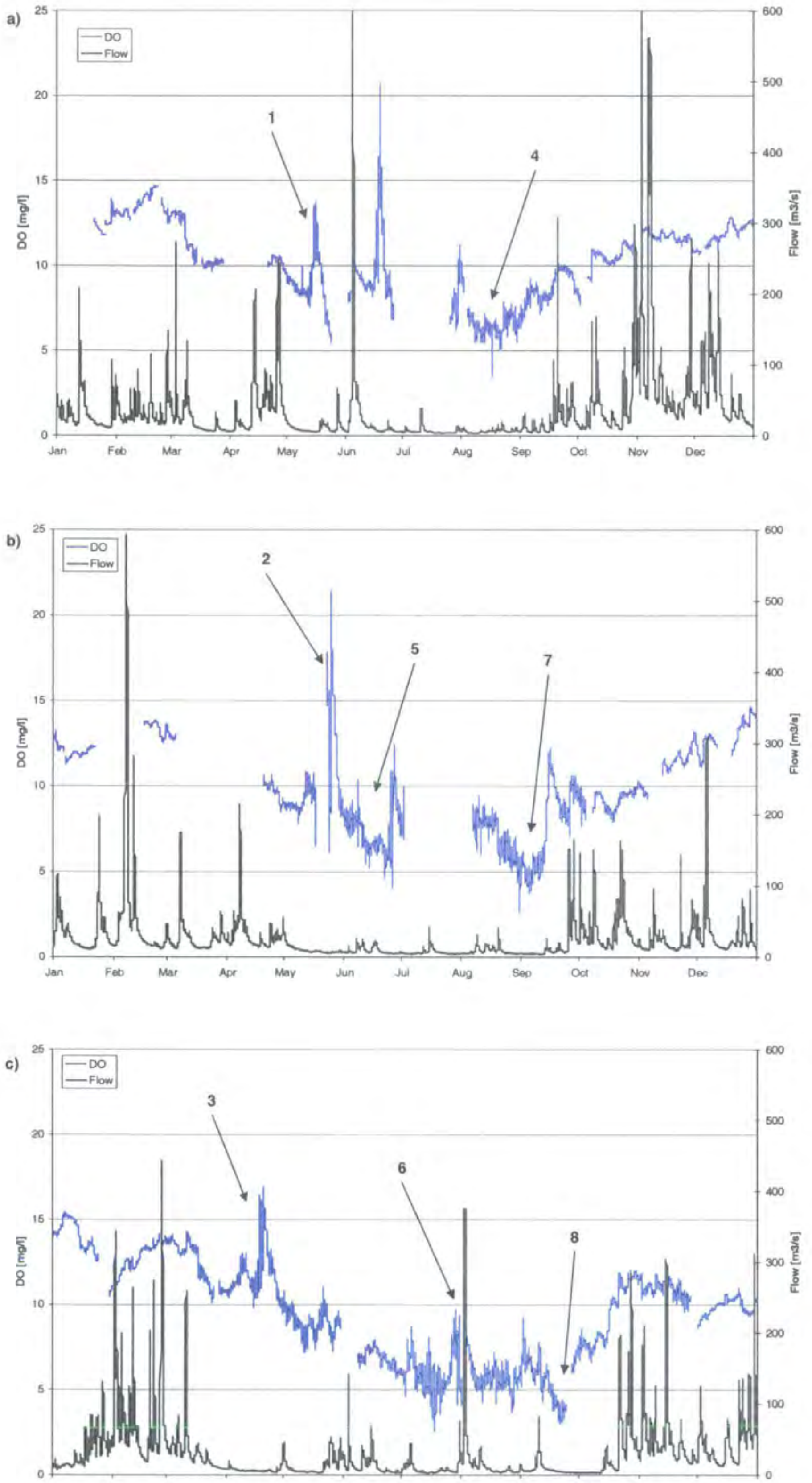

Figure 5-10 Annual time series of DO concentrations within the Tees impoundment for the years a) 2000, b) 2001 and c) 2002

5.3.2.3 pH

The time series for pH indicates small annual variation during the observed years. Therefore, only one annual example is provided (Figure 5-11). The water body shows a well buffered pH range between 7 and 9.

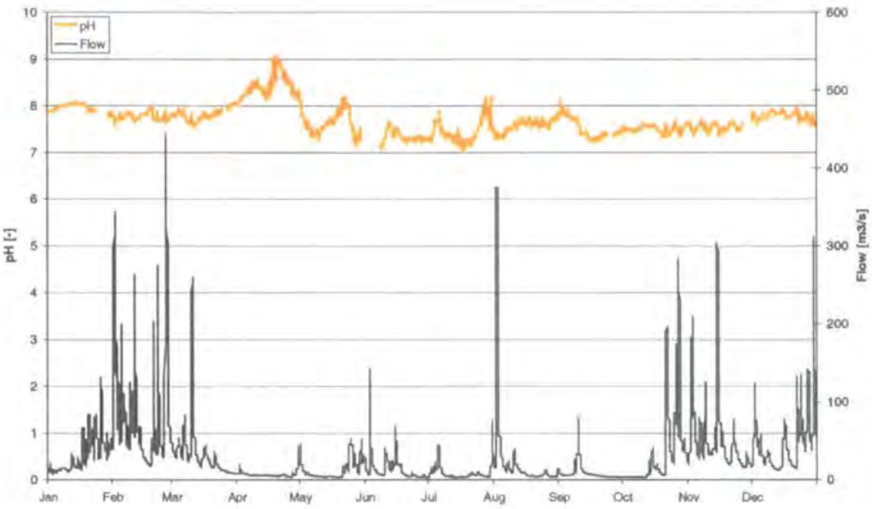

Figure 5-11 Annual time series of pH within the Tees impoundment for the year 2002

Although pH values seemed to be slightly lower in the summer, the early season is also dominated by pH peaks coinciding with the arrival of algae as observed in section 5.3.2.2. Elevated freshwater flows during the winter result in balanced pH conditions between 7.5 and 8.0 with no major fluctuations recorded throughout the observed years.

5.3.2.4 Conductivity

The time series for conductivity levels in the impoundment exhibits a strong dilution effect with increasing flow. Seasonal variation is minimal and corresponds to flow conditions resulting in longer periods of higher conductance levels between 0.6 – 0.8 mS/cm throughout the summer months of 2000 (Figure 5-12 a)). The large dilution effect of even small events led to lower levels and to a high variation during the summer in 2002 (Figure 5-12 c)). A regular distribution of peaks, especially during the low flow summer months of all years, is noticeable. However, the peaks were always curtailed by flow events and are not directly correlated to the tidal frame. High frequencies of flow events during the winter months result in values between 0.2 –0.4 mS/cm with peaks occurring on the recession of flows (Figure 5-12 Point 1-5). This is explained in detail in the following section.

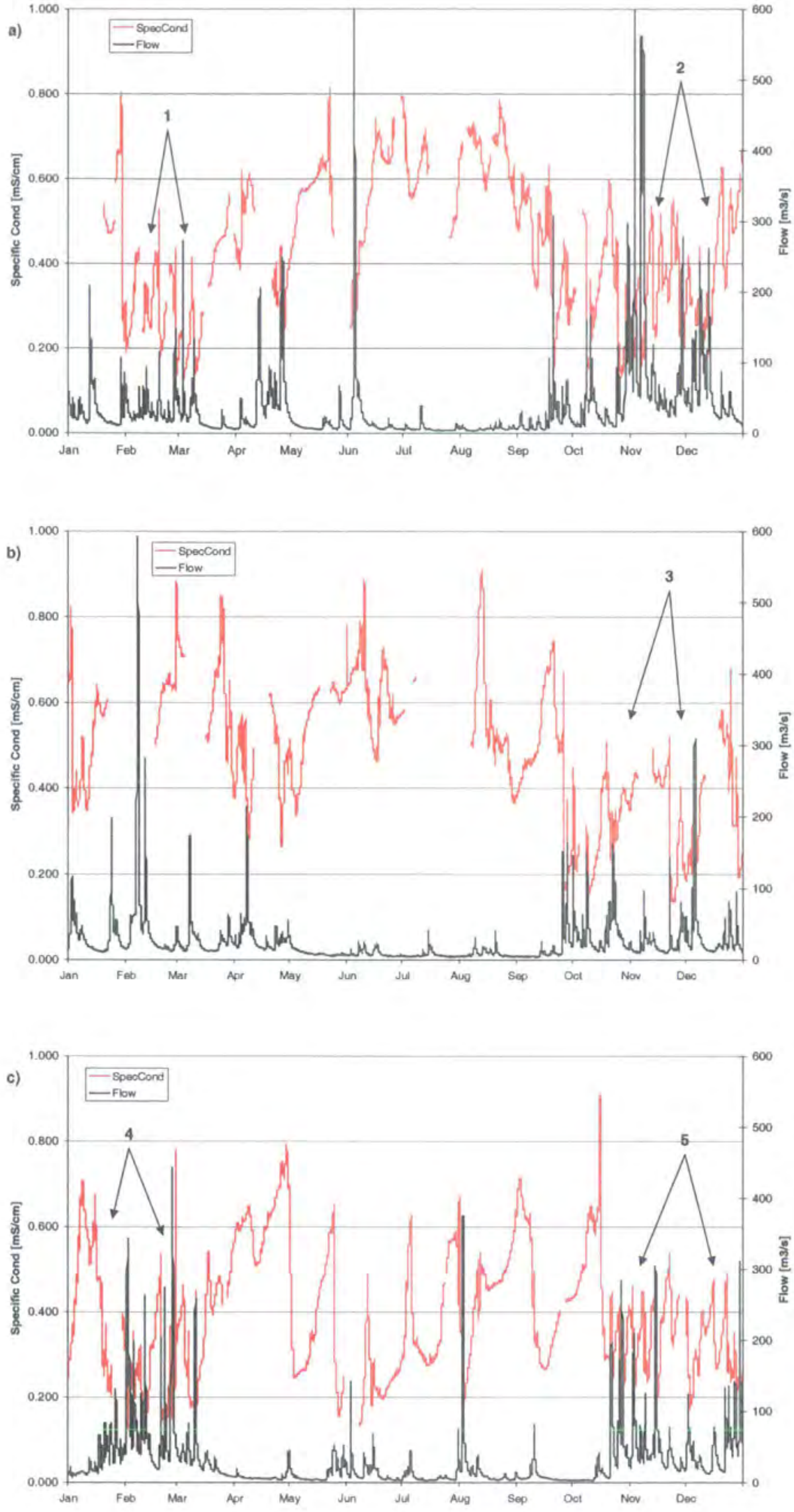

Figure 5-12 Annual time series of Specific Conductance within the Tees impoundment for the years a) 2000, b) 2001 and c) 2002

5.3.3 Temporal variations/Events

The following extracts from the recorded water quality time series are presented as 20-40 day periods in order to demonstrate season specific patterns observed within the impoundment. Since temperature conditions and DO levels are directly related the results are displayed in combined plots in comparison to radiation levels and hydrographs for the specific interval. Conductivity conditions were already found to be largely flow dependent during analysis of the annual variation and are therefore presented separately in section 5.3.3.2. The pH range recorded throughout the three years did not indicate problematic levels in terms of metal sediment release and will be therefore only quoted for explanatory purposes.

5.3.3.1 Temperature and DO

As seen in section 5.3.2.1, early spring season provides rapid warm up within the impoundment but is also accompanied by the occurrence of flow events. A closer look at a monthly time series (Figure 5-13) covering this season reveals the impact that different sized events can have on the process of raising water temperatures and oxygen levels of the impoundment.

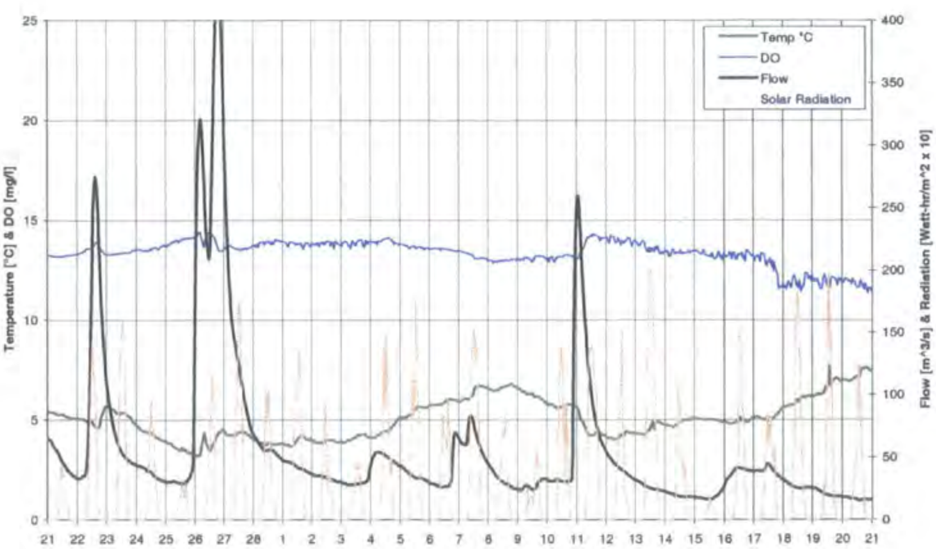

Figure 5-13 Early spring season time series of the Tees impoundment 21/02-21/03/02

It is indicated that the temperature curve generally follows the progression of radiation levels with increases occurring after longer periods of constant radiation on the 07/03 and 20/03. However, the three events above 250 m^3/s on the 22/02, 26/02 and 11/03 resulted in a temperature adjustment below 5 °C. A direct cooling effect is noticeable for the event on the 11/03, which was preceded by a cloudy day. The larger event on the 27/02 with a peak flow of 433 m^3/s coincided with higher radiation levels and buffered the impact to temperatures below 5 °C until the next radiation wave on the 04/03 set in to warm up the impoundment. DO levels were constantly high at 11-14 mg/l due to the low temperatures and high frequency of incoming freshwater pulses. DO variations during events and warm up seemed directly related to the temperature changes occurring.

In 2001 and 2002 the period of early spring was characterized by a decrease in major flow events (>100 m^3/s), which led to a progression of water temperatures that correlated with the trend in rising radiation levels (Figure 5-14).

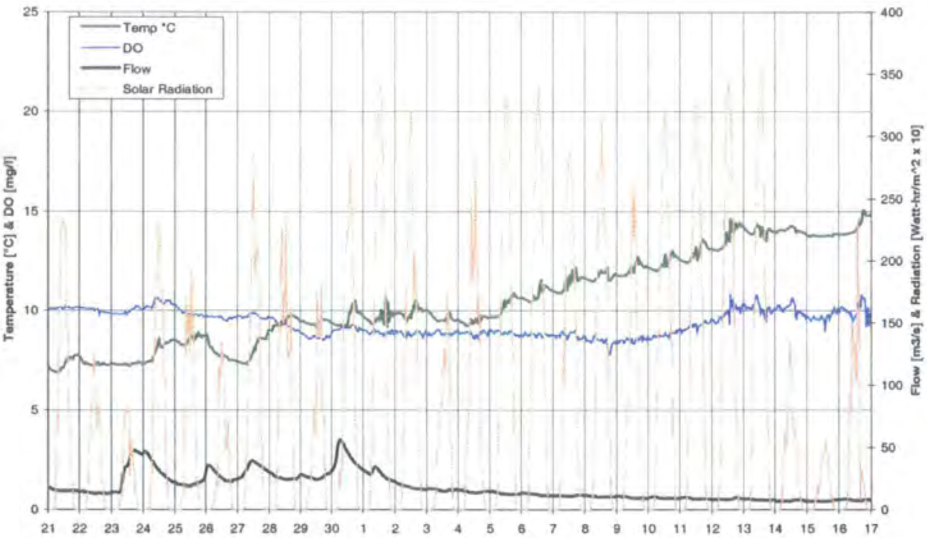

Figure 5-14 Spring season times series from the Tees impoundment 21/04 – 17/05/01

Longer periods of radiation levels >2000 Watt-hr/m^2 and low flow conditions excluding major events led to an overall warm up of >8 °C over the course of 27 days in 2001 (Figure 5-14). As seen before, water temperatures under low flow conditions directly represent the daily radiation curve (01 – 12/04/01) as well as the general trend in radiation levels. An additional, interesting, feature is the temperature buffer capacity of the water body observed on the 14/05 and 15/05/01 when a temperature of around 14°C was maintained despite major decreases in radiation levels. DO levels slightly suffered from the rising temperatures but did not fall below 70 % DO saturation. Whereas the first two days in the series still showed constant daily values of around 10 mg/l DO, diurnal variations in DO levels became noticeable from 12/05 onwards which are algae characteristic. Peak levels started to climb in the following weeks of the same year and the yearly maximum of 21.43 mg/l DO (227 %) was reached on the 25/05/01 (Figure 5-15).

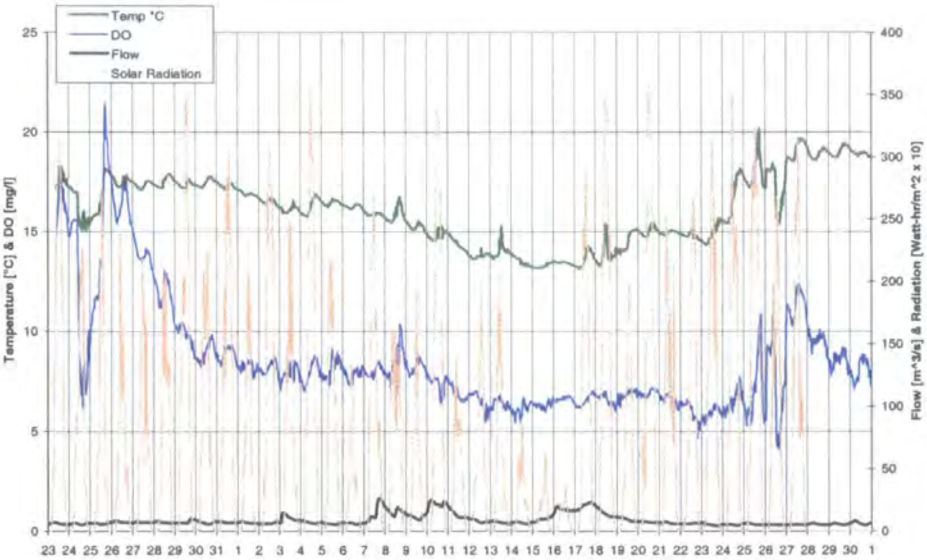

Figure 5-15 Spring season time series of the Tees impoundment 23/05 – 01/07/01

After the hyper-saturation peak, the DO quickly receded to values of 8.15 mg/l (85 %) DO with midday peaks ranging from 0.5 to 1.0 mg/l DO higher. Although conditions were further reduced towards the 5 mg/l mark during the course of the series a second major saturation peak of 12.41 mg/l DO (135.7 %) occurred on the 27/06 at the height of day temperatures (19.62 °C) which was triggered by five days of radiation above 2000 watt-hr/m^2. Both saturation peaks were accompanied by a large temperature and DO drop unrelated to flow or radiation conditions. Nevertheless, in the absence of flow events the temperature generally described the course of radiation levels and also showed a certain storage capacity during cloudy days (14 - 16/06/01). This also applied to the DO levels which nearly followed the temperature curve although the reverse would be expected considering saturation capacities. Another example of a hyper-saturation event at high water temperatures was recorded for a similar period in 2000 (Figure 5-16).

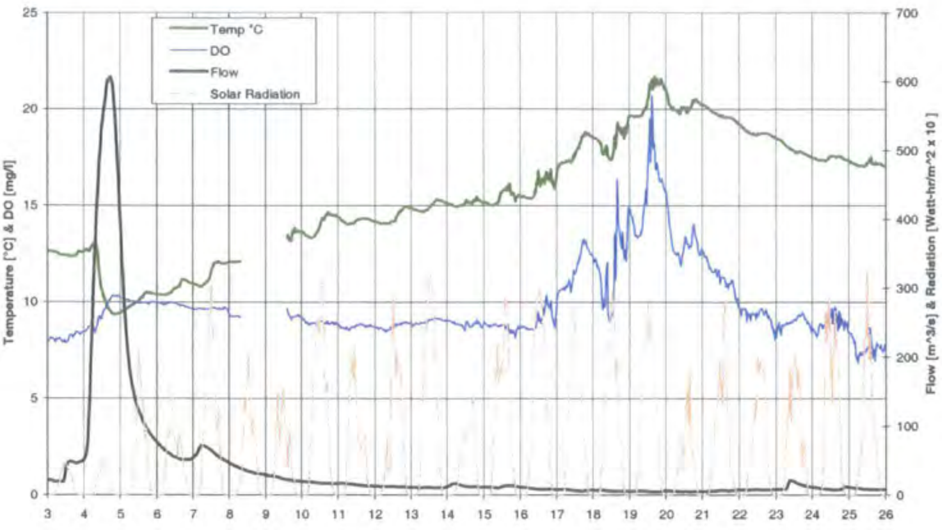

Figure 5-16 Spring season time series for the Tees impoundment 03/06 –26/06/00

The plot shows the maximum DO concentration recorded in 2000 of 19.51 mg/l DO (221%) occurring at the height of midday water temperatures of around 19 °C. Although the impoundment was initially cooled by the major flow event on the 04/06/01, water temperatures rapidly climbed as a result of the constantly high radiation levels and the quickly receding flows. The actual saturation peak was again preceded by 5 days of high radiation levels >2000 watt-hr/m^2 and accompanied by very low flows (5-6 m^3/s). The impact of a large event occurring at maximum summer water temperatures ranging around 20 °C was captured in 2002 (Figure 5-17).

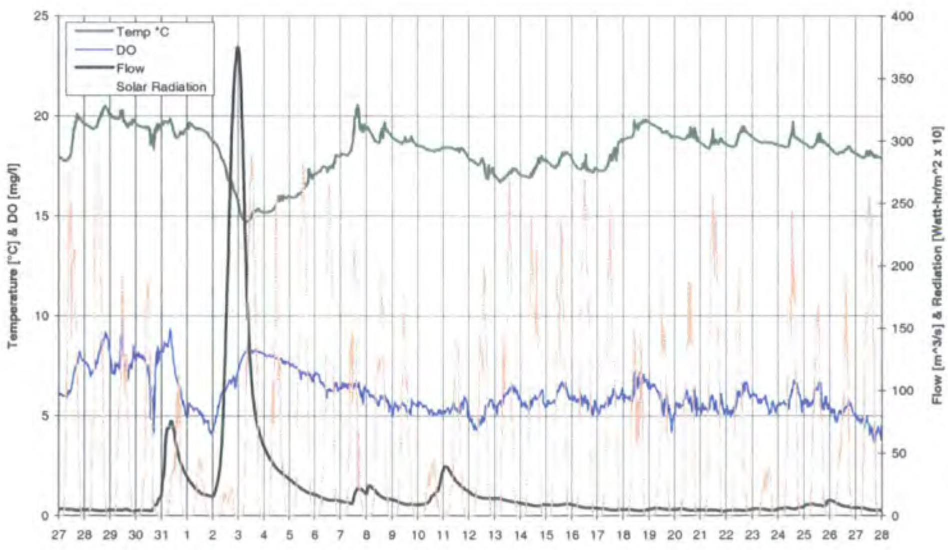

Figure 5-17 Summer season time series of the Tees impoundment 27/07 – 28/08/02

Whilst the preceding small event of 75 m^3/s on the 31/07/02 only had a minor impact on the water temperature, the main flow event of 375 m^3/s on the 02/08/02 led to a temperature drop of more than 4°C within a few hours. Although, this event was also accompanied by decreasing radiation levels the main cause has to be found in the incoming flow since following radiation drops did not result in similar cooling. Temperature levels did recover within 5 days to initial values of 20°C as a result of constantly high radiation levels and quickly receding flows. The low flows led to a temperature graph representing the course of radiation levels with afternoon temperature peaks following midday radiation maxima. DO levels before the event showed a large diurnal variation with occasional values dropping below 5 mg/l DO. During the event, the DO was restored to constant levels and slightly declined again afterwards as a result of rising water temperature and receding flows. However, DO values did not show significant diurnal variation for a period of 3-4 days following the event. With the restoration of initial temperatures and low flows, daily DO variations started again and levels were further reduced with lows <5 mg/l DO on several occasions. By the end of the summer the 5 mg/l DO mark was breached several times during the years 2001 and 2002 (compare section 5.3.2.2). In 2002 conditions at the end of the summer severely deteriorated as a result of absent fluvial exchange within the impoundment (Figure 5-18).

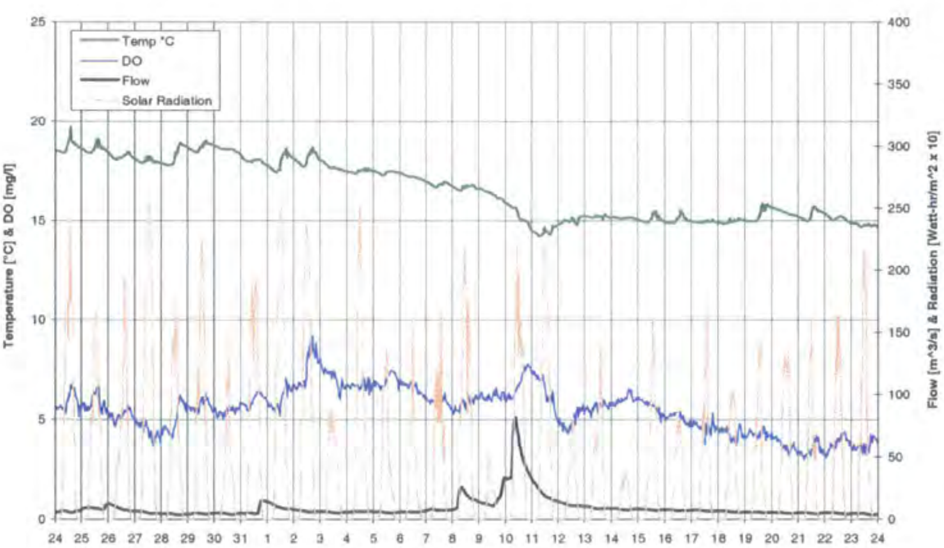

Figure 5-18 Summer season time series from the Tees impoundment 24/08 – 24/09/02

Only the minor event on the 11/09/02 with a maximum of 80 m^3/s provided a slight cooling and raised DO levels but did not have a lasting effect on the reduced conditions. Levels remained below values of 5 mg/l DO for periods of two days on the 27/08 and

28/08/02. Although temperatures decreased, a long-term breach of the 5 mg/l DO mark started on the 17/09/02. The end of the low summer conditions in 2000 was marked by the arrival of a series of high flow events (Figure 5-19).

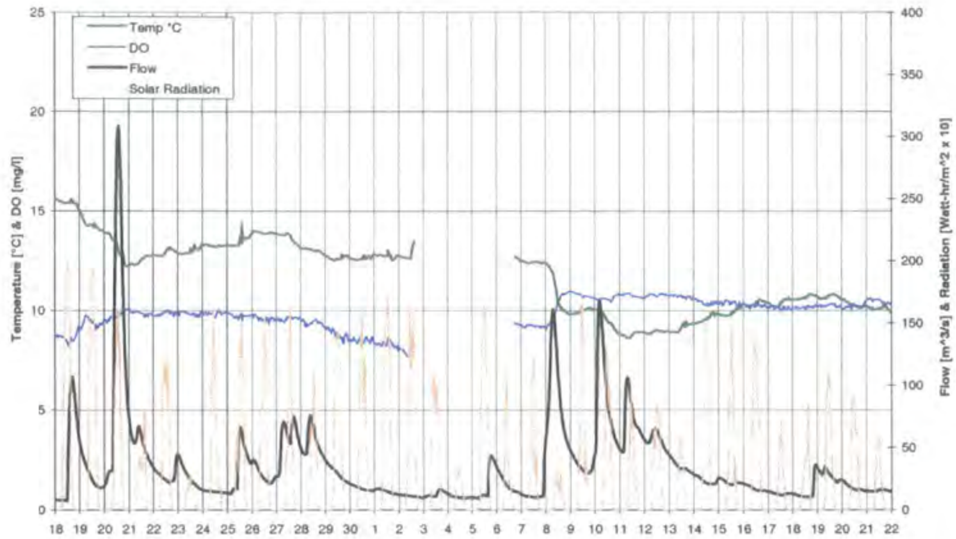

Figure 5-19 Autumn season time series in the Tees impoundment 18/09 – 22/10/00

With the general decline of radiation levels the impoundment seemed more sensitive to riverine inputs as documented in the temperature drops introduced by the events on the 18/09/ and 20/09/00 as well as the impact observed by the smaller events on the 08/10, 10/10/ and 11/10/00. The overall temperature drop of 6.76 °C was buffered after each event during recession flows which suggests that the observed radiation levels would have maintained a higher water temperature under low flow conditions. The DO was restored to values above 10 mg/l DO as part of the introduced temperature drops and the algae characteristic diurnal variations were replaced by constant daily levels after the second series of events on the 12/10/00. The end of summer conditions and the occurrence of the first larger flow event was also recorded in October 2002 when DO values were already slightly increased beforehand due to falling water temperatures (Figure 5-20).

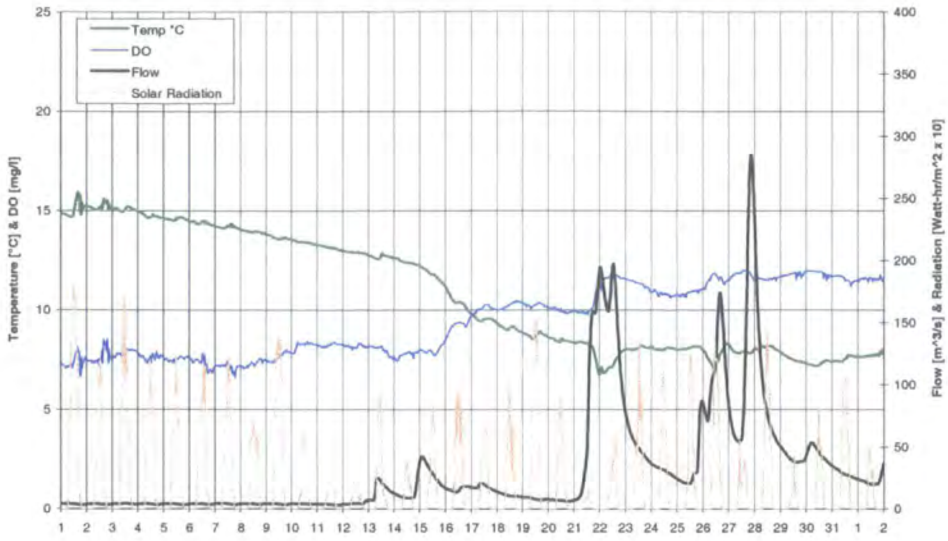

Figure 5-20 Autumn season weekly time series for the Tees impoundment 01/10 – 02/11/02

The events on the 21/10 and 26/10/02 introduced a temporary drop in temperature while the main cooling was related to the radiation curve. As the high flows declined temperature returned to the preceding values. Although the water temperature was restored to a value similar prior to the event on the 21/10/02, DO levels remained constantly high at 10.72 mg/l DO (91 %) as a result of the freshwater input. The largest event on the 27/10/02 did not demonstrate an immediate impact on water temperature suggesting a different source to the foregoing events. Finally, the effect of larger events during winter time is demonstrated by the time series recorded in 2000 (Figure 5-21).

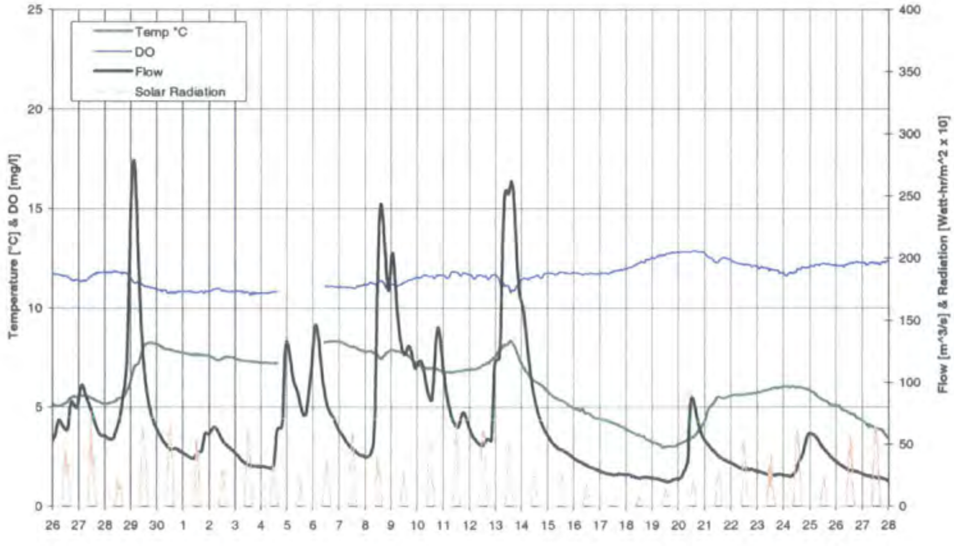

Figure 5-21 Winter season time series of the Tees impoundment 26/11 – 28/12/00

The record shows a temperature increase of >3°C during a series of large events which can not be related to variation in radiation levels. Temperature peaks following (29/11/00) or coinciding (13/12/00) with flow peaks indicate a warmer riverine input to the impoundment which maintains a slightly elevated temperature. With declining flows and dropping radiation levels the temperature falls rapidly until increasing radiation levels initiate another warm up of the impoundment. DO levels are consistently high during this season and variations result from the progression of temperature as indicated by the drop on the 13/12/00 and the higher values on the 19/12/00.

5.3.3.2 Conductivity

The episodic pattern of specific conductance depressions associated with higher flows throughout the annual time series suggests the need to focus on event based variability rather than the seasonal variability. Conditions recorded during a series of minor flow elevations in June 2001 show no direct relationship between the hydrograph and conductivity levels (Figure 5-22).

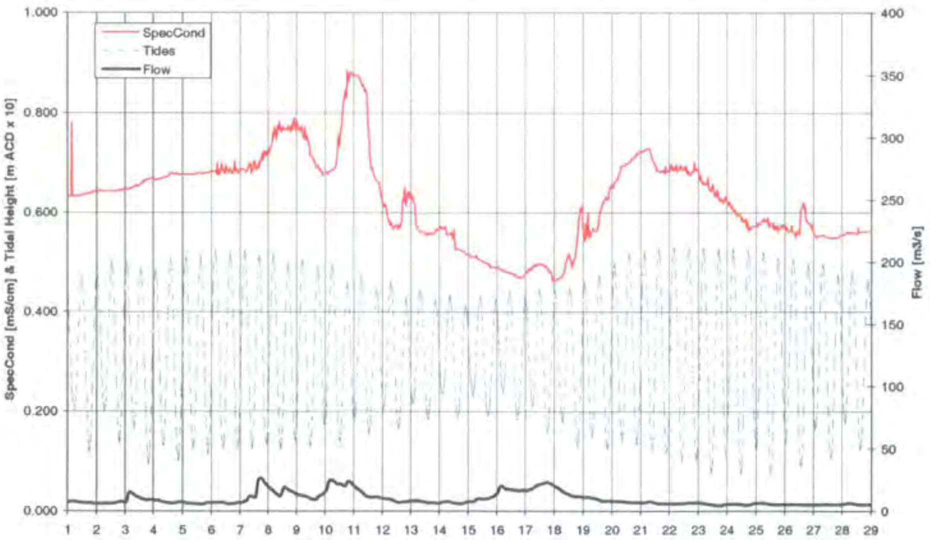

Figure 5-22 Spring season time series of conductivity for the Tees impoundment 01/06 – 29/06/01

When compared to the tidal frame of that period, an underlying correspondence in the overall trend of highest high water levels and conductance seems noticeable. However, the conductance peaks recorded on the 09/06/, 10/06/ and 12/06/01 cannot be related to the tidal frame nor to variations in flow. The dilution effect of larger events on conductivity was monitored on several occasions as illustrated by a series of events that occurred during September/October 2001 (Figure 5-23).

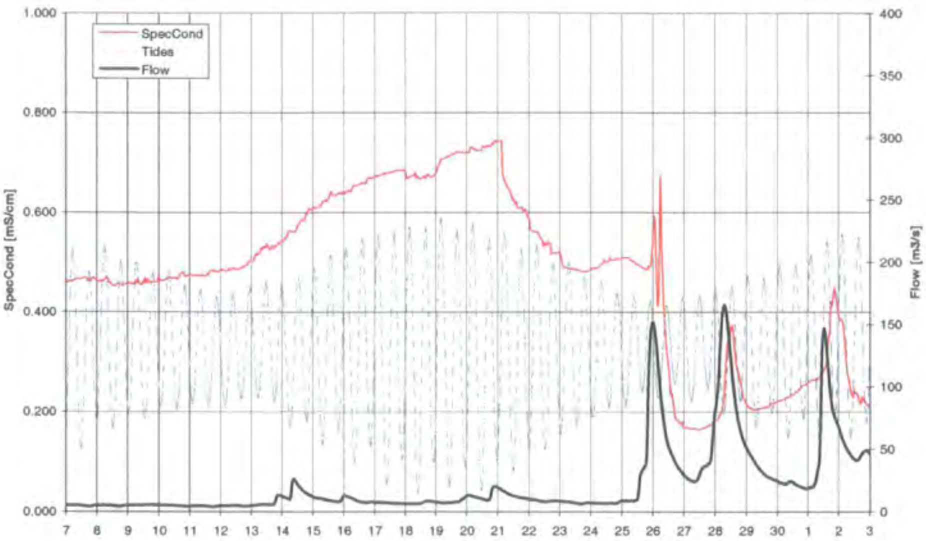

Figure 5-23 Autumn time series of conductivity for the Tees impoundment 07/09 –03/10/01

A slow increase in EC levels is indicated for the low flow conditions preceding the events with a major drop occurring on the 21/09/01 that is unrelated to variations in flow. However, the following three events on the 21/09, 28/09 and 01/10/01 lead to EC peaks that slightly lag the flow peak by about 6-10 hours. Despite their initial ionic load the events lead to an overall dilution of EC within the impoundment.

5.3.3.3 Comparison with the upstream source data

Water quality monitoring at the riverine source 25 km upstream of the impoundment started in June 2002 and commenced throughout the summer and the following winter season. Data extracts are presented in comparison with the monitoring records from the impoundment in order to determine the impact of riverine inputs and identify processes that lead to the transformation of determinants within the impoundment. The first record shows a large event that occurred at the height of summer temperatures in July/August 2002 (Figure 5-24).

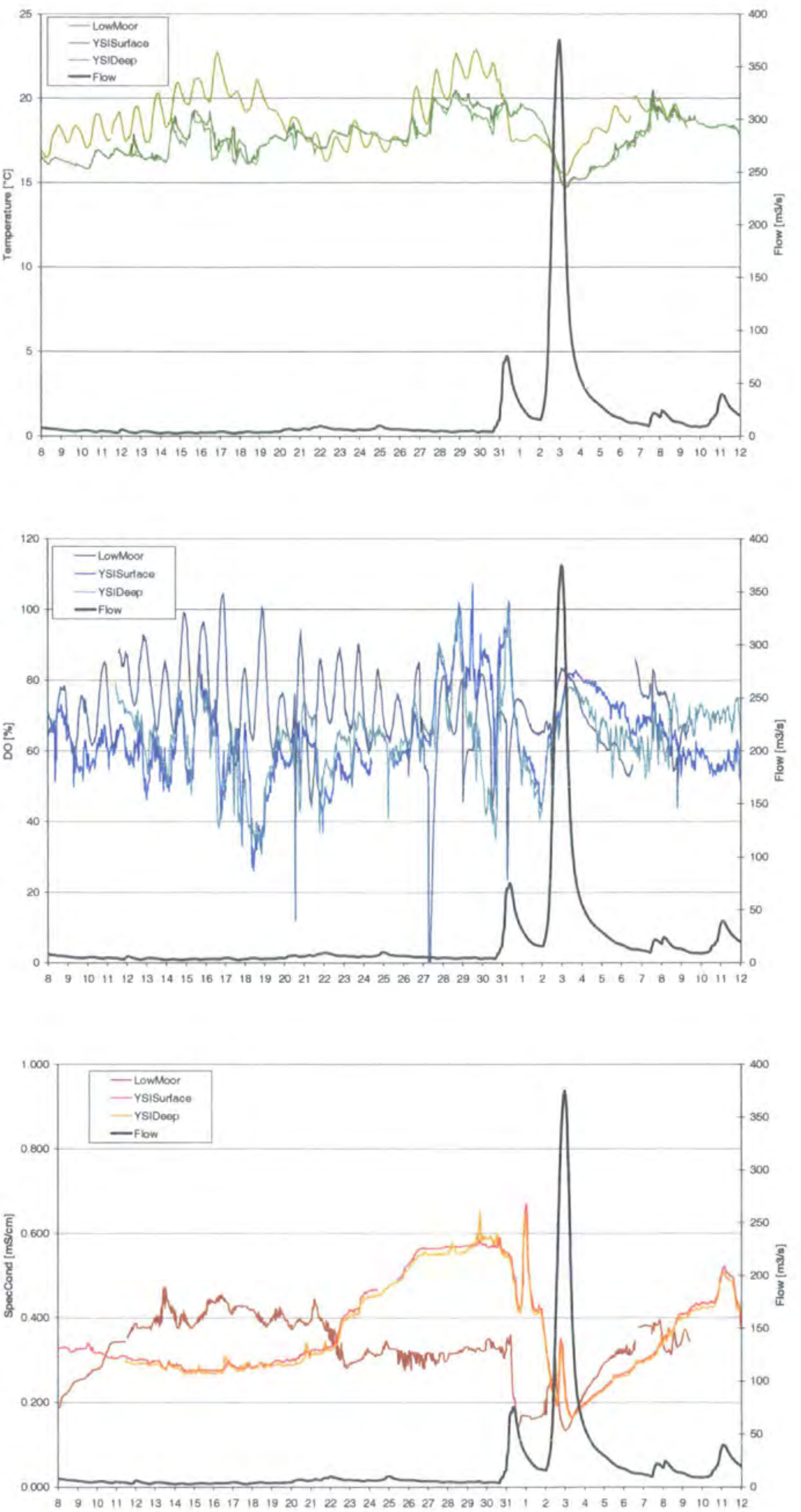

Figure 5-24 Summer event time series of a) Temperature, b) DO and c) SpecConductance 08/07 – 12/08/02 for Low Moor and Tees Barrage

Diurnal progressive temperature variation is indicated for the low flow conditions preceding the event. The surface and deep record from the impoundment does not show a temperature or DO difference during the warm up. The upstream source at Low Moor exhibits a larger diurnal variation and higher levels in temperature and DO compared to the records taken at the barrage. DO conditions at the barrage suffer considerably on the 19/07 and on the 21/07 while levels at the upstream source still show a diurnal range from 65 to 100 % DO saturation during low flows and high temperatures >16°C. The peak preceding the main event leads to a major drop in temperatures of >3°C at the upstream source whereas the impoundment only experienced a minor temperature change of 0.5°C. The main peak resulted in a rapid temperature decrease at the barrage which even exceeded the lowest value at the upstream station by 1°C. Temperatures climbed quickly at the Low Moor station whereas the impoundment buffered an increase slightly up until the 07/08 when initial values above 20°C were reached again. DO conditions at the barrage showed characteristic, algae-related, diurnal variation before the event with deteriorating levels throughout the water column. Rising radiation levels and temperatures from the 26/07 onwards led to increased photosynthetic activity at the surface of the impoundment with levels quickly reaching 107 % DO saturation while deep levels dropped to 40 %. The arrival of the first smaller peak of the event on the 31/07 provided mixing within the impoundment and equalized the DO gap within the water column. However, temperatures were still high at >19°C and the recession of flows led to a major drop in DO levels at the barrage. The second larger peak of the event restored DO conditions to values equal to the upstream levels of >80 %. Diurnal temperature and DO variation disappeared at both stations during the course of the event and reappeared again 3-4 days later (also compare Figure 5-17). The readings for EC during July indicate that the low flows initially carried a high ionic load (0.4 mS/cm) which was then slowly accumulated within the impoundment whereas levels at the upstream end declined with the slight increase of flows from the 22/07 onwards. EC accumulation at the barrage reached highest levels of around 0.6 mS/cm on the 30/07 and experienced rapid dilution to levels below 0.2 mS/cm during the course of the events. Although upstream levels indicated low input concentrations of the discharges the arrival of the quickflow component at the barrage was accompanied by conductivity peaks. With the quick recession of flows after the floods EC accumulation started again accompanied by smaller events entering the impoundment.

The final restoration of DO conditions commenced with the arrival of a series of flow events as shown under 5.3.2.2., which was captured at the upstream and barrage location during October 2002 (Figure 5-25).

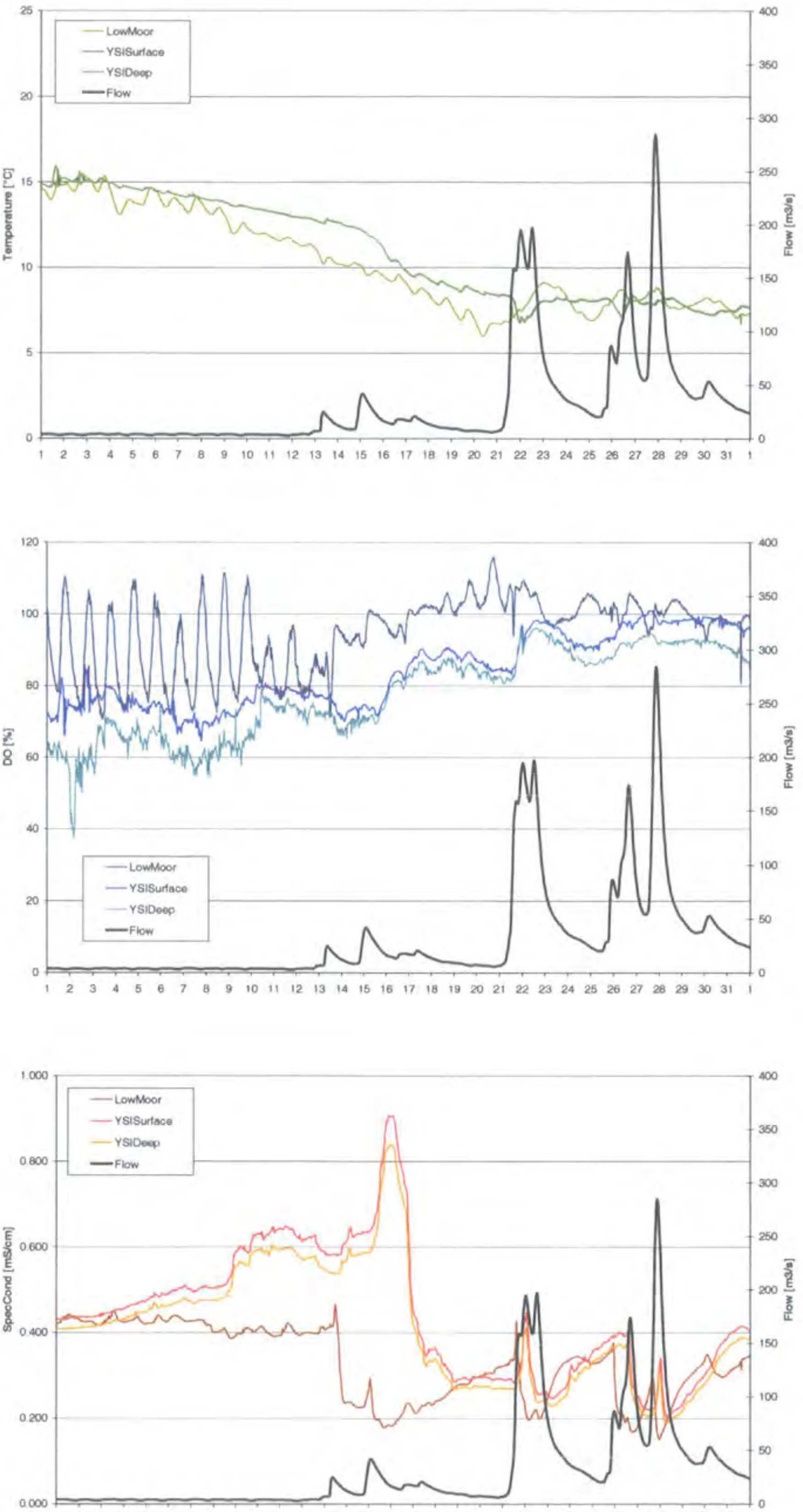

Figure 5-25 Autumn event time series of a) Temperature, b) DO and c) SpecConductance for Low Moor and Tees Barrage 01/10 – 01/11/02

The record initially illustrates the characteristic diurnal summer variation of DO levels under low flow conditions, which again is more pronounced at the upstream end although temperature variation decreased compared to the July record. With the introduction of the first slight elevation in discharges from the 13/08/02 onwards the daily DO variations became less pronounced and temperatures dropped steadily. Conditions in the impoundment initially lagged behind in terms of temperature and DO but equalized with the arrival of the first major peak flow on the 22/10/02. Constant daily DO levels predominated the impoundment from then on although the upstream end still showed slight diurnal variations up until mid-November. Conductivity values at the barrage exhibited an extraordinary ion load (>0.9 mS/cm) associated with two minor events entering the impoundment after a month of dry and hot conditions. Upstream levels show EC dilution during these events which quickly followed after the peak levels measured on 16/07/02. The following series of larger events also exhibited EC peaks on their quickflow components but did not further dilute EC during baseflow due to the already low levels of <0.3 mS/cm. Instead accumulation started again on the recession of flows.

5.3.3.4 The barrage effect

The main hydraulic effect introduced by the barrage is the artificially raised water level to a constant height of 2.65 m (± 0.15 m) AOD. The pre-barrage River Tees would have only experienced comparable amounts of water under HHWL spring tide conditions on a diurnal basis. The constant water volume for the river stretch from the barrage to the boundary limit of river Leven confluence was calculated using a 3 D model created in GIS software ArcMap© based upon a 2002 bathymetric survey by British Waterways (Figure 5-26). The volume and surface area of the impoundment were defined as:

$V_{impoundment}=3{,}760{,}464\ m^3$

$A_{impoundment}=910{,}252\ m^2$

As a result of the water storage, flow velocities decrease towards the barrage and residence time increases. The retention time of neutral water particle within the 12 km stretch for a mean daily flow of 20.72 m^3/s (50 percentile-Figure 5-7) is prolonged to 50 hrs (Figure 5-27). It is not possible at this stage to estimate the pre-barrage conditions.

Figure 5-26 3-D model of the River Tees bathymetry from the barrage to Leven confluence (unscaled schematic)

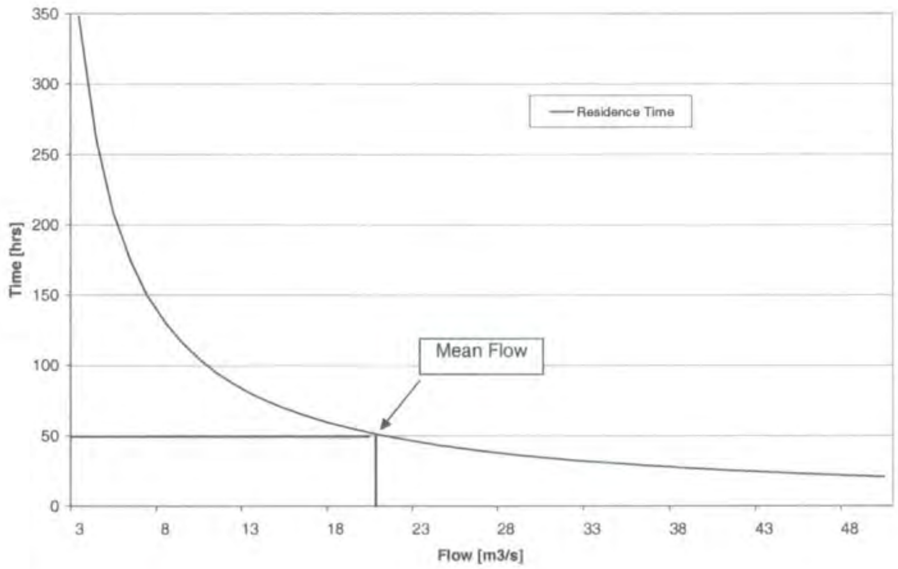

Figure 5-27 Q95 to Q10 Flow range vs residence time in the Tees impoundment

The extended residence time affects the measured water quality parameters, which can be defined as the residual between the source (Low Moor) and the impoundment (Barrage YSI) readings (Figure 5-28 a-c).

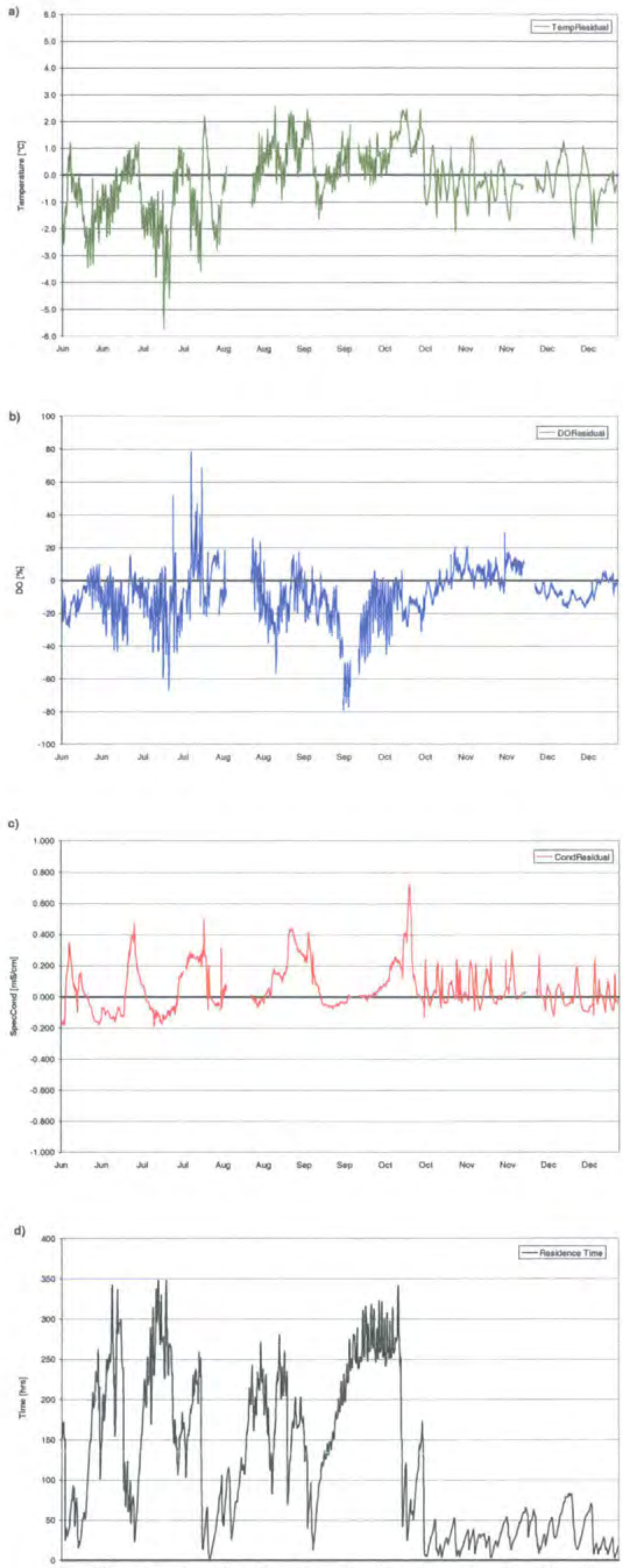

Figure 5-28 Residual time series for a) Temperature, b) DO and c) SpecConductance between Low Moor and Tees Barrage vs d) Residence Time

Residence times above 200 hrs occurred regularly during the summer months in 2002 as a result of low flows (Figure 5-28-d). The temperature residual (Figure 5-28-a) indicates that the longer retention time can result in a temperature increase of 2.0 °C compared to the upstream source. Flow spates reversed the conditions, since the temperature response is quicker at the upstream source, which led to negative residuals of up to – 5.0 °C for short residence times below 100 hrs. The DO conditions at the barrage showed a general decline towards the end of the summer months with average removal rates between 20 and 40% for residence times above 200 hrs (Figure 5-28-b). Large variation and positive residuals occurred during the summer due to increased in algal activity. As already demonstrated in the previous section, an accumulation of conductivity (+ 0.2 mS/cm) occurs under low flow conditions at the barrage when residence times exceed 200 hrs (Figure 5-28-c). However, the runoff component of flow spates also led to peak residuals of >0.4 mS/cm which is illustrated for most of the residence times below 50 hrs throughout the summer period. From mid-October onwards the regular occurrence of flow spates decreased the retention period below 50 hrs, which resulted in much lower residuals for DO and Conductivity. Temperature residuals, however, still demonstrated a large variation due to the rapid response of stream-water temperature at the source compared to a slower impact at the barrage.

A residence time of 200 hrs corresponds to a 75 percentile exceedance flow of 5.4 m^3/s (Low Moor + Leven Bridge) (Figure 5-7), which was applied to calculate mean residuals experienced in relation to flow (Table 5-3).

Parameter	>Q75 >5.4 m^3/s	<Q75 <5.4 m^3/s	Total flow range
Temperature	+ 0.01 °C	+ 0.58 °C	+ 0.13 °C
DO	- 9.61 %	- 17.15 %	- 12.53 %
Conductivity	+ 0.09 mS	- 0.02 mS	+ 0.05 mS/cm

Table 5-3 Mean residuals for water quality parameter between Low Moor and Barrage

On average, the barrage increases water temperature by 0.58 °C and reduces the oxygen levels by 17.15 % during low flow (Q75) conditions. For higher flows, the temperature effect becomes negligible, but DO still shows 9.61% lower values than the upstream source. Average values for conductivity only show slight elevation due to the combined pattern of low flow accumulation and high flow peaks loads.

5.4 Discussion

The presented time series of water quality recorded in the Tees impoundment for the period 2000-2002 exhibited a high annual as well as flow dependant variability of the observed parameters. Temperature conditions showed a strong correlation with the progression of radiation levels during spring time, which led to rapid warm-up of the impounded water body. Gu and Li (2002) and Webb et al. (2003) suggested that river water temperature is as sensitive to hydraulic loading as to weather conditions depending on the catchment size. However, the reduction of flow velocity and increased residence times in an impoundment generally results in a lower sensitivity towards flow variability as described by Petts (1984) and Churchill and Nicholas (1967). For the Tees impoundment the impact of spring events on the overall temperature increase was found to be only temporary, as documented by the time series showing the arrival of cold-water events (snow melt) in February 2002 (Figure 5-13). Although larger events >100 m^3/s led to short term cooling in the impoundment, the spring trend in rising temperatures was not considerably affected by the hydraulic charges. Periods of consistent low flows provided the best conditions for a warm up of the impounded water body, as demonstrated in spring 2001, when increasing radiation levels resulted in a rapid diurnal temperature progression (Figure 5-14) of >8°C within 27 days.

Diurnal DO variations started from as early as mid of April (2002) onwards. They can be attributed to rising temperature and radiation levels, while physical stresses are reduced during lower flows. Reynolds (1996) and Kawara et al. (1998) predicted an increased likelihood of algal growths occurring in impoundments due to longer residence times, decreasing flow velocities and nutrient accumulation. Evidence for algal activity in the Tees was not only given by the DO super-saturation peaks (227% on the 25/05/01) reached during spring season, but also by the coincidental pH maxima (>9.0), both occurring under constant low flow conditions. These episodic pH peaks resulted from increased photosynthetic activity leading to a net reduction of CO_2 and H^+ in the water as reported from seasonal records by Howland et al. (2000) and during continuous measurements by Neal et al. (1998) and Jarvie et al. (2001). After spring peak levels in DO and pH were reached, conditions receded to normal saturation values within days reflecting the course of radiation levels and water temperature. In comparison, all three years exhibited the first hyper-saturation peaks in April/May at water temperatures above 10°C (usually 18°C) after at least 4 days of flows <8 m^3/s and radiation levels exceeding 2500 Watt-hr/m^2.

The significance of the parameter flow was also observed in May 2001, when solar and temperature conditions were already perfect for the existing algae (Figure 5-14) but hydraulic stress possibly prevented a bloom. Once developed, the algal community seems remarkably stable against a series of smaller events but does eventually experience flushing in a larger event as documented by the change in conditions following a 370 m^3/s event in July 2002 (Figure 5-24). Algae-related diurnal DO variation was replaced by constant conditions for 3 days following this event and up to 10 days following a larger event in June 2000 (Figure 5-16). However, the progression towards super-saturation peaks was also accompanied with major DO and sometimes temperature drops (Figure 5-15) that could not be explained by variations in flow or radiation levels. The regularity of occurrence excludes an instrument failure while nutrient limitation and shading can be ruled out with respect to the further progression of DO levels. The short-term effect suggests an external control such as barrage operation but conductivity records did not indicate excessive saltwater stress via lock operation, which leaves these drops unexplained.

Longer periods of DO depressions followed in the summertime as a result of high temperatures and low flows. Peak water temperatures of around 20°C were observed for the summer months of all three years, which relates to DO saturation values of 9 mg/l DO. Mean monthly values of 7 mg/l were indicated for August 2000 and 2001, and conditions deteriorated further in 2002, when mean levels dropped <6 mg/l in August and September. Of all the recorded years, the summer of 2002 imposed the highest physiological stress on the aquatic community in the impoundment with episodic DO levels <5 mg/l already occurring by mid-July (Figure 5-24). Apart from slightly improved conditions during higher flows beginning of August and mid-September, the DO levels remained low until the end of September. Reasons for these long periods of DO depletion can be found in the nature of inflow, sediment characteristics and temperature conditions at the barrage. Results from the continuous sediment monitoring programme of the SiMBa-Project and sediment sampling (Wright, 2003) suggested that a large portion of the organic rich fluvial load (Wright and Worrall, 2001) is trapped within the impoundment due to the decreasing flow velocities towards the barrage. These BOD loads have a larger impact during summer temperatures as sediment oxygen demand from bacterial decomposition increases and re-aeration rates are reduced as a result of lower flows. In addition, the decay of dying algae further increases the oxygen demand on the water column. Water quality records at the upstream monitoring station indicated considerably higher DO levels throughout the summer of 2002 (Figure 5-24 & Figure 5-25) since sedimentation rates and sediment oxygen demand are lower within the flowing stream. Nevertheless, these DO enriched inputs did not lead to raised DO levels at the barrage since they only contributed a minor proportion to the impoundment volume. The DO levels at the barrage were on average 17.15% lower during dry conditions (Q75) and 9.6% lower for higher flows.

Conditions only started to improve with increasing flow rates and declining water temperatures during autumn of each year. In 2002, these urgently required flow elevations were delayed until mid-October (Figure 5-25), thus prolonging the period of DO depletion. On their arrival, conditions quickly equalized with upstream levels of temperature and DO saturation. The flow events also resulted in a flushing of algae at the barrage since diurnal DO variations disappeared in the impoundment whereas levels at the upstream end still showed daily variations.

Water temperature at the barrage however, exhibited a slower response to the riverine inputs during autumn season, which was initially believed to be a mixing effect of a possible thermal stratification within the impoundment. With the inclusion of deep water quality records from July 2002 onwards, it was evident, that the impoundment does not experience thermal stratification despite slight differences in the observed DO recordings during the summer (Figure 5-24). The matching temperature records of surface and deep YSI probe indicated a mixed water column throughout the seasons which is generally an advantage, considering a water depth of approximately 10 m at the barrage. However, this also excludes a successful application of the classic remediation schemes mixing and low level extraction since deteriorated water is evenly distributed within the water column.

Spatial distribution of water quality parameters was examined for a 25 km stretch upstream from the Barrage during a water quality sampling programme conducted by Durham University in summer 1999 (Wright, 2003). Observations showed that the deterioration of DO levels during summer temperatures is unlikely to extend beyond the boundary of the barrage backwater (approximately 3 km distance at mean flow) due to the existing flow velocities upstream of this point. Thus, DO reduction is mainly confined to the vicinity of the barrage and does not apply to the rest of the upstream river. Nevertheless, this could potentially affect migratory fish, which have to pass this impounded stretch to reach upstream spawning places (Gough, 1996). If it is not the low DO levels, it would be the high temperatures reached at the barrage, which pose a threat to aquatic life in the summer. Whereas water temperature downstream of the barrage is regulated by the diurnal tide to levels below 18°C, the upstream impoundment acts like heat sink/storage. The increasing surface area towards the barrage absorbs more sunlight, which tends to warm up the impounded water relatively quickly as demonstrated before. On the annual decline of radiation levels, however, the impoundment temperature lags behind (Figure 5-9) which suggests a heat storage capacity. One could argue that this is a result of air and soil temperature but comparison with upstream records did reveal a considerable barrage effect on temperature variation. Low Moor records demonstrated a large diurnal temperature variation as well as a quick response to events (Figure 5-24), whereas the temperature of the impoundment responded more smoothly to variations in flow and meteorological conditions. The average temperature gain for low flows (Q75) at the barrage was calculated as 0.58°C.

The lag in temperature and the buffer capacity against small event extremes is lifted with the arrival of larger flushing events as seen in July and October 2002 (Figure 5-25) when barrage temperatures quickly reflected the recordings taken at the upstream end. As well as the impounded water volume buffers against warm water extremes it also reduces the cooling impact of inputs and thereby maintains higher water temperatures at the end of the summer (Figure 5-25). With reference to the increased biological activity and lower DO saturation rates this has to be rated as a negative side effect of the barrage.

In terms of conductivity it has to be noted that the magnitude of levels did not indicate a direct intrusion of saltwater to the impoundment on a larger scale. Levels stayed below 1 mS/cm throughout the three year monitoring period (Table 5-2). Pulses in conductivity that were expected as a result of lock gate operation did not even appear in the depth records, which were included in 2002. However, with regard to the lower levels at the upstream end, a noticeable accumulation of conductivity occurred in the impoundment with regular progressive peaks up to 0.800 mS/cm (Figure 5-12) throughout the annual cycle. A significant correlation with the observed tidal frame and weather data (wind speed and direction), as suggested by Jarvie et al. (2001), was not established. The build up generally occurred under low flow conditions and experienced sudden break up by larger events, which provided overall dilution despite peak loads in their quick flow component observed at the barrage (Figure 5-24 & Figure 5-25). These short-lived EC peaks occur as a routing effect following quick-flow and can possibly be associated with urban surface runoff within the Teeside area since they could not be identified in the upstream readings.

EC generally showed a negative response to increases in flow at Low Moor while readings quickly peaked at the barrage and then resulted in a reduction of EC during events.

5.5 Conclusions

Continuous water quality monitoring, which was conducted over a period of three years in the Tees impoundment, proved to be a valuable tool in assessing the seasonal and short-term temporal behaviour of water quality, as well as the specific impacts arising from the construction of a barrage. The following main patterns were identified:

- Water temperature within the impoundment is largely related to solar radiation during spring time warm-up and shows a lag effect during autumn time recession of radiation levels as a result of heat storage of the impounded water body. The temperature is on average 0.58 °C higher at the barrage during low flow conditions. This prolongs the period of relatively high water temperatures at the end of the summer, until a series of larger flow events provides remediation via flushing of the impoundment.
- Despite maximum water depths of 11 m, the impoundment was not subject to temperature stratification during the summer.
- DO conditions are constantly high during periods of regular fluvial inputs occurring from October to March. Diurnal variation of DO levels, including episodic hypersaturation events, occur from April onwards as a result algal activity that develops under low flow conditions. Prolonged periods of low flows and high temperatures lead to intervals of reduced oxygen conditions <5 mg/l from August till October due to high sediment oxygen demands within the impoundment. However, conditions below 5 mg/l only contributed a small portion to the DO levels measured in each year. On average DO levels at the barrage were found to be 17.15 % lower for dry conditions (Q75) and 9.6 % lower for all other river flows.
- Conductivity shows a negative response to increasing flows with accumulation occurring under low summer flow conditions. Maximum levels below 1 mS/cm do not indicate marine water intrusion and records showed no sign of density stratification in the impoundment. Flow events in summer and autumn are accompanied by short-lived peaks, which are potentially related to local urban surface runoff since upstream levels indicate an external source.

6 Conclusions

6.1 *Review of aims of study*

The presented study covered an extensive set of water quality data that was compiled in order to provide more information on the main objectives of this research, which were:

- To identify adverse water quality effects arising from barrage construction in partial exclusion systems and determine the role of freshwater inflow.
- To describe temporal variation and impacts on water quality in a total exclusion system.
- To describe and evaluate remediation principles that mitigate these water quality impacts.

6.2 *Major findings*

6.2.1 Partial exclusion barrages

Statistical and observational analysis of water quality monitoring data prior to the installation of remediation equipment in one of the major partial exclusion schemes in the UK, the Tawe Barrage, revealed that:

- Significant deterioration of water quality occurred as a result of a saline stratification within the upstream impoundment. Vertical mixing processes were restricted due to the formation of a stable, two-layer system under low flow conditions. As a result, sediment oxygen demands led to rapid decrease in DO levels within the stagnant lower layers.
- The frequency of DO events breaching a set standard of 5 mg/l within the Tawe impoundment was largely dependent on variations in freshwater flow. The majority of DO events were observed during low flow conditions. Flows exceeding a margin of 9 m^3/s provided sufficient mixing to erode deoxygenated saline layers.
- Spatial distribution of DO events was also governed by the freshwater flow due to restriction of saline propagation up estuary.

- DO events occurred for all tidal conditions since upstream saltwater pockets were not always replenished during overtopping periods.

6.2.2 Total exclusion barrages

Continuous monitoring of water quality parameters within the Tees Barrage impoundment demonstrated the following temporal variations and effects resulting from barrage construction:

- Due to its freshwater nature, the impounded water body showed no sign of density stratification and no significant thermal stratification was detected despite a maximum impoundment depth of 11 m.
- DO levels are constantly high during periods of regular inflows (October-March) but start to show diurnal variation including hyper-saturation peaks as a result of algal activity from April onwards.
- Low inflows and high water temperatures led to episodes of reduced DO conditions during the summer season as a result of high sediment oxygen demands and lower DO saturation.
- Flow events in the summer season were accompanied by point-source characteristic peaks in conductivity. In general, conductivity showed non-flow related accumulation patterns with dilution occurring on increasing flows.
- The effects of the higher residence time as a result of barrage construction were measured as an increased heat storage capacity, a reduction of DO levels and an accumulation of conductivity within the impoundment relative to upstream conditions.

6.2.3 Remediation strategies

The remediation strategies included in this study represented the three main principles (mixing, prevention and excursion) applied and proposed to reduce saline stratification within impoundments of partial exclusion barrages. The following conclusion were drawn from the examples presented:

<u>Mixing Devices</u>

- The aerator/diffuser scheme installed in the Tawe impoundment proved successful in preventing saline stratification through introduction of artificial mixing within the water column.
- As a result DO conditions of the mid and deep layers improved significantly and DO breaches of the 5 mg/l standard were prevented.
- The equipment showed the highest efficiency at deep sites that are not subject to regular saline replenishment. Therefore, the application of the equipment can be adjusted in relation to saline intrusion patterns and flow conditions.
- The deepest sections of the river bed should include a limited number of mixing devices that are constantly running to prevent pooling of saline waters.

<u>Baffles</u>

- Flume tank experiments confirmed the general feasibility of a boom/skirt system in reducing saline intrusion for partial exclusion impoundments.
- The concept works through its self-sustained principle of utilizing the hydrostatic force of the upstream freshwater body for positive displacement of the saltwater.
- A successful operation in the field largely depends on the stability of stratification which requires an adjustable underflow opening via length or weight variation of the skirt.
- The system has to be carefully designed for field use since hydrostatic and hydrodynamic forces require considerable material strength and anchorage.
- A boom/skirt system does not provide a remediation strategy for stratified upstream sites in river bed depressions since these saline pools will be cut off by the falling halocline.

<u>Flushing</u>

- Flushing of the lower saline layers mainly effects the water volume in the vicinity of the withdrawal point while saline water upstream is not drawn by the release of water at the barrage.
- The additional shear stress introduced to the system is not sufficient to induce large mixing processes which could break up of stratification.
- The release of water at the barrage leads to dropping water levels in the impoundment which competes with demands for fish pass operation, amenity use of the impoundment and lock operation.

- Release of water is restricted to periods when the tidal regime allows sufficient head difference which can make a complete evacuation of the deteriorated water volume impossible
- Saline water tends to be trapped in bed depression upstream by the falling halocline which further limits dilution and replenishment possibilities

6.3 Implications

The research demonstrated that saline stratification is one of the major drivers for deteriorating water quality in estuarine impoundments. In terms of the overall aims of the SiMBa-Project, it would be advisable to avoid the future construction of partial exclusion barrages as amenity schemes. However, if the scheme is operational or environmental parameters do not allow a total exclusion system, remedial measures have to be included in the design. The limited success of flushing showed that this option is not advisable for future projects. Artificial mixing devices proved to be an efficient method for destratification of impoundments but also require a considerable amount of equipment and energy to be applied. Energy consumption could be reduced through implementation of a standby system that is triggered by online monitoring devices. The Tawe remediation scheme is at present undergoing a change towards this strategy following the results of this study and now included two continuous monitoring sites that are used to automatically adjust the application of diffusers.

Although the installation of a boom/skirt system provides an energy efficient alternative it shows limitations once saline propagation reaches riverbed depressions further upstream. These saltwater pools can only be flushed by the means of additional mixing equipment which should always be considered in conjunction with a boom/skirt system at the barrage. It has to be noted that all the information provided so far on boom/skirt systems as a remedial measure in estuarine impoundments relies on laboratory experiments. Therefore, a field installation would provide further information on issues relating to system performance and design requirements.

So far, the cost for installation, application and maintenance of remedial measures has not been considered in barrage projects and existing schemes have to allocate budgets for these additional costs. Therefore, remediation strategies have to be specifically addressed in the feasibility studies in order to be defined as a cost factor within the later contract. This will alter the cost-benefit analysis of proposed projects significantly and might lead to rejection of schemes as seen for the River Usk barrage (Murphy, 1993).

As for the construction and operation of total exclusion systems, the water quality implications are less crucial. The Tees impoundment experienced periods of low water quality during dry summer conditions which are characteristic to standing water bodies. This can not be mitigated by remediation devices since it is a matter of residence time rather than stratification. The impoundment proved to inherit a heat storage capacity which, in the light of future climate prospects (UK Climate Impacts Programme, 2002), will lead to an increased likelihood of DO deficiencies as a result of higher water temperatures and longer periods of low flows. A strategy for avoiding these conditions could lie in an improved flow regulation through increased use of upstream reservoir capacities (Malatre and Goose, 1995). However, this will compete with the existing demands of issued water abstractions and might therefore be difficult to realize as a management option.

6.4 Suggestions for further work

Several areas that require further research have been identified during the course of this study. The majority of which are extensions and improvements of the attempts undergone in the overall SiMBa-Project. The main topics in need of further analysis include:

- Installation of a prototype boom/skirt system to provide more information on its performance under field conditions.
- Research into the impact of upstream reservoir releases and different water abstraction techniques at the barrage on the mixing patterns and temperature variation of the impoundment.
- Assessment of online water quality monitoring as a tool to improve the performance of mixing devices in partial exclusion impoundments.
- Inclusion of meteorological predictions into operation and management of barrage schemes
- Introduction of online continuous monitoring at identified hot spots instead of sample monitoring throughout the whole impoundment.
- Extension of monitoring to downstream sites to determine the impacts of the barrage within the tidal reaches of the river.

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A Parameters

A.1 Environmental Variables

A.1.1 River discharge

A rivers hydrograph is a graphical presentation of variations in flow over time. The hydrograph consists of two components which are baseflow and runoff (Wilson, 1990). The baseflow is the portion of water that enters the river from groundwater and tends to be relatively constant compared to the runoff portion, which results from rainfall flowing overland and just below the surface towards the river. Several water quality variations can be explained by the relative influence of these two sources. Whereas the runoff component often contains high TSS, organic carbon, nutrients (N and P) and substances that increase conductivity of water, such as salts from surface wash off, the groundwater usually carries elements derived from rock weathering (SiO_2, Ca^{2+}, Na^+, Mg^{2+}, K^+) (Chapman, 1996).

A.2 Physical/Chemical

A.2.1 Temperature

Temperature was measured with the internal thermistors of the applied meters/probes which change their resistance output predictably with temperature variation. The algorithm to convert the signal into a reading of degrees Celsius (°C) is incorporated in the meter software. All instruments applied use functions which include the temperature reading to compensate for specific values of e.g., oxygen, conductivity and salinity.
During fieldwork with the handheld meters, the temperature reading of the oxygen meter was chosen to be recorded, since this parameter shows the highest temperature dependency and had priority in terms of water quality aspects.

A.2.1.1 Specifications of the applied probes

Combined Conductivity and Temperature probe Campbell CS547

Construction:	Three wire half bridge thermistor Belatherm 100K6A1
Temperature Range:	0° to 50°C
Accuracy:	± 0.4°C

Hanna Instruments Combined Temperature and DO Probe Handheld Meter HI 9145:

Construction:	Three wire half bridge thermistor
Temperature Range:	0° to 50°C
Accuracy:	± 0.5°C

YSI Combined Temperature and Conductivity Probe

Construction:	Three wire half bridge thermistor
Temperature Range:	-5° to 50°C
Accuracy:	± 0.4°C

A.2.2 pH

The pH was measured after the electrometric method (Cleceri, 1998), using a standard glass bulb electrode filled with buffered chloride solution of pH 7. This is connected to a reference electrode filled with Silver Chloride (YSI sensor) or Potassium Chloride (Hanna Instruments Sensor HI9025). Upon immersion, protons (H+) of the media interact with the glass electrode creating a potential gradient across the glass membrane which is compared to the reference electrode. Since the glass electrode buffer solution is invariant, the potential difference is proportional to the pH of the media and is converted into a mV signal applying a variation of the Nernst Equation. The pH sensors on the YSI sonde and the handheld meter (HI9025) were calibrated in regular intervals with buffer solutions of 4.01, 7.01 and 10.01. Automatic temperature compensation to 25 °C was applied to the signal resulting in a specific reading. Both instruments cover a range of pH 0-14 with an accuracy of +/- 0.04 pH for the YSI sensor and +/- 0.01 pH for the HI sensor.

A.2.3 Conductivity

The electrical conductivity of a solution is determined by applying an alternating current across an arrangement of electrodes while the voltage drop between them is measured as the resistance of the medium. The resistance is then converted in to Conductivity (K) in mS/cm according to the equation (Reeve, 1994;):

$$K = L / A * R$$

K=Conductivity
L=distance between electrodes (cm)

A=surface area of electrodes (cm^2)

R=resistance (Siemens, S)

The conductivity of a medium is temperature dependent and increases at a rate of 1.31 to 2% per °C (Bartram and Balance, 1996; YSI, 1999). The applied meters include temperature probes and are programmed to compensate for an increase 1.31% per °C (YSI Manual, 1996). The measured EC value is compensated to a temperature of 25°C and as this referred to as specific conductivity. However, the EC values presented in this study are always temperature compensated and referred to as Conductivity in order to comply with the associated references listed.

Specific Conductance = Conductivity * 100 / (Temp. – 25°C)* 1.91 + 100

Conductivity is an indicator for the presence of dissolved ions in the solution since the potential for carrying an electrical current increases with a higher content of salts. Therefore, it is a very important parameter to be considered in this research which is partly concerned with location and strength of seawater present in an impoundment. The following parameter of salinity is directly related to the measurement of conductivity resulting from a conversion of the initial signal into values of Salinity. Hence, an accurate calibration of the applied conductivity cells was maintained throughout the study using manufacturer calibration solutions.

A.2.3.1 Specifications of the applied probes:

<u>Campbell Scientific Conductivity Probe CS 547:</u>

Construction:	Stainless Steel passivated 316SS electrodes with DC isolation capacitors
EC Range:	Approx. 0.005 to 7.0 mS/cm
Accuracy	± 5% of reading 0.44 to 7.0 mS/cm
	± 10% of reading 0.005 to 0.44 mS/cm
Temp. Range:	0° to 50°C

<u>Hanna Instruments Handheld Conductivity Meter HI 9635:</u>

Construction:	Stainless Steel electrodes with DC isolation capacitors
EC Range:	Approx. 0.000 to 500 mS/cm
Accuracy	± 1% of reading
Temp. Range:	0° to 60°C

YSI Multisonde Probe YSI

Construction: Nickel electrodes with DC isolation capacitors

EC Range: Approx. 0.000 to 500 mS/cm

Accuracy ± 1% of reading

Temp. Range: 0° to 60°C

A.2.4 Salinity

Salinity values are derived from the conductivity measurement by applying algorithms found in Clesceri et al. (1998) to the temperature reading and the conductivity reading. The Salinity readings are given parts per thousand (ppt).

A.2.5 Dissolved Oxygen (DO)

As one of the most important parameters for aquatic life the DO content of water is always of great interest for authorities and researchers. It was measured in this research by the use of Clark Type sensors that are covered with Teflon membrane. The sensors measure a current proportional to the amount of oxygen reduced through the membrane at their cathode and anode. This current is proportional to the partial pressure of oxygen in the measured medium and is then concerted into a mg/l reading including temperature compensation through the internal temperature reading.

The Hanna Instruments steady state Sensor is continuously polarized during measurement using up the oxygen of the solution at all time and had to be supplied/provided with a minimum movement of the solution during the reading. This condition was given through the nature of measurement which took place in the flowing river channel. The YSI DO sensor minimize this depletion of oxygen through rapid polarization and depolarization (YSI, 1999) reducing the solution only 1/100th of the total measurement time. This was especially important for the deployment of the YSI sonde for longer time intervals in places where the movement of water was limited. All the sensors were calibrated according to the manufacturer instructions before every sample run and the YSI sensors were additionally equipped with a new membrane before deployment.

A.2.5.1 Specifications of applied probes

Oxyguard Model 420/Type 1:

Construction: Membrane covered galvanic cell, self polarising, self-temperature compensating, 2-wire measurement transmitter with 4-20 mA output

DO Range: 0-200%

Accuracy ± 0.1 mA

Temp. Range: 0° to 40°C

YSI Multisonde DO sensor :

Construction: Membrane covered rapid pulse Clark type sensor

DO Range: 0-200%, 0-20.0 mg/l

Accuracy ± 0.2% of reading

Temp. Range: 0° to 40°C

Hanna Instruments Handheld DO Meter HI 9145 with Temperature compensation:

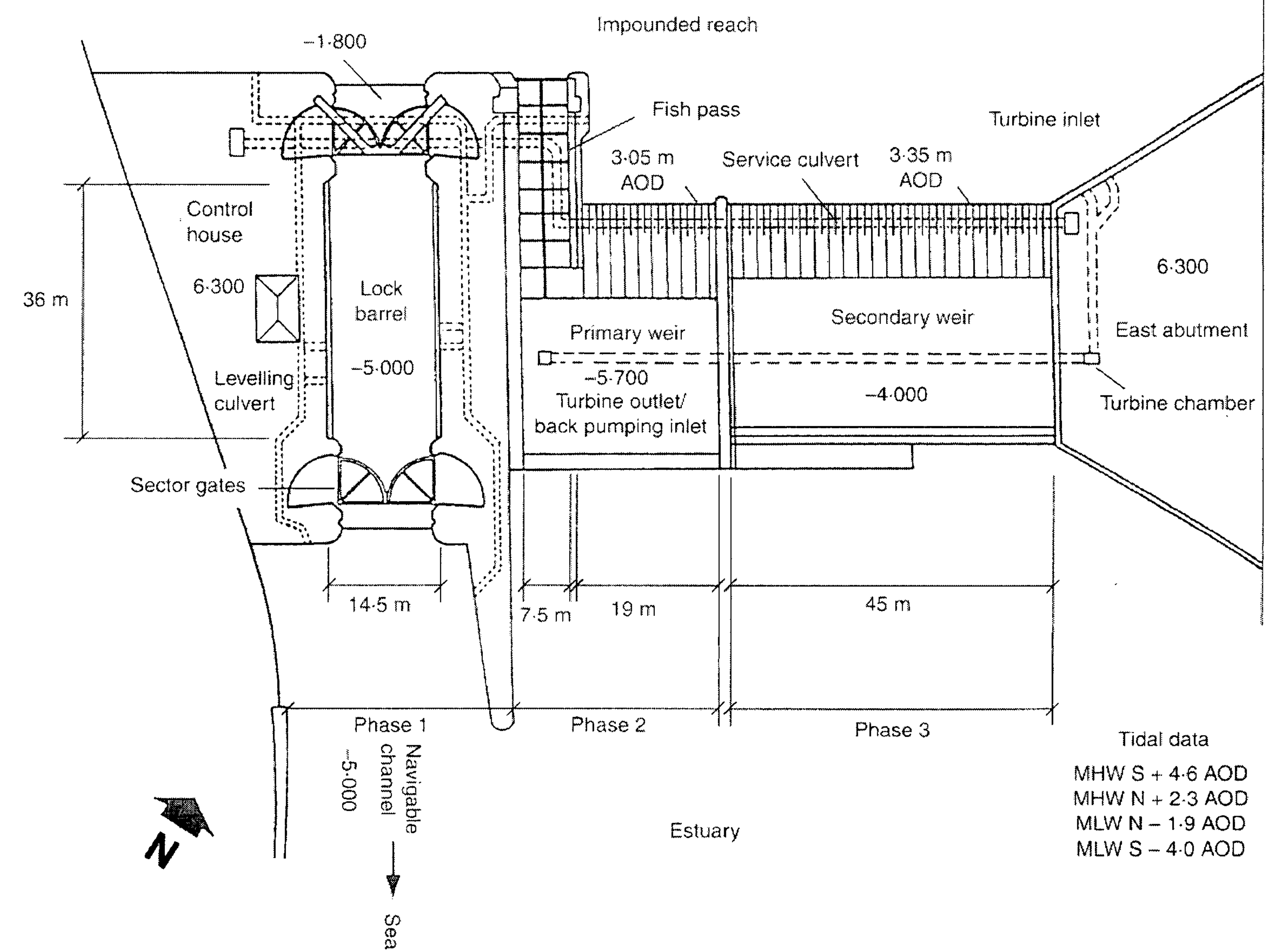
Construction: Membrane covered steady state Clark Type sensor

DO Range: 0-300%, 0-45.00 mg/l

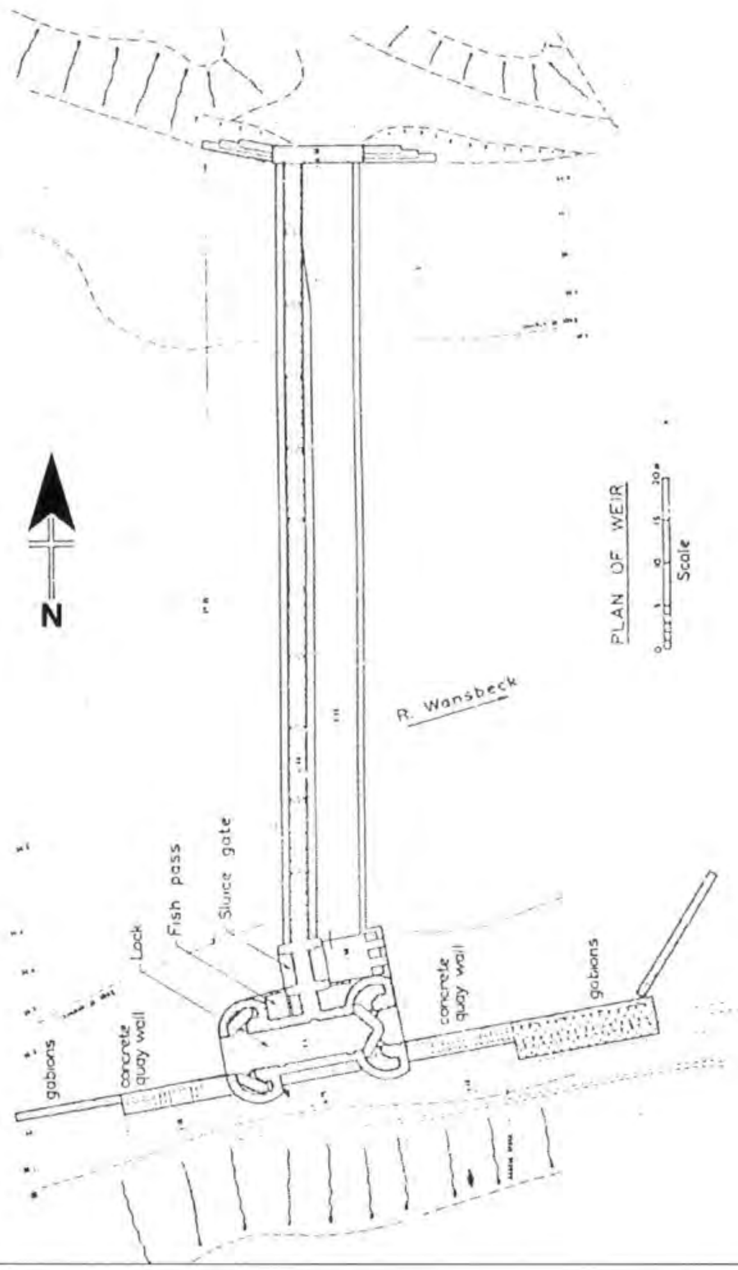
Accuracy ± 1.5 % of reading

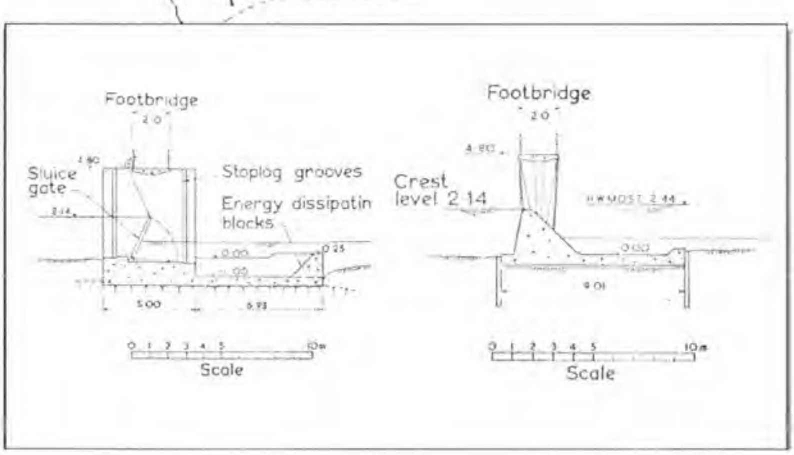
Temp. Range: 0° to 50°C

B Tawe Barrage plan



C Wansbeck Barrage plan





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