UPGRADING A TRICKLING FILTER WASTEWATER TREATMENT PLANT TO BIOLOGICAL NUTRIENT REMOVAL STANDARD

by

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A thesis submitted in partial fulfilment of the requirements for the degree of
Master of Environmental Studies

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September 1998

Statement

This thesis contains no material which has been accepted for the award of any other higher degree or graduate diploma in any tertiary institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person, except when due reference is made in the text of the thesis.

Scott Morgan

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10 December 1998

ABSTRACT

The Selfs Point Wastewater Treatment Plant in Hobart was upgraded to biological nutrient removal standard in early 1997. The upgrade incorporated the existing trickling filters and anaerobic digesters. The new nutrient removal capability is provided by the BioDenipho process, and is supported by prefermentation and alum dosing of the digested sludge sidestream.

This study reviews the development of wastewater treatment processes and the history of sewage treatment and collection in Hobart leading to this upgrade. All of the major processes at the Plant are reviewed as to how each impacts and is impacted by nutrient removal processes. Results from the first 18 months of operation of the plant following commissioning of nutrient removal are discussed, along with some of the factors affecting performance.

Prefermentation has been found to be critical in achieving good biological phosphorus removal. A solids residence time of 5 days in a combined activated primary tank and sidestream prefermenter system has provided good performance. Use of existing rock trickling filters in a combined carbonaceous removal and nitrification role has permitted a smaller BNR reactor volume and a plant with greater flexibility for treating high influent loads or flows The anaerobic digestion of phosphorus rich activated sludge led to a solubilisation of about 30% of the phosphorus entering the digesters. Alum dosing of the digested sludge prior to the belt filter press at 1 to 1 1:1 molar (Al:P) ratio has been effective at reducing return of phosphorus to the main process to less than 1%.

ACKNOWLEDGEMENTS

I would like to thank the following people for their help in achieving the completion of this thesis, though there are numerous others whose assistance has been appreciated.

Ray Farley has been a fund of practical ideas, with an agile mind and an enthusiastic attitude towards just about everything. Fran Gilroy, Tracey Pitman, Brenda Reinhardt and Enrique Quinteros have got through a mountain of work in the laboratory and suffered a range of odd requests to prepare some of the results presented in this study, as well as proofing some chapters of this thesis. Rob Pearson and Per Nielsen of ANI-Krüger have always been ready with many ideas, prepared to consider alternatives and trial options to test the Selfs Point Plant performance. Jill McBride and Sue Burrill at the Hobart City Council library have been very efficient and helpful in accessing reference documents for this study. Glenda Fahey's work in organising the administrative process at the Centre for Environmental Studies has also been appreciated.

Mostly, I would like to thank my partner Sharon Moore for proof reading and providing support and encouragement when it was needed most.

TABLE OF CONTENTS

	Page No.
ABSTRACT	i
ACKNOWLEDGEMENTS	ii
TABLE OF CONTENTS	iii
LIST OF FIGURES	ix
LIST OF TABLES	x
ABBREVIATIONS AND ACRONYMS	xi
CHAPTER 1: INTRODUCTION	1
1.1 Background	1
1.2 Aim and Objectives	2
1.3 Research Approach	3
1.4 Outline of Following Chapters	4
CHAPTER 2: HISTORY AND BACKGROUND	6
2.1 Brief History of Sewerage Reticulation	6
2.2 Early Forms of Sewage Treatment	7
2.3 Development of Wastewater Unit Operations	9
2.3.1 Settlıng Tanks	9
2.3.2 Fixed Film Biological Processes	9
2.3.2.1 Sand and Contact Filters	9
2.3.2.2 Trickling Filters	10
2.3.2.3 Rotating Biological Contactors	11
2.3.2.4 Biological Aerated Filters	11
2.3.3 Activated Sludge	12
2.3.3.1 Early Activated Sludge Systems	12
2.3.3.2 Oxidation Ditch	13
2.3 3.3 Sequencing Batch Reactor	13
2.3.3.4 Phased Isolation Ditch	13
2.3.3.5 Aeration Methods	14
2.4 Development of Wastewater Nutrient Removal Processes	14
2.4.1 Nitrification	14
2.4.2 Denitrification	15
2.4.3 Phosphorus Removal from Wastewater	16

2.4.3.1 Chemical Phosphorus Removal 16

2.4.3.2 Biological Phosphorus Removal 16

2.4.3.3 Microbiology and Biochemistry of EBPR 18

2.4.3.4 Prefermentation to Promote EBPR 19

2.4.4 Biological Nutrient Removal Plants in Operation 20

2.5 Disinfection 20

2.5.1 Introduction 20

2.5.2 Disinfection by Chlorine and Hypochlorite 21

2.5.3 Disinfection by Other Chemicals 22

2.5.4 Disinfection by Ultraviolet Light 22

2.6 History of Sludge Treatment 23

2.6.1 Sludge Digestion 23

2.6.2 Sludge Thickening and Dewatering 25

2.6.3 Sludge Disposal and Use 26

2.7 Chapter Summary 27

CHAPTER 3: HISTORY AND BACKGROUND IN TASMANIA 28

3.1 History of Sewage Collection and Treatment in Hobart 28

3.1.1 Early History of Sewage Collection 28

3.1.2 Later History of Macquarie Point Treatment Plant Catchment 29

3.1.3 History of Selfs Point Catchment 30

3.1.4 History of Sandy Bay Catchment 32

3.2 Legal and Policy Framework of Wastewater Treatment in Tasmania 34

3.2.1 Provision of a Sewerage System 34

3.2.2 Commonwealth Policy 35

3.2.3 State Government Environmental Legislation 36

3.2.3.1 Environment Protection Act 1973 36

3.2.3.2 Resource Management Planning System 36

3.3 Effluent and Biosolids Reuse Legislation and Policy 39

3.3.1 Effluent Reuse Legislation and Policy 39

3.3.2 Biosolids Reuse Legislation and Policy 40

3.4 Derwent Estuary Monitoring 40

3.4.1 Nutrient Monitoring of the Estuary 40

3.4.2 The Case for Nutrient Removal 41

3.4.3 Wastewater Effluent and Faecal Pollution in the Derwent Estuary 42

3.5 Chapter Summary 43

CHAPTER 4: THEORY OF WASTEWATER TREATMENT PROCESSES IMPACTED BY NUTRIENT REMOVAL 45

4.1 Introduction 45

4.2 Catchment Considerations 45

4.2.1 Catchment Phosphorus Reduction Strategies 45

4.2.2 Industrial Contribution to the Selfs Point Plant 46

4.2.3 Effect of Sewer System on Plant Influent 46

4.3 Primary Sedimentation 48

4.3.1 Introduction 48

4.3.2 Comparison of Selfs Point Plant Design with Typical Parameters 48

4.3.3 Length to Width Ratio for Rectangular Tanks 49

4.3.4 Preaeration of Wastewater 49

4.3.5 Impact of Solids Residence Time 50

4.3.6 Impact of Freshness of Wastewater 50

4.4 Trickling Filters 50

4.4.1 Introduction 50

4.4.2 Nitrogen Removal in Trickling Filters 51

4.4.2.1 Introduction 51

4.4.2.2 Organic Load 52

4.4.2.3 Hydraulic Load 53

4.4.2.4 Ammonium Load 53

4.4.2.5 Temperature 53

4.4.2.6 Alkalinity and pH 54

4.4.2.7 Oxygen Concentration and Mass Transfer 55

4.4.2.8 Trickling Filter Medium 55

4.4.2.9 Denitrification in Trickling Filters 56

4.4.2.10 Inhibition 56

4.4.3 Other Plants Using Trickling Filters in BNR Layout 56

4.5 BioDenipho™ - Phased Isolation Ditch Nutrient Removal Process 58

4.5.1 Introduction 58

4.5.2 Biological Phosphorus Removal 58

4.5.3 Nitrogen Removal 59
4.5.3.1 BioDenitro™ Process 59
4.5.3.2 Nitrification Usually Limiting in Nitrogen Removal 60
4.5.3.3 Temperature 60
4.5.3.4 Alkalinity and pH 60
4 5.3.5 Aerobic Sludge Fraction 61
4.6 Prefermentation 61
4.6.1 Description and Background 61
4.6.2 Types of Prefermenters 63
4.6.3 Temperature and Solids Residence Time 64
4.6.4 Effect of Hydraulic Residence Time 65
4.6.5 Impact of pH 65
4.6.6 Impact of Mixing 66
4.6.7 Other Factors Impacting on Prefermenter Performance 66
4.6.8 Effects of Prefermentation on Anaerobic Digestion 67
4.7 Secondary Clarification 67
4.7.1 Introduction 67
4.7.2 Measurement of Settleability and Bulking 68
4.7.3 Bulking, Foaming and Microthrix parvicella 69
4.7.3.1 Bulking 69
4.7.3.2 Foaming 69
4.7.3.3 Factors Affecting the Abundance of *Microthrix parvicella* 69
4.7.4 Denitrification in Secondary Clarifiers 71
4.8 Chemical Phosphorus Removal 73
4.8.1 Introduction 73
4.8.2 Use of Alum as a Phosphorus Precipitant 73
4.8.3 Location of Precipitant Dosing 74
4.8.4 Factors Affecting Precipitant Consumption 74
4.9 Disinfection: Impacts of Nutrient Removal 75
4.9.1 Introduction 75
4.9.2 Disinfection Mechanisms for Chlorine and Ultraviolet Light 76
4.9.3 Impacts of Nutrient Removal on Disinfection 77
4.9.3.1 Chemical Phosphorus Removal 77
4.9.3.2 Reduction of Ammonium Nitrogen in Effluent 77

4.9.3.3 Impacts of pH Changes 78

4.10 Thickening of Prefermenter and Waste Activated Sludges 78

4.11 Digestion and Dewatering 79

4.11.1 Introduction 79

4.11.2 Aerobic Stabilisation 80

4.11.3 Selection of Anaerobic Digestion at Selfs Point 80

4.11.4 Anaerobic Digestion 81

4.11.4.1 Background 81

4.11.4 2 Anaerobic Digestion and Biological Phosphorus Removal 82

4.11.4.3 Impact of Precipitant Chemicals on Anaerobic Digestion 83

4.11.4 4 Ammonia Return From Dewatering Digested Sludge 84

4.11.4.5 Impact of Increased Solids Content on Digester Performance 85

4.12 Vermiculture and Biosolids Quality 86

4.12.1 Background 86

4.12.2 Organic Substrate Factors Affecting Earthworm Productivity 87

4.12.3 Impact of Substrate pH 87

4.12.4 Impact of Ammonia Content 87

4.12.5 Heavy Metals and Phosphorus Precipitant Chemicals 88

4.12.6 Vermiculture Summary 89

4.13 Chapter Summary 89

CHAPTER 5: PROCESS DESCRIPTION AND METHODS 90

5.1 General Design and Catchment Description 90

5.1.1 Introduction 90

5.1.2 Design Criteria 91

5.1.3 Combined New Town and Sandy Bay Catchment Characteristics 92

5.2 Description of Upgraded Treatment Plant 92

5.2.1 Equipment and Process Design Summary 92

5.2.2 Inlet Works 94

5.2.3 Trickling Filters 97

5.2.4 Biological Nutrient Removal Process and Clarifiers 97

5.2.5 Disinfection 98

5.2.6 Sludge Thickening and Prefermentation 99

5.2.7 Digestion and Dewatering 99

5.2.8 Control, Instrumentation and Power Supply 100

5.3 Effluent and Biosolids Reuse 101

5.4 Process Monitoring and Experimental Methods 102

5.4.1 Sampling Methods 102

5.4.2 Laboratory Methods 103

5.4.3 Laboratory Scale Experimental Work 104

5.4.4 Online Instrumentation 105

5.4.5 Operator Log Books 107

5.5 Chapter Summary 107

CHAPTER 6: RESULTS AND DISCUSSION 109

6.1 Introduction 109

6.2 BNR Process Startup 109

6.3 Summary of Operating Results 110

6.3.1 Introduction 110

6.3.2 Effluent Ammonia Nitrogen 112

6.3.3 Effluent Total Nitrogen 113

6.3.4 Effluent Total Phosphorus 114

6.4 Catchment Impacts 115

6.4.1 Trends in Influent Characteristics 115

6.4.2 Major Events in the Catchment 116

6.4.2.1 Fire at Milk Processor 116

6.4.2.2 Fire at Sandy Bay Sewage Pump Station 116

6.4.2.3 Commissioning of Storage Tank at Milk Processor 117

6.5 Primary Sedimentation 118

6.5.1 Initial Primary Sedimentation Tank Performance 118

6.5.2 Settling Removal Efficiency with Activated Primary Operation 119

6.5.3 Activated Primary Tank Prefermentation Performance 121

6.6 Trickling Filters 123

6.6.1 Introduction 123

6.6.2 Oxygen and Alkalinity Limitations 124

6.6.3 Organic and Nitrogen Load Impacts 125

6.6.4 Sludge Production 127

6.7 BNR Reactors 128

6.7.1 Low Alkalinity and Nitrification 128

6.7.2 Phosphorus Content of WAS 129

6.7.3 Aeration Requirements for Phosphorus Uptake 131

6.7.4 Volatile Fraction of Mixed Liquor Suspended Solids 131

6.8 Secondary Clarifiers 132

6.8.1 Sludge Settling Properties 132

6.8.1.1 Selection of SVI Test 132

6.8.1.2 Variations of Sludge Settling Characteristics 133

6.8.2 Denitrification in Sludge Blanket 135

6.9 Prefermentation 135

6.9.1 External Carbon Dosing Trial 135

6.9.2 Replacement of Mechanical Screen 137

6.9.3 Dilution Water to Prefermenter 138

6.9.4 Prefermenter Efficiency 139

6.9.5 VFA Profile of Prefermenter Contents 141

6.9.6 Trial of DAF Waste in Prefermenter 142

6.10 Thickening 143

6.10.1 General Performance 143

6.10.2 Prefermenter Sludge Thickening 143

6.10.3 Waste Activated Sludge Thickening 144

6.11 Anaerobic Digestion and Dewatering 145

6.11.1 Phosphorus Release in Anaerobic Digesters 145

6.11.2 Chemical Dosing for Released Phosphorus 146

6.11.3 Digested Sludge Filtrate Ammonia Load on BNR Process 147

6.11.4 Impact of Digester Foaming 148

6.12 Chapter Summary 149

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS 150

REFERENCES 155

PERSONAL COMMUNICATIONS 178

LIST OF FIGURES

5.1 Map with Locations of Selfs Point Plant and Pipelines to Sandy Bay 90

5.2 Selfs Point Liquid Process Flow Diagram 95

5.3 Process Flowsheet for Upgraded Selfs point Plant in August 1998 96
6.1 Effluent Ammonia Nitrogen for March 1997 to August 1998 112
6.2 Effluent Total Nitrogen for March 1997 to August 1998 114
6.3 Effluent Total Phosphorus for March 1997 to August 1998 114
6.4 Online COD and Aeration Tank DO Readings after Milk Discharge 118
6.5 Hourly Profile of Trickling Filter Performance 124
6.6 Typical Diurnal Ammonia and Nitrate Concentrations in Tank C 129
6.7 Phosphorus Content of Waste Activated Sludge 130
6.8 Release of Phosphorus at Low Dissolved Oxygen 131
6.9 Changes in Mixed Liquor Volatile Suspended Solids 132
6.10 Mixed Liquor SVI from July 1997 to July 1998 134
6.11 Acetate Trial Comparing Effluent TP to Dosing Rate 136
6.12 Online TP Analyser in Tank C Including Acetate Trial Period 137
6.13 Change in Effluent Nutrients After Diverting Digester Overflow 149

LIST OF TABLES

4.1 Design Removal Efficiencies for Primary Sedimentation at Selfs Point 49
5.1 Influent and Effluent Parameters for Selfs Point Plant 91
5.2 Selfs Point Plant Technical Data Summary 93
5.3 Self's Point Plant Process Design Parameters at August 1998 96
6.1 Summary Influent and Effluent Concentrations Data 110
6.2 Chronology of Major Events since Startup of BNR Process 111
6.3 Primary Sedimentation Operation with One and Two Tanks 119
6.4 Removal Efficiencies of Usual and Activated Primary Tank Operations 120
6.5 Diurnal SCOD for Influent and Primary Effluent in Different Modes 122
6.6 Organic and Nutrient Removal Performance of Trickling Filters 123
6.7 Prefermenter VFA Concentrations 139
6.8 Estimated Typical Data on Prefermenter Operating Modes 140
6.9 Prefermenter Operating Data and Parameters 141
6.10 VFA Profile in Prefermenter Compared to Typical Values in Literature 142

ABBREVIATIONS AND ACRONYMS

ADWF	average dry weather flow
AmWWA	American Water Works Association
ANI-Kruger	ANI-Krüger Pty Ltd
ANZECC	Australian and New Zealand Environment Conservation Council
A/O™	Anaerobic/Oxic process
APHA	American Public Health Association
APT	activated primary tank
ARMCANZ	Agricultural Resource Management Council of Australia and New Zealand
AS	Australian Standard
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing of Materials
ATAD	Autothermal thermophilic digestion
AWWA	Australian Water and Wastewater Association
AWT	Australian Water Technologies Pty Ltd
BAF	biological aerated filter
BFP	belt filter press
Bio-P	biological phosphorus
BNR	biological nutrient removal
BOD	biochemical oxygen demand
BOD_5	5-day biochemical oxygen demand
CASS™	Cyclic Activated Sludge System
CD-ROM	compact disc - read only memory
CEE	Consulting Environmental Engineers Pty Ltd
cfu	colony forming units
COD	chemical oxygen demand
CSSE	Committee of Sewerage and Sanitary Engineers
CSO	Colonial Secretary's Office
DAF	dissolved air flotation
DCHS	Department of Community and Health Services
DELM	Department of Environment and Land Management
DNA	deoxyribonucleic acid
DO	dissolved oxygen
DoE	Department of Environment

DPIF	Department of Primary Industry and Fisheries
DSSV	diluted settled sludge volume
DSVI	diluted sludge volume index
EBPR	enhanced biological phosphorus removal
EIP	Environment Improvement Programme
EMPCA	Environmental Management and Pollution Control Act 1994
EP	equivalent population
EWS	Engineering and Water Supply (South Australia)
FID	flame ionisation detection
FISH	fluorescent in situ hybridisation
FSS	fixed suspended solids
HAc	acetic acid
HCC	Hobart City Council
HMCA	Hobart Metropolitan Councils' Association
HRT	hydraulic residence time
IAWQ	International Association on Water Quality
IDAL	intermittently decanted aerated lagoon
ISO	International Standards Organisation
LUPAA	Land Use Planning and Approvals Act 1993
ML	megalitres
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
NFR	non-filterable residue
NO_x^-	oxidised nitrogen anions (nitrite and nitrate)
NRC	National Research Council (USA)
NSW EPA	New South Wales Environment Protection Authority
NTA	nitrilotriacetic acid
PAO	phosphorus accumulating organism
PDWF	peak dry weather flow
PHB	poly-β-hydroxybutyrate
PLC	programmable logic controller
Proc	Proceedings
PSRP	Process to significantly reduce pathogens
PWWF	peak wet weather flow

RAS	return activated sludge
RBC	rotating biological contactor
RBCOD	readily biodegradable chemical oxygen demand
RDT	rotary drum thickener
RMPS	Resource Management and Planning System
RNA	ribonucleic acid
RPDC	Resource Planning and Development Commission
SBR	sequencing batch reactor
SCADA	supervisory control and data acquisition
SCOD	soluble chemical oxygen demand
SCVFA	short chain volatile fatty acid
SDAC	Sustainable Development Advisory Council
SEMF	Scientists, Engineers, Managers and Facilitators Pty Ltd
SPPA	State Policies and Projects Act 1993
SRT	solids residence time
SS	suspended solids
SSSV	stirred settled sludge volume
SSV	settled sludge volume
SSVI	stirred sludge volume index
STIP	Siepmann und Teutscher GmbH
STP	sewage treatment plant
SVI	sludge volume index
TCOD	total chemical oxygen demand
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TP	total phosphorus
TSS	total suspended solids
UASB	upflow anaerobic sludge blanket
USA	United States of America
USEPA	United States Environment Protection Agency
UV	ultraviolet light
VFA	volatile fatty acids
VSS	volatile suspended solids
WAS	waste activated sludge

WEF	Water Environment Federation
WPCF	Water Pollution Control Federation
WTP	wastewater treatment plant
WWTP	waste water treatment plant

CHAPTER 1: INTRODUCTION

1.1 Background

Throughout history, whenever human settlements reached a size of more than a few hundred individuals, problems associated with treatment and disposal of the increased volume of wastewater have invariably arisen. Wastewater, particularly that carrying faecal material, can readily contaminate surface water supplies, be the source of many diseases and result in organic and nutrient loads greater than the local environment's receiving capacity

There is evidence of early wastewater treatment systems in the Mediterranean over two thousand years ago. However following Roman times there was little systematic treatment of wastewater and the collection and treatment of sewage did not occur widely until the 19th Century. Discovery of the link between contaminated water supplies and disease and the subsequent public pressure gave impetus to the development of processes to treat wastewater prior to disposal.

As well as being a major contributor to disease, inadequately treated sewage can have severe environmental impacts by reducing dissolved oxygen levels and causing eutrophication due to increased concentrations of nutrients. Early in the 20th Century, wastewater treatment plants focused on reducing the organic content of sewage, which did improve the dissolved oxygen levels in the receiving waters. The problems associated with the nutrients, nitrogen and phosphorus, became apparent in the 1950s and 1960s and resulted in development of nutrient removal processes.

Following a number of studies and as a result of stricter environmental controls set by the Tasmanian State Government, the Hobart City Council made the decision to treat sewage from the Sandy Bay catchment at the existing Self's Point Wastewater Treatment Plant and upgrade the Plant to biological nutrient removal standard.

This study both examines the operation of unit processes and evaluates the effectiveness of the new treatment facilities at Selfs Point. The Self's Point Plant was selected as the subject of this study because it is the first Tasmanian wastewater treatment plant designed for both biological phosphorus and nitrogen removal, it is the first major plant using phased isolation ditch treatment technology commissioned in Australia and it combines trickling filters and activated sludge in a process format used in only a few other plants around the world.

1.2 Aim and Objectives

The aim of this thesis is to investigate the performance of biological nutrient removal in the Self's Point hybrid trickling filter/activated sludge wastewater treatment process and assess the impact of changes in process parameters on effluent quality. In undertaking this study it examines the historical context of the Self's Point Plant, the processes incorporated into the upgraded plant design and theories associated with the biological removal of the nutrients nitrogen and phosphorus.

The objectives of the thesis are:

1. To review literature in the field of biological nutrient removal, providing a summary of the most recent developments in theory as they apply to the processes being used at Self's Point;
2. To discuss the history of wastewater collection and treatment and assess in more detail the developments in the Hobart City Council area, with particular reference to the Self's Point Plant;
3. To describe the treatment processes used at Self's Point and the methods used to measure the performance of the plant;
4. To present results and discuss the limitations of using a full scale operating facility to undertake experimental work;
5. To analyse the impacts of a number of process changes, variations in influent characteristics and the incorporation of trickling filters into a biological nutrient removal treatment train; and
6. To draw conclusions about the impacts of changes in process control parameters and influent variables on treatment plant performance and make recommendations regarding future control strategies and areas for further research.

The process areas considered within the scope of the thesis are:

i) Use of one primary sedimentation tank, when two are available;

ii) Nitrification performance of trickling filters at different temperatures;

iii) The impact of the return sidestream from anaerobic digestion;

iv) Impact of chemical precipitant dosing on effluent phosphorus concentration;

v) Impact of carbon source dosing on phosphorus removal;

vi) Impact of high milk content wastewater loads on the plant;

vii) Low organic load on plant after shutdown of dairy production;

viii) Modifications to dairy wastewater discharge times;

ix) Alteration of primary sedimentation tank to activated mode; and

x) Effect of elutriation water on prefermenter performance.

The focus of the thesis is on nutrient removal performance, but other aspects of plant operations are included to provide an improved understanding of the degree that external factors affect control strategies.

1.3 Research Approach

The methodology used in developing this thesis incorporated literature review and analysis, personal communications with practitioners in the field of wastewater treatment, site inspections of several other facilities and conference attendance. The experimental results were obtained through field measurements and laboratory analysis, online instrumentation, plant log books, data stored on the plant control computer and information from plant operators and laboratory staff.

The literature review included an extensive study of:

- Texts and design manuals describing the theory and history of wastewater treatment;
- Journals specialising in water quality and wastewater treatment, notably *Water Science and Technology*, *Water Environment Research* and *Journal of the Water Pollution Control Federation*;
- Proceedings from a number of recent conferences specialising in biological nutrient removal, both in Australia and overseas;
- Hobart City Council (HCC) reports, files, by-laws and policy documents;
- Design documentation and equipment specifications supplied by ANI-Krüger Pty. Ltd., including Self's Point operation manuals;
- Brochures and other data from other wastewater treatment plants;
- Notes from specialist courses on biological nutrient removal and prefermentation;

- Relevant Tasmanian and Commonwealth government legislation and policy documents; and
- Websites of some water and wastewater organisations.

A total of sixteen other nutrient removal wastewater facilities in Tasmania, Queensland, New South Wales, Denmark, Germany and Luxembourg were inspected, including four plants using the Bio-Denipho™ process, which is the nutrient removal process used at the upgraded Self's Point Plant. During each visit discussions were held with site managers and operators, when available, to discuss plant performance.

The BNR3 conference on biological nutrient removal held in Brisbane late in 1997 was attended and discussions were held with several other conference attendees regarding various aspects of nutrient removal.

Numerous discussions have been held with employees of both ANI-Krüger Pty Ltd. and I. Krüger A/S regarding the Bio-Denipho™ process and factors which are expected to affect its performance.

Frequent and regular contact with the Selfs Point Plant site personnel and laboratory staff provided additional information and observations not noted in formal plant records.

1.4 Outline of Following Chapters

The early chapters are a summary of the literature review and background information obtained from various sources as described.

A history of the development of wastewater collection and treatment processes is presented in Chapter 2. Some of the pressures driving the construction of systems are discussed, along with a summary of the advances in wastewater treatment processes with particular emphasis on the biological nutrient removal and phosphorus removal

Chapter 3 provides an overview of the history of wastewater in Hobart and of the Self's Point Plant including the background to the upgrade to nutrient removal standard. The chapter commences with a description of the history of sewage collection and treatment in Hobart. The recent history of the Self's Point Plant and the Sandy Bay Sewerage Project is presented in more detail. The legislative and policy framework relating to wastewater treatment in Tasmania is also described.

The focus of Chapter 4 is on the theory and findings of recent research relating to the process unit operations utilised at Self's Point. The results of both laboratory and full scale work is described. While all major unit operations are reviewed, the major emphasis is on how each process affects nutrient removal within the treatment plant.

A description of the processes and equipment used at the upgraded Self's Point Plant and the methods used to measure the performance are presented in Chapter 5. The description includes information on the layout, process design parameters, control systems and required effluent quality. The analytical and sampling methods, details of the online monitoring equipment and other data sources are also presented.

Due to the interrelatedness of the results and the associated interpretation and discussion, these are presented together in Chapter 6. This chapter covers the general plant performance, aspects of the nitrogen removal processes and the effects of external impacts on effluent quality. The phosphorus removal performance of the plant, both chemical and biological, with major emphasis on the effects of modifications to provide a readily biodegradable carbon source is reported and discussed, with reference to the theoretical considerations presented in Chapter 4.

Chapter 7 provides a brief summary of the findings of the research and presents conclusions based on interpretation of the results. Recommendations are made in relation to some of the factors which should be considered in future plant operations and further areas for research, particularly as they apply to the plant configuration used at Self's Point.

CHAPTER 2: HISTORY AND BACKGROUND

2.1 Brief History of Sewerage Reticulation

Channels and pipes have been used to transport stormwater and sanitary wastewater since ancient historical times. One of the earliest engineering works which is considered to have carried wastewater rather than drinking or irrigation water is a sewer arch at Nippur in India. The construction has been dated at about 3750 BC (Babbitt 1953). There is evidence at Knossos in Crete that the palace of King Minos incorporated cleverly designed plumbing and latrines, including an earthenware pan and flushing reservoir, at about 2000 BC (Kilroy 1984).

One of the major sewers of Rome, the Cloaca Maxima, was constructed in the period 616 - 578 BC to carry stormwater and sanitary waste as well as draining a swamp (Hodge 1992). It was a sewer for the Roman Forum and was still in use earlier this century (Fair & Geyer 1954). At about 300 BC water was used for flushing sewers in Rome and a portion of the water brought to Rome by aqueduct is noted as being reserved for this use at the time of Frontius (Goldfinch 1997). A water-borne system for toilet waste was used in Ephesus, Turkey during Roman times, the ruins of which have recently been restored (pers. obs., 1991)

Following the fall of the Roman Empire the systematic collection of sanitary wastewater is not noted as having recurred until the early 1800s. Disposal of human waste in urban areas often consisted of emptying chamberpot contents out of the window into the street. There were some efforts by King Henri II of France in about 1550 to get the Parliament of Paris to construct sewers in Paris. However neither the King nor the Parliament wanted to pay for them and a sewer system was not constructed until the nineteenth century during the time of Napoleon III (De Camp 1963).

In Europe, drains for the transport of storm runoff during wet weather were used in many towns prior to 1800, but the discharge of faecal wastes into these systems was usually forbidden. In 1815 regulations were changed so that excreta was thenceforth legally permitted to enter sewers in London (Babbitt 1953). A report prepared by the Clerk of the London County Council in 1913 (cited in Escritt 1965:20) considered that this change in regulations resulted from the advent of the water closet, which produced large volumes of wastewater, in excess of the capacity of the existing cesspools. By 1847 it was compulsory in London for wastewater drains to connect to the sewer (Escritt 1965). There appears to have been a high degree of resistance to constructing sewers for waste disposal due to capital costs. It was cheaper to use the existing stormwater collection drains to transport wastes rather than build separate sanitary sewerage. Thus the first sewerage drains were "combined" systems, conveying both sanitary waste and stormwater. Sydney's first sewers, built in 1854, were the beginning of a combined drainage system (Bowen 1998).

Separate sewers for sanitary waste were not constructed until the latter half of the nineteenth century, as the public health and environmental impacts became better understood. The connection between disease outbreaks and contamination of drinking water supplies was noted by John Snow in 1853 (Isaac 1996). As a result of a study in London he found that cholera in a particular part of the city was dramatically reduced by removing the handle from a manual water pump. This isolated the water supply which was the source of the disease outbreak. He noted that untreated wastewater from a nearby boarding house was close to the well and may have been the cause of the contamination (Isaac 1996a). This and other work on disease transmission in the middle of the 1800s led to campaigns to have sanitary sewers installed in place of the cesspits prevalent at the time. In England, Sir Edwin Chadwick was an avid crusader for separate sanitary sewers and he used the slogan "the rain to the river and the sewage to the soil" (NRC et al. 1996: 17). In the USA, for example, the first such separate system was constructed in Memphis, Tennessee in 1880 (Babbitt 1953).

2.2 Early Forms of Sewage Treatment

The sewers, once built, transported wastes from the point of creation, but merely moved the problem to the nearest water body. This diversion of the effluent had the effect of contaminating the receiving water and overloading its natural capacity to treat wastewater. This situation was apparent near many cities and Samuel Taylor Coleridge was moved to pen the following about the city of Köln in Germany in 1797 (cited in Vesilind, Pierce and Weiner 1989: 153):

> In Köln, a town of monks and bones
> And pavements fanged with murderous stones
> And rags, and bags, and hideous wenches;
> I counted two and seventy stenches,

All well defined, and several stinks!

Ye Nymphs that reign o'er sewers and sinks,
The river Rhine, it is well known,
Doth wash your city of Cologne;
But tell me Nymphs! What power divine
Shall henceforth wash the river Rhine?

Similarly in Boston the impact of wastewater on the local water body was felt most obviously through odour and the Board of Health in 1885 put it that "large (surrounding) territories were at once, and frequently, enveloped in an atmosphere so strong as to arouse the sleeping, terrify the weak, and nauseate and exasperate everybody" (Clark 1885 cited in Fair and Geyer 1954: 7). Thus pressure increased for the treatment of wastewater prior to its release to the environment. By the end of the 19th Century the British Royal Commission on Sewage Disposal proposed a goal of final effluent quality for treatment plants of 30mg/L suspended solids and 20mg/L biochemical oxygen demand (BOD) (Bitton 1994).

The first sewage treatment systems consisted of the use of raw sewage on land or in underground soakage systems. There is evidence that wastewater was treated by use of a soakage system in ancient Athens and in the Sicilian town of Aidone by use on a sewage farm, both of these occurring several centuries BC. There are indications that the type of system at Aidone was concurrently in use at a number of other locations in the Mediterranean (Hodge 1992). Such treatment of wastewater generally lapsed and did not occur again until the advent of sewers collecting wastewater in the 1800s. Several cities in the late nineteenth century either had sewage farms or used sewage for irrigation and over 50 sewage farms were providing land treatment in England in 1875 (NRC et al. 1996). Cities employing sewage farms included Berlin, Paris, Mexico City and Melbourne (Metcalf & Eddy 1979) The Werribee Sewage Farm commenced operation in August 1897 and a significant portion of Melbourne's sewage is still treated there (Swinton 1998).

However as urban areas expanded and water consumption increased many of these systems were abandoned. With the advent of compact treatment processes the pressures to develop the land or the absence of further land to expand the operation led to the decline of the use of these systems. If wastewater is used for irrigation one hectare can treat the sewage from 25-125 persons depending on the site conditions

(Novotny et al. 1989). Thus the area of land required for large towns and cities was in the hundreds or thousands of hectares and the volumes of wastewater exceeded the land available for sewage treatment. In Sydney a sewage farm operated at Botany Bay from 1882 to 1916, following which the sewage was diverted to the Malabar outfall The farm worked well until after the turn of the century when it became overloaded due to the combination of increasing volume of wastewater and the inability to expand the area under irrigation at the site (Aird 1961).

2.3 Development of Wastewater Unit Operations

2.3.1 Settling Tanks

The first vessel-based process to treat wastewater consisted of allowing solids to settle in tanks, this system being developed in the 1850s (Metcalf & Eddy 1979). The separation of the settleable solids soon led to the development of digestion processes to treat the sludge as discussed in Section 2.6. The settling or sedimentation process was later supplemented with chemical treatment such as lime dosing, which became prevalent during the 1880s in Europe and the USA. Chemical treatment peaked in England during that decade (Babbitt 1953). Use of chemicals in treatment processes declined following the advent of biological processes which achieved better effluent quality at lower cost

2.3.2 Fixed Film Biological Processes

2.3.2.1 Sand and Contact Filters

The first documented form of biological treatment incorporating a specific process unit was that of sand filters. While the sand did perform a filtering function it also provided a surface for bacteria to grow on and was thus was an early form of fixed film biological treatment process. In 1865, Alexander Mueller noted that microorganisms in a filtration column could purify sewage (Peters and Foley 1983). During the years 1868-70 Sir Edward Franklin developed the use of intermittent sand filters to treat sewage in England (Babbitt 1953). In the early 1900s the majority of Sydney's wastewater was treated with sand filter beds (Aird 1961).

The principle of a surface to support bacteria was further developed in contact filters (or beds), which were a form of fill and empty tank containing a coarse media such as stones (Metcalf & Eddy 1979). This system was relatively slow, due to the length of time the tank had to remain empty for the process to be prevented from going septic

(Babbitt 1953). The process was modified to create trickling (also known as sprinkling or percolating) filters, where the sewage flows down over a material providing a surface on which bacteria can grow (Fair and Geyer 1954).

2.3.2.2 Trickling Filters

The first documented installation of a trickling filter was in Salford, England in 1893 (Metcalf & Eddy 1979). An early example in Australia of a trickling filter installation was at Parramatta, with the plant commencing operations in 1910 (Bowen 1998). The trickling filter became the preferred secondary treatment process for many years due to its low maintenance requirements, consistent performance and ability to withstand shock loads, however odour and filter fly (principally *Psychoda spp.*) were often nuisances (Imhoff and Fair 1956; Poppelen 1998).

Further development of the trickling filter took place, with the high rate process acknowledged as having first been used in the USA in 1936. This entailed higher loading on the trickling filter than had previously been used and allowed more compact plants to be built (WPCF and ASCE 1977). The use of trickling filters to provide tertiary nitrification following a BOD only secondary removal process was developed in the 1980s (Reardon 1995)

Early trickling filters usually used rock media, though coal, coke, blast furnace slag, broken bricks and wooden laths were also employed (Imhoff and Fair 1956). The maximum height was limited to about two metres to prevent crushing of the lower layers of media and also to permit adequate hydraulic flow to occur without ponding of liquid being caused by clogging from biomass buildup.

The development of plastic media similar to that used in chemical engineering mass transfer operations permitted the use of higher towers, as a result of the light weight of the plastic. The greater specific surface area and increase in hydraulic capacity made it possible to construct a more compact process unit (WPCF and ASCE 1977). Hobart City Council's Macquarie Point Wastewater Treatment Plant in Hobart has tower trickling filters with randomly packed plastic media, and the relatively small footprint of this process was one of the reasons that it was selected, due to the limited area available (Chee Liew, pers. comm., 1996). There are still many plants using trickling filters, though due to the limitations of this type of unit operation in meeting tight nutrient effluent criteria and relatively high capital costs few new plants incorporating trickling filters are being built today.

2.3.2.3 Rotating Biological Contactors

Other forms of the aerated fixed film process have been developed and one of the major alternatives to the trickling filter is the rotating biological contactor (RBC). This system consists primarily of discs connected to a rotating shaft. The discs are partially submerged and pass alternately through the wastewater and into the air every revolution of the shaft. While the idea for a RBC was patented as early as 1900 (Peters and Foley 1983), it was not until the early 1960s that the first rotating biological contactors were installed in Germany following work at the University of Stuttgart (Horan 1990). While more compact, this system has similar limitations to the trickling filter in terms of controlling dissolved oxygen levels to remove nutrients and meeting high effluent requirements. It appears that few of these systems are presently operating in Australia, though some are used in Australia's Antarctic stations (Sharon Moore, pers. comm., 1998).

2.3.2.4 Biological Aerated Filters

New submerged fixed film processes have been developed which can remove nutrients biologically. A generic term for these processes is "biological aerated filters" (BAFs). The original patent for this type of process was taken out in Canada in 1974 (Smith and Hardy 1998). The process involves small diameter plastic beads or granular materials (typically 3-6 mm in diameter), which provide a carrier surface for bacteria. The beads are submerged and air is introduced into the vessel containing the beads (M'Coy 1997). The nature of the process permits very high sludge loadings (15-20kg/m^3) and thus the construction of very compact plants. A recently constructed plant near St. Austell in England was estimated to require only 40% of the land area that a conventional plant would have occupied (Anon. 1994).

At present the capital cost of the BAF plants is more expensive than the equivalent activated sludge processes, but particularly where land prices are high or sites are very limited in size BAF plants are becoming competitive (M'Coy 1997). Over one hundred of these plants are in operation world-wide (Smith and Hardy 1998). The BAF processes Biocarbone™ (downflow) and Biostyr™ (upflow) were considered as potential options for the Sandy Bay Sewerage Project, particularly for confined sites in the Sandy Bay area. With the selection of the Self's Point site, the area required for the process was less of a concern and these options were discarded due to higher capital and operating costs (AWT 1993).

2.3.3 Activated Sludge

2.3.3.1 Early Activated Sludge Systems

Early experiments into the aeration of sewage were performed by Angus Smith in 1882, but he found the resulting sludge "difficult to precipitate" (Martin 1927: 2). Martin (1927) provides a summary of the research into aeration of sewage in the period 1882 to 1926. Some of the experiments were able to generate clear effluent, but did not result in a "practicable method of treating sewage under actual working conditions" (Martin 1927: 13). The activated sludge process using settled suspended bacteria to inoculate incoming sewage was first developed in England by Ardern and Lockett at the Davyhulme Treatment Works in Manchester (Ardern and Lockett 1914). The term "activated sludge" is used because it involves "the production of an activated mass of micro-organisms capable of aerobically stabilising a waste" (Metcalf & Eddy 1979: 430). The initial process developed by Ardern and Lockett was a batch process known as the fill and draw method with aeration and settling occurring in the same tank (Gray 1990).

The initial batch process was commercialised as a continuous within a short period by Jones and Atwood of Stourbridge (Urbain et al. 1998; Martin 1927). The use of activated sludge processes grew steadily after the First World War in both the UK and USA (Martin 1927). The first activated sludge plant on continental Europe was constructed in Denmark during the early 1920s (Henze et al. 1995). An activated sludge process was rapidly installed at Folly Point in North Sydney in 1919 in response to neighbours' complaints about odours from the existing trickling filter plant (Aird 1961).

Activated sludge processes were only used in a few large wastewater treatment plants until the 1950s (Arundel 1995). The process had the advantages of using less land and generating less odour than trickling filters, but it required greater knowledge and more attention to operate successfully (Martin 1927). Much of the development of design equations for activated sludge incorporating mass balance and microbial growth kinetics occurred in the 1960s (WEF 1977), while more recent research work has focused on nutrient removal processes. Activated sludge is considered to be "the most widely used biological process for wastewater treatment" (Urbain et al. 1998: 223).

2.3.3 2 Oxidation Ditch

One variety of the activated sludge process, the oxidation ditch, was first used in Europe during the 1920s (Randall, Stensel and Barnard 1992a). Martin (1927. 194-5) describes how the "endless channel" tank system (i.e. oxidation ditch) fell out of favour in England at this time, with better performance being obtained by "once-through tank" systems

The oxidation ditch process did not gain widespread acceptance until the 1950s following the development by Pasveer of the first extended aeration activated sludge system in the Netherlands (Pasveer 1959). Now often described as the Pasveer oxidation ditch it is based on a single reactor for biological treatment and secondary setting with no primary sedimentation (Gray 1990). With the advent of this development the ease of activated sludge systems began to rival that of trickling filters and contributed to the acceptance of activated sludge-based processes (WEF 1996)

2.3.3.3 Sequencing Batch Reactor

The fill-and-draw process initially used by Ardern and Lockett did have some advantages, such as only requiring one reactor and no return sludge pumping, but in 1917 Ardern outlined some of its disadvantages compared to continuous flow plants, including the additional head requirement, increased operator attention and greater potential for diffuser blockages (Martin 1927). While the prevailing view in the early years of activated sludge was that the fill and draw system was more efficient, the continuous system was favoured due to its greater practicality (Martin 1927) Thus while sequencing batch reactors have been used for much of this century, more widespread use only began to occur in the 1970s with the advent of microprocessor controllers and improved instrumentation hardware, negating the need for close operator attention (Randall, Stensel and Barnard 1992a).

2.3.3.4 Phased Isolation Ditch

The phased isolation ditch is a process in which multiple parallel oxidation ditches are alternately fed during a timed cycle. The first development of the process occurred in the 1970s by the Krüger company based in Denmark, which patented the BioDenitro™ arrangement in 1976 (WEF and ASCE 1992). In an assessment of a number of phased isolation ditch plants in Denmark Tetreault et al. (1985) found this type of process could reliably achieve good effluent quality at net present values significantly less than oxidation ditch or SBR systems of equivalent capacity. Tetreault et al. (1985) also noted that at the time of the study 62 full scale plants were operating in Denmark.

2.3.3.5 Aeration Methods

In the activated sludge process delivery of air to the biomass is crucial. Diffused aeration has been used since the turn of the century, with the refinement of porous tiles occurring in 1914 (Martin 1927). Diffused aeration became the most popular method in the 1930s. Fouling problems and maintenance difficulties led to surface aeration methods becoming more popular, although they consumed more energy. With energy costs rising significantly in the late 1970s and aeration accounting for 50 to 90% of the total process power demand at an activated sludge plant diffused air systems again became prevalent (WEF and ASCE 1992).

Mechanical agitation and aeration systems were developed due to the blockages of diffusers at several locations in England in the early 1920s (Martin 1927). Vertical and horizontal shaft systems and draft tubes were trialled and installed. The horizontal shaft system in use on some tanks at Sheffield (Martin 1927) was an early precursor to the aeration rotors which are presently often used on oxidation ditch plants. In the United Kingdom in 1990 it is noted that at plants of greater than 10,000 equivalent population (EP) about 50% of the installed EP capacity used surface aeration and 50% diffused air, with the latter predominating on larger plants (Gray 1990).

2.4 Development of Wastewater Nutrient Removal Processes

2.4.1 Nitrification

Research by Schloesing and Muntz (1877) into the treatment of sewage in a sand column noted that ammonia was converted to nitrite and nitrate. By introducing chloroform vapour the reaction could be stopped and they concluded that the process was biological in nature. Fair and Geyer (1954) provided the prevailing view towards nitrification in the 1950s. They noted that prior to the concept of biochemical oxygen demand (BOD) being developed there was strong emphasis on the conversion of organic nitrogen and ammonia to nitrate in wastewater treatment, but that this emphasis had declined and that low BOD effluent was considered of much greater importance at the time than ensuring nitrification of the effluent. They considered nitrification should only be performed in a few cases, though provided a 1928 example of partial nitrification in a low loaded trickling filter.

It was not until a decade later that nitrification again became an important consideration in treating wastewater. On many activated sludge plants the process was operated with high sludge wasting rates to ensure nitrification did not occur. This reduced aeration needs and resulted in lower electrical power consumption. In the United Kingdom for example it was not a requirement to nitrify for discharge to inland waters until 1976 (O'Neill and Horan 1994). Nitrification has now become a recognised requirement for wastewater treatment plant effluents being discharged to inland waters and enclosed bays and estuaries (USEPA 1993).

2.4.2 Denitrification

The loss of nitrogen from soils was investigated during the middle of the 1800s. The first person to suggest that bacteria were responsible for the destruction of nitrate was Meusel in 1876 (Payne 1981). The term "denitrification" was coined by Gayon and Dupetit in 1882 to describe the production of gas from bacteria using nitrate under anaerobic conditions (Payne 1981). Similar to the development of processes to nitrify wastewater, the pressure to discharge a more highly denitrified effluent did not occur until the 1960s and it is only in the past thirty years that denitrification process designs have been developed in response to environmental and public health concerns. High nitrite and nitrate levels in drinking water have been shown to have adverse health effects (ANZECC 1992) and high nitrogen loads can contribute to eutrophication in marine waters, such as the Baltic and Black Seas and Chesapeake Bay (Forsberg 1998).

The first process configuration designed to both nitrify and denitrify was that of Ludzack and Ettinger (1962), which used an anoxic zone followed by an aerobic zone, the zones separated by a baffle. However, this process had little success as the researchers insisted that the first zone must never be devoid of oxygen, thus restricting the potential for denitrification (Barnard 1994). The state of the art in the late 1960s was considered to be a three separate sludge process, with carbonaceous removal, followed by nitrification and methanol dosing for denitrification in the final stage (Bart, Brenner and Lewis 1968). Simultaneous nitrification and denitrification was observed in several oxidation ditch plants in the early 1970s, but control of the final effluent quality was considered difficult (Drews and Greeff 1973).

The separation of the anoxic and aerobic reactors of the Ludzack-Ettinger process to achieve denitrification was developed by Barnard (1973). Since that time a variety of process configurations have been developed to achieve denitrification using the carbon in the feed wastewater, rather than supplying an external carbon source (WEF and ASCE 1992). Only a limited degree of denitrification can occur in trickling filters and other non-submerged aeration processes and this is one of the reasons for the decline in popularity of such treatment trains (USEPA 1993, WEF 1996).

2.4.3 Phosphorus Removal from Wastewater

2.4.3.1 Chemical Phosphorus Removal

There was little pressure to remove phosphorus from wastewater treatment plant effluent until it was discovered that eutrophication of some inland waters was occurring due to high levels of soluble orthophosphate, such as was discovered in Lake Constance during the 1960s (Muller 1997) Conventional secondary treatment partially exacerbates the problem by converting relatively inaccessible organically bound phosphorus into inorganic soluble orthophosphate (Ho et al. 1990). In 1972 the Swedish government required all sewage effluent discharged from urban areas to have a phosphorus concentration of less than 0.5mg/L and major discharges to the Great Lakes in North America were given an effluent standard of less than 1mg/L of phosphorus in 1978 (Cullen 1994). In the initial stages these requirements resulted in installation of chemical dosing facilities using trivalent metal ions or lime, but they also gave impetus to the development of biological phosphorus removal processes. Chemical phosphorus removal generates significant quantities of sludge which are relatively difficult to treat (USEPA 1987). In Norway during the 1970s and 1980s, phosphorus only limits without requiring nitrogen removal led to a preference for chemical removal (Barnard 1994).

2.4 3.2 Biological Phosphorus Removal

The biological processes occurring in non-nutrient removal treatment plants take up phosphorus as part of their biomass at a rate of about 1.5 to 2%, which is reflected by a 10-30% reduction in phosphorus concentration of sewage undergoing secondary treatment, the actual value depending on the ratio of organic material to phosphorus (USEPA 1987).

Enhanced biological phosphorus removal (EBPR) beyond these levels was first proposed by Greenburg, Levin and Kauffman (1955) following work on activated sludge. The first observation of this phenomenon was reported by Srinath, Sastry and Pillai (1959), noting that vigorous aeration of sludge could result in low soluble phosphorus concentrations in mixed liquor.

Levin and Shapiro (1965) reported that a greater than 80% removal of phosphorus was achieved without chemicals. Introduction of a chemical known to inhibit bacterial function prevented the phosphorus removal and this indicated that the process was biologically based rather than chemical in nature. Work was continued by the same group to develop the Phostrip™ process which collected phosphorus biologically and then stripped it chemically when it was released under anaerobic conditions (USEPA 1987).

A number of plants in the USA were noted as having high phosphorus removal without chemicals (Vacker, Connell and Wells 1967; Yall et al. 1970; Milbury, McCauley and Hawthorne 1971). These plants were studied intensively to determine why they should remove phosphorus when other very similar plants did not. Theories were developed as to potential mechanisms for this phosphorus uptake. There were basically two camps, those who ascribed to a chemical precipitation of hydroxyapatite (for example Menar and Jenkins 1969) and those who favoured an as yet unknown process of uptake by bacteria. Following detailed investigation the chemical precipitation theory was found not to fit with the actual experience (Barnard 1990), with work by Lan, Benefield and Randall (1983) finding that a maximum in the range of 15 to 27% of phosphates removed could have been chemically precipitated in the anaerobic-aerobic treatment process they observed.

Work by Barnard (1974) noted that an anaerobic zone devoid of both oxygen and nitrates followed by an aeration zone was required to achieve the enhanced phosphorus removal. The first configuration based on this observation was the Phoredox process in which influent and return sludge were treated in an anaerobic zone, prior to entering the aerobic reactor (Randall 1992). This process was first applied in 1974 at Germiston in South Africa, and was later patented in USA by Air Products and Chemicals, Inc. as the Anaerobic/Oxic (A/O™) process (Barnard 1994). Later in 1974, the plant at Meyerton, South Africa was constructed to incorporate two anoxic and aerobic zones after the anaerobic zone, which resulted in a configuration capable of removing both nitrogen and phosphorus (known as the modified Bardenpho process) (Barnard 1990). The Goudkoppies plant in South Africa was also modified with the addition of an anaerobic zone following experimental work by Nicholls (1975).

Following the success of these plants numerous different flowsheets, which can perform EBPR incorporating an anaerobic zone for mixing of the influent and return sludge (sometimes following a pre-denitrification zone), have been developed. The need for the anaerobic zone to provide strictly anaerobic conditions (that is no nitrates) was confirmed by Nicholls and Osborn (1979). Simple oxidation ditches were shown to be capable of performing enhanced nitrogen and phosphorus removal by appropriate control of aeration, though a separate upstream anaerobic reactor was found to permit greater control of the phosphorus removal process (Randall 1992).

As noted in Section 2.3.3.4 phased isolation configurations with multiple oxidation ditches were developed in Denmark by Krüger to remove nitrogen. With the addition of an anaerobic zone prior to the ditches these configurations were able to remove phosphorus as well as nitrogen (Bundgaard, Andersen and Petersen 1989). This process is called BioDenipho™ and is the biological nutrient treatment train used at Self's Point. The process was first used at full-scale in Denmark in the early 1980s and there are now over 100 BioDenipho™ plants in operation (Hans Regnersgaard, pers. comm., 1996)

2.4.3.3 Microbiology and Biochemistry of EBPR

Fuhs and Chen (1975) were the first to identify *Acinetobacter spp.* and propose that these bacteria were responsible for the enhanced uptake of phosphorus after examining sludge from two plants exhibiting high levels of phosphorus removal. While a large amount of research has focused on proving this theory, there have been difficulties in obtaining similar phosphorus release results from pure cultures of *Acinetobacter* (for example Deinema, Van Loosdrecht and Scholten 1985). Other bacteria, such as *Pseudomonas* and *Aeromonas* were implicated in phosphorus uptake processes in later work (as described in Raper 1990). Nicholls and Osborn (1979) suggested that *Acinetobacter*, although they are considered strictly aerobic organisms, could release stored polyphosphates to provide energy to uptake acetic acid in an anaerobic environment. Converting the acetic acid to poly-β-hydroxybutyrate (PHB), the PHB would be available in aerobic conditions to metabolise and allow the bacteria to replenish the stores of polyphosphate by taking up excess phosphorus from solution The proposal also noted that the anaerobic conditions were required as the presence of nitrates would permit denitrifying organisms to preferentially sequester those short chain fatty acids, which appeared to be a requirement for enhanced phosphorus removal.

While some still maintain that *Acinetobacter spp.* is the "dominant bio-P organism" (for example WEF 1998: 92), there is significant evidence that a range of micro-organisms can achieve EBPR and that others can be dominant depending on such factors as the influent characteristics and the particular process configuration used and that *Acinetobacter spp.* may comprise only a small portion of the microbial community in an EBPR sludge (Streichan, Golecki and Schon 1990; Cloete and Steyn 1988, Wagner et al. 1994). Other organisms which are considered as contributing to EBPR include *Pseudomonas, Aeromonas, Moraxella, Klebisella, Enterobacter, Sphaerotilus natans, Micrococcus* and *Arthrobacter* (Bitton 1994; Bond, Blackall and Keller 1994; Wanner 1994)

The biochemical mechanism by which phosphorus was released under anaerobic conditions and taken up again in an aerobic environment evolved over time. Comeau et al. (1986) proposed the first comprehensive model for this mechanism, proposing mechanisms for transport for hydrogen ions, carbon substrate and phosphorus. Further refinements have been made to this model by Wentzel et al. (1986) and it has been used as the basis for the International Association on Water Quality (IAWQ) Activated Sludge Model No. 2 (Henze et al. 1995a). Due to the variety of micro-organisms which appear to be involved, and conflicting results from research, there is a degree of uncertainty over the exact biochemical pathways involved in the process (Carucci et al 1997). There is however considered to be sufficient understanding of EBPR processes to permit the design of full scale plants (Ekama, Wentzel and Marais 1990).

2.4.3.4 Prefermentation to Promote EBPR

Early biological phosphorus removal results were quite variable, with the reasons not readily apparent. Long sewer retention times were noted as providing a partial hydrolysis of the raw sewage, increasing the short chain volatile fatty acid (SCVFA) content (Barnard 1990). Observations of plant performance at Kelowna in Canada, using a side-stream fermenter indicated that SCVFAs were beneficial in achieving enhanced phosphorus removal if added to the anaerobic zone, as confirmed by Oldham (1985). Barnard (1984) proposed prefermentation of primary sludge to increase the fraction of SCVFA being fed to the anaerobic zone. He put forward several methods of which the simplest option was considered to be holding a sludge blanket of several days residence time in the primary sedimentation tank and recirculating sludge to the tank inlet (described as "activated primary" operation).

A variety of prefermentation systems have been used since that time, and recent analysis of performance has indicated a wide range of operating efficiencies (Münch and Koch 1998). Accurate prediction of prefermenter performance without onsite experimentation has not yet been achieved (Barnard 1992), but there are few existing nutrient removal plants which do not rely on some form of prefermentation to achieve reliable biological phosphorus removal (Randall, Stensel and Barnard 1992a). Recently the University of Queensland Advanced Wastewater Management Centre released software for modelling prefermenters, reflecting the advances in understanding of prefermenter design and operation (v. Münch 1998).

2.4.4 Biological Nutrient Removal Treatment Plants in Operation

There has been a rapid increase in the number of plants utilising biological nutrient removal in North America and Europe with increasing certainty in design criteria and pressure by regulatory authorities for reduced nutrient loads in effluents. By 1992, a total of 215 treatment plants in the USA were identified as having biological nutrient removal (Reardon 1994). This has been mirrored by a similar growth in Australia where the first plant designed for nutrient removal commenced operations in 1984, while by 1997 twenty seven plants had been commissioned (Keller and Hartley 1997).

2.5 Disinfection

2.5.1 Introduction

Following on from John Snow's work and the development of the theory of disease transmission the need to disinfect water supplies became apparent (Isaac 1996a). While the ability of sand to filter liquids was known for some time before, the filtration of a water supply was first performed successfully on a town scale at Paisley, Scotland in 1804 (ASCE, AWWA and CSSE 1969). Although early filtration was principally to improve the appearance of the water it was also effective at providing some disinfection

The first application of chlorine to wastewater occurred in England in the 1850s (Darby et al. 1995). One of the first cities to disinfect its drinking water supply with chlorine was Hamburg in 1893 (Isaac 1996a), while in the same year chlorination of the sewage effluent from Brewster, New York commenced (EWS 1979). As knowledge of the impacts of wastewater on drinking water supplies, public swimming waters and shellfish harvesting areas grew, in the USA in particular, the quantity of effluent undergoing disinfection increased. By 1912 several coastal communities in the USA were using chlorine for wastewater disinfection (Isaac 1996a). The plant at Folly Point in North Sydney commenced disinfecting its wastewater effluent with chlorine in 1919 (Bowen 1998).

Wastewater treatment facilities in Europe have not had the same pressure to implement effluent disinfection as in the USA, except for site specific public health reasons. Some Danish wastewater treatment plants are still not required to disinfect their effluent (Hans Regnersgaard, pers. comm , 1996). In an overview of disinfection in Australia by Hamilton and Thomas (1993) it was reported that no Tasmanian treatment plant was using chlorination to disinfect wastewater in 1991 and that only some effluent was disinfected by lagooning, although legislation at the time did require disinfection of wastewater treatment plant effluent

2.5.2 Disinfection by Chlorine and Hypochlorite

Chlorine was readily available and its price had dropped to affordable levels by the early 1900s due to cheaper production processes (USEPA 1986). Chlorine or hypochlorite became the disinfectant of choice for large wastewater treatment plants for many years (WPCF and ASCE 1977). For larger plants liquid chlorine is used while for smaller flows sodium hypochlorite is often selected due to less restrictive storage and handling requirements.

Concerns in the 1970s about the environmental problems that may be caused both by the chlorine residual and chlorinated by-products, including known carcinogenic chemicals, resulted in dechlorination systems being required after chlorination in some plants (Metcalf & Eddy 1979) While still the most prevalent form of wastewater effluent disinfection, other methods are becoming more common, often due to concerns about the environmental impacts of using chlorine (WEF 1996)

2.5.3 Disinfection by Other Chemicals

The properties of other chemical disinfectants such as bromine, ozone and chlorine dioxide have been known for many years to be effective, but the operating costs have generally been too expensive for their wide scale use (for example Fair and Geyer 1954 and EWS 1979). The earliest demonstrated use of ozone as a disinfectant was in France in 1886 to sterilise polluted water (Cheremisinoff, Cheremisinoff and Trattner 1981). While used in many places for drinking water, it was not until the 1970s that ozone was used for wastewater effluent (WPCF 1986). Ozone is presently employed for disinfection at wastewater treatment plants in Europe, USA and the Middle East, and it does have a number of advantages over chlorine, including the removal of additional chemical oxygen demand (COD) and non-creation of toxic compounds (Lazarova et al. 1998). However, ozone can react with many organic compounds and form epoxides, ketones and aldehydes, which may have long term environmental and health impacts (Isaac 1996b).

Chlorine dioxide was first recognised as a bleaching agent for wood pulp in 1854. In following years its disinfectant properties were also noted (WPCF 1986). It has the advantages of avoiding the creation of halogenated by-products and not oxidising bromide to bromine as do chlorine and ozone, both of which caused environmental and health concerns to be raised in the 1970s (Bull 1993; Metcalf & Eddy 1979). Chlorine dioxide has been more frequently used for disinfecting drinking water than wastewater due to its high cost (WPCF 1986). One of the precursor chemicals required to generate chlorine dioxide, sodium chlorite, is very expensive in Australia (Hamilton 1996). Hobart City Council's experience with chlorine dioxide, which commenced in 1991 at Self's Point, was that while reasonably effective it was too expensive and its use stopped in 1995 (Ray Farley, pers. comm., 1998).

2.5.4 Disinfection by Ultraviolet Light

The disinfecting power of ultraviolet (UV) light was first described by Downes and Blount (1877) when the capability of sunlight to destroy bacteria was ascribed to the ultraviolet section of the spectrum. Small wastewater treatment plants can make use of the ultraviolet disinfecting power of solar radiation through the holding of wastewater in shallow lagoons, but this is not viable for most larger plants.

The first mercury vapour lamp with an extended life was devised by Cooper-Hewitt in 1901 (EPA 1986). Drinking water in Marseilles was disinfected with ultraviolet light from 1910 (Stein 1926), but both the capital and operating costs of lamps were high and use was not widespread even for drinking water supplies.

The reason that the ultraviolet light wavelengths emitted by mercury vapour lamps was so effective when compared to other wavelengths of ultraviolet light was still unknown in the mid-1950s (Fair and Geyer 1954). The mechanism causing the damage to micro-organisms was not determined until after the structure of DNA was discovered (Jagger 1967 cited in Darby et al. 1995).

Much of the work in developing and proving the performance of UV disinfection systems occurred during the late 1960s and 1970s (WPCF 1986). This work quantified the important parameters in meeting required levels of disinfection. Until approximately ten years ago the effluent quality achievable from wastewater treatment without advanced processes was not sufficiently high for UV disinfection to be feasible. UV disinfection requires relatively high transmissivity (>30%) and low suspended solids concentrations (<20mg/L) to be effective or the radiation is absorbed by the wastewater or occluded by particles and does not reach the organisms it is being used to treat (Hamilton 1996).

Recent technical advances both in wastewater treatment processes and in the manufacture of UV systems have resulted in this method of disinfection becoming cheaper than chlorination/dechlorination for effluent qualities of less than 20mg/L suspended solids (Darby et al. 1995; Blatchley et al. 1996). It was estimated by Isaac (1996a) that 10% of the wastewater treatment plants in the USA were using UV irradiation for disinfection in 1996. Ultraviolet light disinfection is presently considered as the method of choice for medium and large plants with high quality effluent (Lazarova et al. 1998).

2.6 History of Sludge Treatment

2.6.1 Sludge Digestion

Following the development of settling tanks in the 1850s, L. H. Mouras of France devised a treatment unit based on the observation that if solids were retained in a closed tank they were partially converted to a liquid state (Metcalf & Eddy 1979). Donald Cameron was the first person to note that a combustible gas containing methane was generated when wastewater solids were held in a closed vessel. He collected and used the gas for lighting near a plant at Exeter, England (Metcalf &

Eddy 1979) The first dual purpose settling and sludge treatment tank was installed in Hampton, England in 1904 (Metcalf & Eddy 1979). Dr Karl Imhoff of Germany took out a patent on a dual purpose tank known as the Imhoff tank, some of which are still in use at some small plants (Novotny et al. 1989). For a number of years this process was very popular, but as knowledge developed in the 1920s and 1930s it became apparent that anaerobic digestion in a separate, heated tank was more efficient in producing a stable sludge and generating a greater volume of gas (Babbitt 1953, WEF and ASCE 1992). Up until about 1950 a digestion period of 30 to 60 days was normally used, but with the development of high rate systems incorporating mixing of the digester contents the time could be reduced to about 20 days thus allowing smaller digestion volumes and reduced tank sizes (WEF 1995).

Aerobic digestion as a separate process was developed in the 1960s (WPCF and ASCE 1977). Extended aeration plants with sludge ages of greater than 15 days use the same basic principle to stabilise the sludge within the same reactor (the required sludge age is dependent on temperature). This effect was first noted by Pasveer (1964) in his work with oxidation ditches. Lower in capital cost than anaerobic digestion, the process requires greater energy, is affected by ambient temperatures and can generate odours (WEF 1995).

An aerobic process gaining popularity is autothermal aerobic digestion (ATAD), where energy released by organic breakdown is used to heat the sludge to 50-65°C, which increases reaction rates and thus results in reduced reactor volume. The initial work on this process was performed by Popel and others in the late 1960s with the first successful full scale installation in 1975 (Schwinning et al. 1994). Worldwide the number of plants using ATAD is increasing as it meets high pathogen removal requirements (Schwinning et al. 1994), although at present there are no major installations in Australia (Rob Pearson, pers. comm., 1998).

Anaerobic digestion has remained the process of choice for much of the 20th Century, being simple and well understood (Novotny et al. 1989). However over the past 30-40 years there has been a proliferation of sludge stabilisation treatment processes. Some of these have been driven, particularly in Europe and the USA, by the high disposal costs at landfill sites and the inadequacy of anaerobic digestion to meet parameters for beneficial reuse. While anaerobic digestion is still used, processes such as lime stabilisation, ATAD and fluidised bed drying are becoming more prominent due to increasing disposal costs and the tight microbiological criteria for biosolids reuse (such as given in WEF 1995; NSW EPA 1997).

2.6.2 *Sludge Thickening and Dewatering*

One of the earliest process operations for the treatment of sewage was that of settling. For many years gravity settling tanks were the only method used to increase solids concentration. During the early 1900s methods were developed which used less land area, were cheaper to construct and were more efficient at capturing solids. Martin (1927: 326) describes the processes which had been trialled for "primary removal of water" prior to 1927. These included drying beds, filtering, chamber filter press, vacuum filtration, centrifugation and flotation.

Thickening is the term usually used to describe the increase in dry solids content of sludges to 5-10% resulting in a pumpable fluid, whereas dewatering is usually used to describe processes which result in a solid cake (WEF 1996). Centrifuges which have forces of several hundred or more times that of gravity were first used in the 1920s and can thicken or dewater depending on operating conditions (WPCF 1980). A thickening process called dissolved air flotation (DAF), which uses the propensity for wastewater solids to attach to air bubbles, was first used in large scale units in the 1930s (WPCF 1980). The rotary drum thickeners installed at Self's Point are a relatively recent innovation and their advantages are low space requirements, low capital cost, low power consumption and high solids capture (WEF and ASCE 1992).

Vacuum filtration was first used in the USA in 1920 (WEF and ASCE 1992), and was the most prevalent dewatering system in the 1950s (Babbitt 1953) and through to the 1970s (WPCF and ASCE 1977). However the high power and chemical consumption and poor performance of the system on waste activated sludges compared to centrifuges and belt filter presses has seen it fall out of favour (Kemp 1997; WEF 1996). Belt filter presses for municipal treatment plant sludges were first developed by Klein in Europe and Smith and Loveless in the USA in the 1960s (WEF and ASCE 1992). While a relatively recent innovation for this purpose the Fourdrinier paper-making machine, invented in 1799 operated with the same basic arrangement (WEF and ASCE 1992). Belt filter presses have increased in popularity and at the present time are considered to be the most prevalent mechanical sludge dewatering process in Australia (for some plants see AWWA 1997).

The thickening and dewatering processes which have recently become more prevalent generally require conditioning agents to work most efficiently. Inorganic chemicals such as lime have been used for many years adding significantly to the final quantity of sludge requiring disposal. The advent of polyelectrolytes or "polymers" in the 1960s, which only require relatively low dosage rates, has contributed to the rise of technologies such as belt filter presses (WEF and ASCE 1992).

2.6.3 Sludge Disposal and Use

With the introduction of settling processes came the need to either dispose of or treat the resultant sludge. Initially this became an extension of the night soil service and was often collected and disposed of in waterways or used on land as manure (for examples in Hobart see HCC 1998 and Goc 1997). Early sludge disposal to land often did not consider potential adverse impacts on soil or crops with the main driving force being cost minimisation (NRC et al. 1996). Increased emphasis on applying sludge at rates to suit the soil and level of production has occurred since the 1970s. Land application accounts for the major part of sludge disposal in many European countries, with for example 63% of sludges being used in his way in the Netherlands in 1987 (Webers and Visser 1991). By contrast in Tasmania where landfill volume has been relatively cheap, a survey by Department of Environment and Land Management (DELM) in 1991 determined that 85% of biosolids were disposed of to landfill (Tasmanian Audit Office 1994). Pathogen reduction and control over heavy metals and other contaminants have became major concerns to ensure public health is protected and these have spurred the development of additional treatment processes and tighter discharge controls to sewer (WEF 1995; Vesilind 1991).

There are a variety of sludge disposal or conversion methods other than land use or disposal, including incineration, pyrolysis and wet oxidation, of which only incineration is widely used (Qasim 1994; Hall and Zmyslowska 1997). An early example of sewage sludge incineration is noted as having been installed in 1935 (WPCF and ASCE 1977). A survey of wastewater treatment plants in the USA indicated that an average of 27% of sludge is incinerated, with most of this occurring at very large plants (WEF 1996).

Vermiculture treatment of wastewater sludge, as used by the Hobart City Council, is not used widely. Charles Darwin (1881 cited in Edwards 1998) was probably the first to recognise the ability of earthworms to break down organic matter and release nutrients. However it was not until the late 1970s that earthworms were seriously investigated as a means of treating sewage sludge and commercial scale operations have been slow to develop despite the potentially high value of the end product (Edwards 1998). Several plants in Poland use vermiculture to treat wastewater treatment plant sludges (Anon 1997a). Redland Shire in Queensland recently commissioned a vermiculture plant to process the sludges from its treatment plants (Vermitech 1998; Lotzof 1998).

2.7 Chapter Summary

This chapter has reviewed the history of wastewater collection and treatment and has traced the development of the major processes, with particular emphasis on those incorporated into the Self's Point Plant. There is an increasingly detailed understanding of the various processes, with significant developments in nutrient removal technologies occurring over the past thirty years. The challenges to achieve effluent qualities compatible with environmental values can now be met by a variety of different processes, which are continuing to be refined with further research and development.

The Selfs Point Plant incorporates technologies of varying vintages, as do many plants which have undergone retrofit upgrades. The retention of existing processes can often significantly reduce upgrade costs, but as in the case of Selfs Point results in processes which may not have been selected in a greenfield development.

CHAPTER 3: HISTORY AND BACKGROUND IN TASMANIA

3.1 History of Sewage Collection and Treatment in Hobart

3.1.1 Early History of Sewage Collection

The development of wastewater collection and disposal in the Hobart City Council area has proceeded in a similar fashion to other urban areas in Australia. However the construction of sewers and treatment plants occurred at later dates than most other early settlements, possibly due to the lower population growth rates and the steeper topography permitting ready disposal via existing waterways. Drainage was initially constructed to remove surface runoff and was usually by open channel. Later, sanitary sewers and sewage treatment systems were installed, and then progressively improved, in response to community demands.

The first European settlement in the Derwent Estuary region was at Risdon Cove in 1803. However this was moved in early 1804 to Sullivan's Cove due to the better harbour and more reliable water supply provided by the Hobart Rivulet (Crawford and Ryan 1988). Collins noted in a report to the King in 1804: "(t)he Run which supplies us with clear, wholesome Water, having its source in an adjoining Mountain, leaves me no reason to doubt its proving a constant supply" (cited in Crawford and Ryan 1988: 3). From the earliest months of the settlement Lieutenant David Collins had concerns about the disposal of waste (Rayner 1988).

As the settlement's population grew from 3,500 in 1820 to 21,000 in 1847, much of the development occurred in South Hobart along the banks of the Rivulet and while attempts were made to retain a buffer zone along the banks, such as recommended by the Land Commissioner in 1826, none succeeded (Button 1978). During this period the Rivulet was both water supply and wastewater disposal drain for many people, with those further downstream suffering both discomfort and illness. An official Government inspection in the late 1820s recorded extensive pollution, including the soaking of green hides near the Cascades and pig farm wastes entering the Rivulet (Crawford and Ryan 1988). The Colonial Surgeon in 1831 noted "the good health enjoyed by the residents at the upper part of the town compared to the ill health of those who live near the mouth of the stream" (CSO 1831 cited in Button 1978: 74).

A water supply via aqueduct from the Rivulet above the populated area was completed in 1832, which significantly improved the quality of water for those who had access to it. However it was suggested by Dr. Officer in 1843 that the poor were using the Rivulet for washing and drinking due to the expense of clean water (Rayner 1988). During the 1860s a reticulated potable water supply was installed to all of the built up area of Hobart Town (Crawford and Ryan 1988).

As early as 1834, the suggestion was made to enclose the Rivulet in the town area (Button 1978). However the cost of such an undertaking was too great and it was only over a very extended period that the lower section of the Rivulet was covered due to the perennial lack of funds for such works (Rayner 1988). The use of the Rivulet as a disposal system was officially recognised with the proclamation of it being a public sewer in 1843 (Button 1978). The Public Health Bill of 1884 required the closure of cesspits by 1887, and such sanitary waste was collected by nightsoil cart for disposal in the Derwent or as manure on farms (HCC 1998). Over 500 water closets were continuing to discharge to the Rivulet during the 1890s (Petrow 1984 in Rayner 1988).

Due to its high cost there appears to have been significant resistance by governments to fund a piped sewerage system. During an address in 1897 the Mayor of the time indicated that construction of underground sewerage was only a possibility (Scripps 1995). In 1901 a Royal Commission examined the Council's performance with regard to sanitation and made many recommendations, including the installation of a reticulated sewerage system (Scripps 1991). Following a poll approving the borrowing of money (Walch and Sons 1903), the construction of a separate sewer system was commenced in 1903 by the Metropolitan Drainage Board (Scripps 1995). Work was completed in the Hobart Rivulet catchment area by 1912 (HCC 1998). The decision to construct a new separate sewerage system leaving the existing drainage system to cope with surface runoff led to flowrates during rain events which could reasonably be treated in later years, unlike the combined system still being used in Launceston (Nicholls, Spratt and Crawford 1963).

3.1.2 Later History of Macquarie Point Treatment Plant Catchment

Provision of a treatment works using septic tanks at Macquarie Point was included in the original plans for the Hobart Rivulet sewage catchment (Nicholls, Spratt and Crawford 1963). Some of the planned treatment system was installed, but due to cost work was abandoned in 1910 and the land was subsequently sold (Petrow 1995).

Screening had been in place for many years at Macquarie Point before the installation of a macerator in 1962 (Nicholls, Spratt and Crawford 1963). Primary treatment using sedimentation tanks was commissioned in 1967, followed by construction of digesters to treat the collected sludge and replacement of the outfall to discharge in 10 metres of water (Hepper, Marriott and Associates 1985). Full secondary treatment using high rate trickling filters was not completed until 1990. From this time the plant was able to meet environmental licence requirements for BOD and suspended solids (Scott and Furphy 1990) and was able to comply with the bacteriological licence criterion after commissioning of the effluent chlorination system in 1991 (Ray Farley, pers. comm., 1998).

In 1962 it was noted that the flow being received by the plant had "decreased slightly" from that in 1938 to about 17 ML/day (Nicholls, Spratt and Crawford 1963: 3) In 1977 the daily flow was noted as being 16.3 ML/day (Scott and Furphy 1977), which has since dropped to the 1998 value of about 13.5 ML/day (Ray Farley, pers. comm., 1998). The drop in sewage flow has occurred despite a relatively constant number of households over this period and the drop would appear to be indicative of improved water conservation measures such as dual flush toilets and reduced water consumption by industry, and also in response to a small decrease in residential population.

The site at Macquarie Point is very limited in area and major improvements to its effluent quality are likely to require a relocation of the treatment processes to another location, of which only Self's Point is considered as being feasible (Thornberg, Pearson and Morgan 1997). This is due to the presence of existing operations at that site, an adequate buffer zone and land area for expansion of treatment facilities.

3.1.3 History of Self's Point Catchment

The Self's Point catchment incorporates the suburbs of New Town, Mount Stuart and Lenah Valley. While there had been some development in both New Town and Lenah Valley (then Kangaroo Valley) in the 1890s, New Town was not proclaimed a municipality until 1908. Some of the southern section of the municipality, bordering the City of Hobart, was included in the area provided with reticulated sewerage under the initial Metropolitan Drainage Board in 1909 (Scripps 1993). At least one proposed subdivision appears to have lapsed due to requirements for the installation of sewers to protect New Town Creek (Scripps 1993). When New Town amalgamated with the City of Hobart in 1920 one of the advantages to New Town residents was the extension of the sewerage reticulation system to the area. Work commenced on the system after delays due to financial constraints. By 1927 all except outlying areas had sewerage connections (Scripps 1993). An outfall was constructed at Self's Point and at the time it was planned for this to also serve Glenorchy (Scripps 1993).

New Town Bay was used as a tip during the 1930s and 1940s to reclaim land, much of which is now playing fields. During the latter stages of the tip life the area where the Self's Point Plant is now located was reclaimed from the Bay (Scripps 1993).

Only limited treatment of the sewage from the catchment occurred until 1973, when primary sedimentation tanks, anaerobic digestion and drying beds were commissioned (Hepper, Marriott and Associates 1985). This installation was in response to the requirements of the *Environment Protection Act 1973* and the discharge limits described in the *Environment Protection (Water Pollution) Regulations 1974*. The Regulations required the effluent to have a biochemical oxygen demand (BOD_5) of less than 40mg/L and a suspended solids concentration of less than 60mg/L for bays and estuaries, where the receiving water flow is greater than 50 times the effluent flowrate (Scott and Furphy 1977). The Self's Point effluent did not meet these requirements until the commissioning of trickling filters and humus tanks in 1977 (Hepper, Marriott and Associates 1985).

Disinfection of the plant discharge using chlorine dioxide commenced in 1991 and thereby the effluent was able to meet the bacteriological licence limit of less than 1000cfu/100mL faecal coliforms (Kinhill 1993). Chlorine dioxide was principally selected over chlorine due to the minimal creation of halogenated by-products compared to chlorine and it being more effective over a broader pH range (Kinhill 1993). The use of chlorine dioxide was discontinued in 1995 due to the high operating cost and replaced with chlorine (HCC 1995). From a comparison of effluent disinfection systems used in Australia, chlorine dioxide had the highest operating costs, mainly due to the high cost of sodium chlorite (Hamilton 1996).

All sludge from the Plant underwent primary and secondary anaerobic digestion after gravity thickening in settling tanks. The digested sludge was then pumped to the drying beds for treatment prior to disposal (Ray Farley, pers. comm., 1998).

The modifications made to the Self's Point Plant during the BNR upgrade and plant expansion are described in Chapter 5.

3.1.4 History of Sandy Bay Catchment

During the latter stages of the 19th Century the Sandy Bay Rivulet was used as a disposal system by the residents of the Hobart City Council area immediately north of the Rivulet (Goc 1997). There were numerous complaints about odour in the 1890s. One instance was investigated in 1895 and was found to have resulted from "certain deposits of a deleterious matter caused by the City of Hobart being allowed to run into the Rivulet" (cited in Goc 1997: 228). After years of being at loggerheads, the Hobart City Council and Queenborough Town Board agreed in 1900 to fund improvements to the Sandy Bay Rivulet to improve its flushing capability (Goc 1997).

The population in the Queenborough area grew rapidly after 1900 and sanitary wastewater disposal became an increasing problem. In 1917 the decision was taken to extend the metropolitan drainage scheme to Queenborough (Scripps 1995). Most of the sewer network, pumping station and outfall at Blinking Billy Point were completed in 1920 (Hepper, Marriott and Associates 1985). This system discharged untreated sewage through a short outfall to the river.

Due to increasing concerns about pollution from the existing short outfall pipe, a new outfall was constructed at Blinking Billy Point in 1962 some 730 metres in length and discharging in 30 metres depth of water. At this time a macerator was installed, but due to ongoing maintenance problems was replaced by a comminutor in 1966 (Kinhill 1993).

From 1970 onwards, the Hobart City Council proceeded with numerous investigations into a suitable location for a wastewater treatment plant for Sandy Bay sewerage. Initially the decision was taken to build a plant at Blinking Billy Point and land was purchased for this purpose in 1971. However following objections from nearby residents, the possibility of a joint Hobart-Kingborough plant was proposed for Crayfish Point was raised and rejected by Kingborough Council in 1972 (Kinhill 1993). Following this rebuff, a report by Scott and Furphy (1976) recommended the construction of a plant at Blinking Billy Point. Again public pressure forced Council to consider a joint facility with Kingborough. Various proposals were rejected by Kingborough Council in 1979, 1981 and 1990 (Kinhill 1993). During the 1980s Hobart City Council committed to spending funds on upgrading the Macquarie Point

Plant, leaving insufficient funds to upgrade the Sandy Bay sewage treatment system. An application was made for exemption to the requirements of the *Environment Protection Act 1973* and the exemption was granted in 3 November 1980 on condition that the need for the exemption would be eliminated by 30 June 1995. In 1988 the time frame for this requirement was reduced to compliance by 30 November 1989 (Hodgman 1988).

Funding was sought by Hobart City Council through the Hobart Metropolitan Councils Association (HMCA) and following the release of the *Derwent Sewerage Strategy* (Scott and Furphy 1990) the City of Hobart received $1.95 million on the condition that it would undertake works to meet the environmental licence. At the time the capital expenditure to transfer and treat Sandy Bay sewage at Crayfish Point was estimated at $13.3 million. In late 1990 the decision was taken by HCC to treat Sandy Bay sewage at either Macquarie Point or Selfs Point (Kinhill 1993). Following further discussions with officers from the Tasmanian State Environment and Planning Department a Development Proposal and Environment Management Plan (DPEMP) was prepared by Kinhill (1993) and the Sandy Bay Sewerage Scheme Feasibility Study prepared by AWT (1993). These reports made the recommendation of transferring sewage to Self's Point for treatment. Negotiations between HCC and officers of DELM resulted in the effluent from the plant having to be returned for discharge at Blinking Billy Point outfall and a DPEMP was completed on this basis (CEE 1994).

The AWT (1993) study recommended the CASS™ (Cyclic Activated Sludge System) process be used. There were however some concerns about whether this was the most appropriate process for Self's Point. Following research and recommendations by HCC officers (HCC 1995b), Council decided in early 1995 to instead proceed with a hybrid BNR/trickling filter process. The chlorine dioxide system was also reviewed during this period and was found to have excessive operating costs and the decision was also taken to use ultraviolet light disinfection in its place (HCC 1995b). The design and construct contract to build the plant was let in late 1995 and sewage entered the new section of the plant in early March 1997. Commissioning of the return pipeline occurred at the same time and all final effluent from the plant was discharged at Blinking Billy Point from this time, with the exception of a few occasions of very high flow due to wet weather (Morgan and Farley 1998).

The Council chose design-and-construct in accordance with the Australian Standard AS4300 as the method of project delivery for the treatment plant upgrade. This method gave process designers a large degree of flexibility and scope for innovation in the final design. Council's engineering department prepared the tender documentation and managed the project with inhouse resources (Morgan and Farley 1998).

Expressions of interest were invited from suitably qualified companies to undertake the upgrade of the Selfs Point Plant to the higher standard. No particular process type was specified, but HCC was keen to ensure that the selected process technology was well proven elsewhere, was of high quality construction and had low operating costs, while meeting the required effluent quality (Morgan and Farley 1998).

After an initial review, selected organisations were given the opportunity to expand on their expression of interest with a presentation to Council officers. After an evaluation of these presentations three companies were invited to prepare tenders for the treatment plant project. The three tenders were of a very high standard and all offered innovative process configurations incorporating the trickling filters (Morgan and Farley 1998)

ANI-Krüger Pty. Ltd. was awarded the contract for the design and construction of the new Selfs Point BNR plant in December 1995. The contract sum including allowance for contingencies was $9.7 million, which compared favourably to a previous cost estimate range of $12-17 million (1993 dollars) for a completely new plant to treat the whole flow (Morgan and Farley 1998). The total project cost was $20.3 million, which included the treatment plant upgrade, pipelines to and from Sandy Bay, pump stations and a new laboratory at the Selfs Point site.

3.2 Legal and Policy Framework for Wastewater Treatment in Tasmania

3.2.1 Provision of a Sewerage System

The HCC is required by legislation to provide a sewerage system where necessary. While only indirectly referred to in the *Local Government Act 1993*, which requires councils "to provide for the health, safety and welfare of the community" (s. 20(1)(e)), it is explicitly stated in the *Sewers and Drains Act 1954*. In section 3(1) of this Act it states:

> It is the duty of every local authority to provide such common sewers as may be necessary for effectually draining its municipality for the purpose of preserving the health of the inhabitants of its municipality, and to make such provision, by means of sewage disposal works or otherwise, as may be necessary for effectually dealing with the contents of those sewers.

Most of the Sewers and Drains Act relates to the sewers and little in the Act pertains to the treatment of the collected sewerage. However, sections 66 and 67 of the Act are important in that they provide local authorities with the power to control wastewater discharges from industries through by-laws Along with a model by-law the State Government has developed guidelines for the acceptance of trade wastes to sewer (DELM 1994b). The *Tasmanian Plumbing Regulations* made under the *Local Government (Building and Miscellaneous Provisions) Act 1993* are used to implement sewer management as indicated by the guidelines (SDAC 1997).

Prior to 1998 HCC had specific legislation relating to the discharge of trade waste (*Sewerage and Drainage By-law (No. 130) 1969*). This by-law has been replaced by another encompassing water, stormwater and wastewater (*Water, Sewers and Drains By-Law No. 6 of 1997*), which includes clauses on trade waste. These clauses provide the HCC with the authority to control wastewater discharge quantities and strengths from industries through the issue of discharge permits. This power has been important in regard to the performance of the Self's Point Plant by ensuring control of the discharge from the milk processing facility located in Lenah Valley.

3.2.2 Commonwealth Policy

Wastewater treatment plants have major environmental impacts. In terms of the receiving environment effluent and solids residuals are the major inputs, but odour and noise can also create environmental nuisances In Australia, the power to control environmental impacts of activities generally lies with the State Governments. The joint Commonwealth and New Zealand bodies, Australian and New Zealand Environment Conservation Council (ANZECC) and Agricultural Resource Management Council of Australia and New Zealand (ARMCANZ), have however prepared guidelines for water quality objectives, groundwater management, and operations of a variety of industries and wastewater treatment facilities as part of the Australian National Water Quality Management Strategy (ANZECC 1992; ARMCANZ and ANZECC 1997). These guidelines are not requirements, but have been developed "as a basis for a common and national approach throughout Australia" (ARMCANZ and ANZECC 1997: 1)

3.2.3 State Government Environmental Legislation

3.2.3.1 Environment Protection Act 1973

Legislation to control environmental impacts has developed in a stepwise process in Tasmania. The first major step occurred in 1973 with the *Environment Protection Act* commencing operation. It was followed shortly after by companion legislation including the *Environment Protection (Water Pollution) Regulations 1974.* This legislation gave the then Department of the Environment significant powers to control treatment plant discharges. The Act and Regulations were typical of the time, based on end-of-pipe concentration limits and relatively independent of the quality of the local receiving environment (DoE 1988).

In 1976 the Department of the Environment stated that all wastewater treatment plants on the Derwent were expected to have secondary treatment by 1980 (Scott and Furphy 1976). It appears that for a range of reasons that the powers provided by 1973 Act were not used to the fullest extent and numerous dischargers of effluent were granted exemptions from some or all requirements of the Act and Regulations. In the late 1980s it was noted that after 15 years of existence of the Act that there had been little or no improvement to the environment near industries (DoE 1988).

There was noticeably more pressure put on non-complying industries following research into water quality of receiving environments (DoE 1988; Coughanowr 1997). This increase in pressure was noticeable in the lower Derwent, where some of the major industries and wastewater catchments, at the time, had little or no pre-treatment prior to discharge. As previously noted one example of this is the Sandy Bay sewage catchment for which an exemption was granted in 1980 delaying the need for treatment until 1995, but this period was later reduced to compliance by 1989 (Kinhill 1993). During the early 1990s wastewater treatment plants were commissioned by Australian Newsprint Mills, Pasminco-EZ and Cadbury and upgrades occurred at many of the municipal treatment plants (Coughanowr 1997).

3.2.3.2 Resource Management Planning System

A suite of legislation brought in during the early 1990s, known as the Resource Management Planning System (RMPS), has modernised Tasmania's environment and planning laws. This legislation includes the *Environmental Management and Pollution Control Act 1994* (EMPCA), the *Land Use Planning and Approvals Act 1993* (LUPAA), and the *State Policies and Projects Act 1993* (SPPA), which are discussed below.

In 1996 the *Environment Protection Act 1973* was replaced by the *Environmental Management and Pollution Control Act 1994* (EMPCA). This new Act in effect did away with the environmental exemption and put in its place the environmental improvement programme (EIP), which has a maximum term of 3 years (s. 39 of EMPCA). If an activity is causing environmental harm, then the Board of Environmental Management and Pollution Control can require the person responsible to submit an EIP for approval. This situation was the case with the Sandy Bay Sewerage Project for which an EIP had to be prepared by June 30 1994, when all environmental exemptions expired (CEE 1994).

Under EMPCA, activities which have environmental impacts are separated into three levels. Level 3 activities are those which have been made a project of State significance under section 18(2) of the *State Policies and Projects Act 1993* and these are referred to the Resource Planning and Development Commission (RPDC) for assessment and approval (s 26 of EMPCA). Level 2 activities are those described in Schedule 2 of EMPCA, which includes sewage treatment works "with a design capacity to treat an average dry-weather flow of 100 kilolitres or more per day" (EMPCA: 86). Level 1 activities are those not described as Level 2 or 3, but which do have an environmental impact and would require approval by local government under the *Land Use Planning and Approvals Act 1993*.

Monitoring of Level 1 activities is the responsibility of appointed officers of the local authority, which in most instances is the municipal council Level 2 activities are monitored and licensed by the Director of Environmental Management, an officer of DELM. In the instance of upgrading a sewage treatment plant the Act requires that a DPEMP has to be assessed and approved by the Board of Environmental Management and Pollution Control. Once this process has been finalised the Director of Environmental Management issues an Environment Protection Notice (EPN) (previously Licence to Operate Scheduled Premises) which includes conditions under which the treatment plant has to operate (for example see DELM 1995). The licence conditions include monitoring and reporting requirements, maximum dry weather daily flow, effluent concentration limits and other environmental performance criteria.

As each wastewater treatment plant is upgraded, effluent quality requirements will be individually determined under EMPCA, based on the achievement of desired receiving water quality objectives for the specific water body, rather than standard discharge limits. This process has been complemented with the approval of the *State Policy on Water Quality Management 1997* (made under the SPPA), which in Section 16.2 states that one of the key principles in setting discharge limits is that they "will not prejudice the achievement of water quality objectives" (Tasmania 1997: 15). The approach engendered by both documents overcomes the problems associated with using blanket end-of-pipe standards. However it is considered that there should be some flexibility in setting water quality objectives due to inherent natural variability (Moss 1998).

It was envisaged that a number of State Policies would be instituted to enact the principles of the RMPS. To date only two State Policies have been finalised, and both the *State Policy on Water Quality Management 1997* and, to a lesser extent, the *Tasmanian State Coastal Policy* (Tasmania 1996) have important implications for wastewater treatment and disposal. It was suggested that a State Policy on integrated catchment management was also to be prepared (Edwards 1997). If such a State Policy is prepared it would be anticipated to have an impact on these activities.

The State Policies are required to contain as little regulation as possible (section 5(1)(d) of SPPA), and thus are necessarily general in character rather than prescriptive. Supporting regulations are required to enforce the Policies. The Sustainable Development Advisory Commission (SDAC) (predecessor of the RPDC) noted that:

> "(t)he principal intent of developing a Water Quality Management Policy was to support the EMPC Act - not over-ride it The Policy (is) intended to provide a framework for interpreting and implementing the EMPC Act in terms of water quality issues - for example, recognising the reality that environmental harm may occur in mixing zones around outfalls." (SDAC 1997: 138).

Thus while the EMPC Act provides the regulatory powers, the State Policies provide guidelines for the setting of objectives to be achieved by regulation

3.3 Effluent and Biosolids Reuse Legislation and Policy

3.3.1 Effluent Reuse Policy and Legislation

The effluent criteria for faecal coliforms and phosphorus specified by the HCC for the Self's Point Plant upgrade were lower than those required to meet the environmental licence conditions (Morgan and Farley 1998). These lower values were anticipated to markedly improve the potential for effluent reuse. A major reason for the decision to specify effluent reuse standard quality was that the return pipeline transports effluent for a distance of over ten kilometres, passing close by a number of areas which require irrigation, at least during the summer months (Morgan and Farley 1998).

Any proposal for the reuse of wastewater treatment plant effluent in Tasmania has to be approved by the Co-ordinating Group for Wastewater Reuse, which is comprised of members from public health, water resources and environmental State government departments and has local government representation (DELM 1994a). The legislated regulatory power to control the reuse of effluent is provided by EMPCA, through conditions in the treatment plant environmental licence or EPN. In the Self's Point Plant environmental licence there is a condition stating that any programme involving effluent re-use requires approval by the Director of Environmental Control (DELM 1995). The document on which approvals are based is the *Guidelines for Re-use of Wastewater in Tasmania* (DELM 1994a).

In 1993 the Minister for the Environment and Land Management released a strategy including the aim "that there will be no discharge of effluent from sewage lagoons into inland waters unless Councils have demonstrated that land disposal is not feasible" (Tasmanian Audit Office 1994: 39). A number of effluent reuse schemes have been or are close to being implemented in Tasmania and there appears to be a high degree of support at all three tiers of government for reuse of effluent.

There is a significant degree of public support for effluent reuse and in a recent survey at several potential reuse sites in the HCC area, there was an 80 to 90% approval rating for using effluent to irrigate a variety of public grassed areas, including sportsfields (Duddles and Mulcahy 1996). The HCC in its Strategic Plan has stated its intention to "develop proposals for ... using treated effluent" (HCC 1996: 13).

To date, the only effluent reused from the Self's Point Plant is as process water for dewatering equipment, for general hosing down duties and for irrigating the majority of the Self's Point site. Funding has been sought for the installation of a system to irrigate nearby public sportsgrounds using effluent from the Plant (Mike Street, pers. comm., 1998) and effluent is to be used as the make up water source for a wetlands being created on land adjoining the site (SEMF 1998).

3.3.2 Biosolids Reuse Policy and Legislation

The use of biosolids in Tasmania is not as advanced in either a regulatory sense or in practice as wastewater effluent reuse. In a review of biosolids production in the Derwent Estuary region, Chrispijn et al (1996: 21) considered that for social and environmental reasons it was "critical" that beneficial re-use of biosolids be regulated and that guidelines be developed.

The disposal of biosolids has been administered by DELM, while any sale or reuse has been regulated by Department of Community and Health Services (DCHS) (Tasmanian Audit Office 1994). Officers of the Department of Primary Industries and Fisheries (DPIF) and DELM are presently in the process of preparing guidelines for biosolids reuse closely based on the New South Wales Environment Protection Authority (NSW EPA) guidelines (see NSW EPA 1997). It is anticipated that the Co-ordinating Group for Wastewater Reuse will be the approval body for any biosolids reuse project (David Dettrick, pers. comm., 1998).

Due to the relatively low costs of landfill disposal in Tasmania and the smaller environmental impact of biosolids compared to effluent there has been significantly less emphasis on the reuse of biosolids. The Tasmanian Hazardous Waste Management Strategy does mention "sewage ... sludges" and appropriate management systems, but the only recommendation refers to halting "inappropriate methods of land disposal of septage" (DELM 1994c: 14).

The HCC Worm Farm is now an operating program within the Solid Waste Management function of Council and, while the use of vermiculture for treating biosolids is not widespread, it is considered to be a sustainable method for processing biosolids into a value added product (Lotzof 1998).

3.4 Derwent Estuary Monitoring

3.4.1 Nutrient Monitoring of the Estuary

Monitoring of the Derwent Estuary water and sediment quality occurred only sporadically during the 1970s and most of the 1980s. The first pollutants studied intensively were heavy metals, for which public health concerns were raised after the discovery of high concentrations of several heavy metals in shellfish (Coughanowr 1997). The then Department of the Environment carried out semi-annual surveys of surface water quality from 1972 to 1988, but while these did include faecal indicators, they did not include nutrients, nitrogen and phosphorus (Coughanhowr 1997).

It was only during the Derwent Estuary Nutrient Program in 1993/94 that detailed information on nutrient concentrations and fluxes in the Estuary were obtained. The results suggested that the nitrate and phosphate concentrations were in the middle to low end of the range at which problems occurred, while ammonia concentrations were relatively high (DELM 1995). Further monitoring was undertaken in 1996 (Coughanhowr 1997).

In summary these studies indicated that there were significant concentrations of bioavailable nutrients in the water column and that these appeared to be due to inputs from Southern Ocean waters rich in nutrients. The phytoplankton levels were noted as not appearing to respond to the excess of available nutrients and a number of other potential factors are described by Coughanowr (1997). Work by Hallegraeff and Westwood (1994) indicates that phytoplankton growth rates may be limited by light levels and possibly by humic substances While further research may indicate otherwise, based on present information it appears that discharge of the nutrients nitrogen and phosphorus near the mouth of the estuary may not have major, adverse environmental impacts.

3.4.2 The Case For Nutrient Removal

Despite the apparent situation in the Derwent, care must be taken in assuming that the phosphorus and nitrogen loads from wastewater treatment plants are of minimal impact. In 1986, the Danish Parliament decided to reduce the nitrogen and phosphorus loads from Danish wastewater treatment plants by 50% and 80% respectively. As expert advisors had conflicting views, action was taken based on the precautionary principle and the measures put in place were in excess of what could be proven as being necessary (Harremöes 1998).

In fresh waters extensive research has demonstrated that phosphorus is most often the limiting nutrient (Cullen 1994). In seawater, however the nitrogen concentration is usually considered as more important (ARMCANZ and ANZECC 1997), but that phosphorus can be limiting in some circumstances (Forsberg 1998). In estuaries the dynamics of the fresh and sea water interface under different flow regimes make it very difficult to assess which nutrient, if any, may be limiting. Modelling of Chesapeake Bay in USA indicated that reducing phosphorus concentrations of point discharge sources, without reducing the nitrogen content would actually increase eutrophication problems near the mouth of the bay, as a result of interactions between biological processes, nutrient loadings and hydrodynamic effects within the estuary and bay (Thomann and Linker 1998).

Wastewater treatment plants are contributing significant quantities of both nitrogen and phosphorus to the Derwent Estuary. An important factor in assessing their impact is the relative bioavailability of the nutrients from different sources. Recent research by Gerdes and Kunst (1998) indicates that on average 87% of phosphorus in effluent from wastewater treatment plants not designed for phosphorus removal is considered to be bioavailable, compared to 53% for urban runoff and 30% for erosion effluent. In the same work it was found that the effluent from treatment plants with biological or chemical phosphorus removal contained 67% bioavailable phosphorus. Biological wastewater treatment processes are noted as generally increasing the soluble phosphorus proportion from about 50% to 90% by Ho et al. (1990). Most nitrogen in treatment plant effluent is either in nitrate or ammonia form, both of which make the nitrogen readily bioavailable. Thus wastewater treatment plant effluent may have a greater relative impact on the receiving environment than total phosphorus and total nitrogen mass balances would indicate.

The cost of incorporating nutrient removal into wastewater treatment plants has significantly reduced in recent years and presently may add only 10% to the capital cost of a plant required to meet 20mg/L BOD and 30mg/L suspended solids limits (USEPA 1987). As the technology is now well proven and readily available at relatively little extra cost, it would appear that the precautionary principle should apply and that nutrient removal treatment of wastewater be incorporated in all new plants or upgrades to existing plants or the effluent beneficially reused in a manner which ensures that neither nitrogen or phosphorus enters the environment at large.

3.4.3 Wastewater Effluent and Faecal Pollution in the Derwent Estuary

There is a significant amount of faecal contamination of the Derwent Estuary with the results at several monitoring stations in 1995 to 1997 being in excess of the ANZECC guidelines for primary contact activities, which includes swimming (Coughanowr

1997). The regions of the Estuary with the highest results are embayments in the mid-estuary, where there is considered to be less flushing. While sewage treatment plants may be the source of some of the pollution, the peaks in faecal indicators following rainfall from samples taken by HCC officers appear to be more indicative of stormwater runoff pollution as the source (Samantha Watkins, pers. comm., 1998). A microbiological study of the Hobart Rivulet measured significant concentrations of the faecal indicators in the lower reaches of the Rivulet (Blacklow 1995). Work by Green (1997) on another Hobart stormwater catchment indicates that only a small percentage of the stormwater faecal contamination is from humans and that by far the largest proportion is from dog faeces Thus while wastewater treatment plants contribute faecal contamination to the Estuary, they discharge into salty deep water, whereas surface runoff is acknowledged as having relatively high concentrations of faecal indicators and would appear to be the most significant source of such contamination.

3.5 Chapter Summary

The development of sewage treatment and disposal systems in the Hobart City Council area have proceeded in much the same manner as elsewhere. Hobart was, however fortunate that the decision was taken to install a sanitary sewer system, separate to the surface runoff system at the turn of the 20th Century, making treatment of high flows during rain events feasible. Secondary treatment of wastewater in the Hobart City Council area commenced with the commissioning of trickling filters at the Selfs Point Plant in 1977 and this was followed by high rate biofiltration at the Macquarie Point Plant commissioned in 1990. The sewage from the Sandy Bay catchment was still entering the Estuary virtually untreated until early 1997.

The upgrade of the Self's Point Plant to accept the sewage from the Sandy Bay catchment was the culmination of an extended process, in which numerous and various options were considered over a period of about twenty years. During this period the development of treatment plant technology had reached the stage where it was only slightly more expensive to install nutrient removal than conventional treatment and this was the level of treatment required by the State Government.

While research to date indicates that nutrients are not the limiting factor for phytoplankton growth in the Derwent Estuary, the precautionary approach has been adopted by DELM and nutrient removal effluent quality is now the standard for all plants discharging to estuaries or inland waters in Tasmania.

CHAPTER 4: THEORY OF WASTEWATER TREATMENT PROCESSES IMPACTED BY NUTRIENT REMOVAL

4.1 Introduction

This chapter reviews theory and research findings as they apply to the process unit operations utilised at the Self's Point Water Reclamation Plant. Only those processes which have an impact on or are affected by the degree of nitrogen and phosphorus removal are discussed in detail. These processes include primary sedimentation, trickling filtration, activated sludge, prefermentation, digestion and dewatering. Disinfection and vermiculture (the biosolids post-processing method) are also discussed with regard to their impacts on the design of the nutrient removal systems.

4.2 Catchment Considerations

4.2.1 Catchment Phosphorus Reduction Strategies

A complementary action to reducing phosphorus at the treatment plant, which some wastewater authorities have undertaken, is to restrict or ban the use of phosphorus containing detergents It has been estimated that 25 to 40% of phosphorus in raw sewage is derived from detergents and cleaners (Morris and Bird 1994). A summary of studies in the USA have indicated that an average of 32% decrease in the raw sewage total phosphorus and a 39% decrease in raw sewage orthophosphate can be achieved with a ban on phosphorus detergents (WEF 1998), whereas European experience indicates a 10% decrease in influent phosphorus concentrations (Randall 1998). An education awareness programme at Albury-Wodonga is considered to have reduced the influent phosphorus to the wastewater treatment system by about 25% (Anon. 1995).

In the 1970s nitrilotriacetic acid (NTA) was used as a substitute for phosphorus in detergent formulations. However this chemical was found to combine with some metals, which had damaging impacts on animals and NTA was subsequently banned (Connell 1981). From the wastewater treatment and environmental perspectives there are drawbacks with all of the present substitutes for phosphorus and as phosphorus can be relatively easily removed, some consider that allowing detergents containing phosphorus is a better option (Morris and Bird 1994). To date these catchment phosphorus reduction methods have not been considered for Selfs Point, with the exception of the milk processor, which has been requested to consider possible alternatives to cleaning solutions based on phosphoric acid (Rob D'Emden, pers. comm., 1998).

4.2.2 *Industrial Contribution to the Selfs Point Plant*

The only significant industrial load in the catchment is a milk processing plant (Morgan and Farley 1998) While the waste milk being discharged to sewer has a high organic content, much of this is not readily available with a significant proportion being fats and proteins, both of which are relatively difficult to degrade (Hanaki et al. 1987) It has also been noted that activated sludge plants receiving wastewater from milk factories often have poor secondary sludge settling properties (Per Nielsen, pers. comm., 1996).

Council officers considered that the retention of this industrial influent without biological pre-treatment was desirable due to the higher organic load potentially eliminating the need for supplementary carbon dosing for nutrient removal (Ray Farley, pers comm., 1998) Due to the significance of this source the controlled release of a portion of the stronger wastewater from the milk processor has been negotiated in an effort to equalise the diurnal organic load to the Selfs Point Plant (Rob D'Emden, pers. comm., 1998).

4.2.3 *Effect of Sewer System on Plant Influent*

As discussed in Section 4.5 one of the prerequisites for enhanced biological phosphorus removal (EBPR) is an adequate concentration of readily biodegradable chemical oxygen demand (RBCOD). While the suitable organic fraction is often taken to be volatile fatty acids (VFA) there are other substrates which will support EBPR (for example aspartate has been shown to a suitable substrate to support phosphorus accumulating organisms (PAOs) (Satoh, Mino and Matsuo 1998)). Raper, Crockett and Glover (1994) have noted at the Lower Plenty Plant that the VFA content of the wastewater did not directly correlate with better phosphorus removal, so other compounds may also be implicated.

In some wastewater catchments industrial discharges can provide adequate RBCOD to achieve good EBPR. One such example is as at Carole Park Wastewater Centre in Queensland, where the influent VFA concentration was typically 50-60mg/L and the soluble COD 250mg/L. The plant was able to achieve 2mg/L total phosphorus in the effluent without an anaerobic zone (Belgrove and Tennakoon 1997). Barnard (1992) notes that given a sufficiently high concentration of very available COD, such as acetate, that some oxygen or nitrate in the anaerobic zone may be tolerable and preferable to excessive anaerobiosis. It appears that high concentrations of RBCOD increase the selective pressures favouring phosphorus accumulating organisms (PAOs).

More usually there is inadequate RBCOD in wastewater reaching a treatment plant to achieve adequate EBPR. This appears to be particularly so in Australia. In a summary of data from several plants Griffiths (1997) assessed typical domestic sewage loading in Australia as being 14.8g/person/day of RBCOD compared to 23.6g/person/day typically used as a default value in numerical models. In a study of a treatment plant at Pakenham in Melbourne, the average influent RBCOD was 7% of the influent BOD, with 6mg/L of VFA (as acetic acid) (Crosher and Harding 1994). These values were considered as being due to the fresh nature of the sewage and transport only by gravity and were considered inadequate to achieve EBPR for typical municipal wastewater. This can be compared to typical values for influent VFAs obtained in Denmark of 35mg/L as COD (Henze 1992).

In Hobart the short residence time of the collection system and steep grade of the sewers could both contribute to low concentrations of RBCOD in the influent to Self's Point. Work has shown that volatile fatty acids degrade in sewers in aerobic conditions (Raunkjær, Hvitved-Jacobsen and Nielsen 1995) and that aerobic conditions can exist for an hour following aeration if the sewage entering the network has a high dissolved oxygen concentration (Tanaka and Hvitved-Jacobsen 1998) Once the wastewater becomes anaerobic volatile fatty acids can be produced in the sewer (Hvitved-Jacobsen, Raunkjær and Nielsen 1995). The rate of VFA production measured by Tanaka and Hvitved-Jacobsen (1998) in a force main in Japan was 7.3gCOD/(m^3.h) or 0.48gCOD/(m^2.h) at 20°C. The latter value is based on the inner surface area of the pipe This situation is expected to be the case in the latter section of the rising main from Sandy Bay and at low flow rates in the New Town catchment which passes through a low lying trunk main near the Self's Point Plant. Approximate calculations indicate that less than 10mg/L of COD would be converted to VFA through this mechanism in the rising main from Sandy Bay due to the relatively short residence time. Much of this VFA would then be degraded in the gravity section following the high point in the pipeline, where the pipe does not flow full.

Hvitved-Jacobsen, Vollertsen and Nielsen (1998) have prepared a model of the microbial system in gravity sewers to estimate the likely removal of RBCOD in aerobic conditions in the sewer system. The model results indicate that for larger diameter sewers and steeper grades there will be greater removal of RBCOD. The grade is of particular importance and in one example it is estimated that for a 2 hour residence time in a 500mm diameter main the reduction in RBCOD at a grade of 0.2% is about 10% while for a grade of 0.5% it is about 70%. Sewers in the Hobart City area typically have steep grades, with most having a mean grade of greater than 0.3% (Ed Kleywegt, pers. comm., 1998). Thus it is considered that a significant proportion of the RBCOD, which would otherwise be available is lost due to the physical structure of the sewer system.

4.3 Primary Sedimentation

4.3.1 Introduction

The basic theories related to settling in primary sedimentation tanks are detailed elsewhere (for example WEF and ASCE 1992) and this study discusses some of the practical issues as they relate to the Self's Point Plant Primary settling performance can vary greatly from plant to plant and would appear to be due, at least in part, to influent characteristics.

An important impact of primary settling is the removal of biochemical oxygen demand (BOD), reducing aeration requirements of the biological processes, while the majority of phosphorus and nitrogen in sewage are typically in soluble forms, which are not removed by settling (USEPA 1993; Ho et al. 1990). Thus, the primary settling removal efficiency of nitrogen and phosphorus is significantly lower than that of BOD, reducing the BOD:TP and BOD:TN ratios. The lowering of these ratios can adversely impact on biological removal processes denitrification and phosphorus removal, by reducing the available carbon source and decreasing the biomass production rates (Randall 1992).

4.3.2 Comparison of Selfs Point Plant Design with Typical Parameters

The Self's Point Plant upgrade design incorporated the two existing primary sedimentation tanks with an average dry weather flow residence time of a little under 2 hours. While performance of primary sedimentation is acknowledged as being variable this would typically result in removal efficiencies as described in the design report provided by ANI-Krüger (1996), which are given in Table 4.1.

The design removal efficiencies for BOD and suspended solids are slightly lower than typical sets of actual operating data (WEF and ASCE 1992) and generalised performance curves (Qasim 1994). However the design overflow rate of 42 $m^3/m^2.d$ and detention time of 1.7 hours are in the ranges typically used (WEF 1996). Data provided by Kennedy et al. (1998) indicates a suspended solids removal of about 60% at $40m^3/m^2/d$, dropping to about 25-30% at $80m^3/m^2/d$ for a primary clarifier.

Table 4.1 Design Removal Efficiencies for Primary Sedimentation at Selfs Point

Analyte	Design Removal Efficiency
Chemical Oxygen Demand (COD)	25%
Biochemical Oxygen Demand (BOD)	25%
Suspended Solids	45%
Total Nitrogen	5%
Total Phosphorus	10%

Some of the other factors pertinent to the performance of primary sedimentation relevant to Self's Point are the length to width ratio of the tanks, whether there is aeration of the wastewater prior to primary sedimentation, the sludge blanket retention time and the characteristics of the influent.

4.3.3 Length to Width Ratio for Rectangular Tanks

The length to width ratio for rectangular primary sedimentation tanks has in the past been used as design tool, but is not now considered as reliable (WEF and ASCE 1992). The primary tanks at Selfs Point have a length to width ratio of 6:1, which is above the typically used range of 3:1 to 5:1 (WEF and ASCE 1992). There is some evidence to suggest that enhanced settling may occur in tanks of greater length to width ratios, where at least some of the influent sewage has been transferred by force main to the plant (Zlatko Tonkovic, pers. comm., 1997).

4.3.4 Preaeration of Wastewater

Preaeration of wastewater has been used over many years, principally to reduce odour, but it has been noted that it can assist settling removal efficiencies, by promoting "flocculation of finely divided solids into more readily settleable flocs" (WEF and ASCE 1992: 474). While preaeration times of 45 minutes or more are recommended to obtain increased BOD removal, there appears to be some impact at shorter periods, particularly at higher suspended solids concentrations (WPCF and ASCE 1977). At average dry weather flow the design detention time in the aerated grit chamber at Self's Point is 15 minutes, which is at the lower end of preaeration times considered to have an impact on BOD removal rates. The steep grade of the catchment also provides a degree of aeration within the sewer.

4.3.5 Impact of Solids Residence Time

The sludge retention time in the primary sedimentation tank can impact on suspended solids removal. Work by Albertson and Walz (1997) indicates that both suspended solids and COD removal efficiencies decrease with increased sludge inventory in the primary tanks. The results for COD were more erratic, but on average at a solids retention time (SRT) of 0.5 days removal of COD was about 50%, while at 2 days the efficiency averaged about 30%. Thus if operating a primary sedimentation tank in "activated" mode to increase VFA for phosphorus removal, then the total COD (TCOD) removal efficiency may be reduced when compared to simple solids removal operation. In a pilot scale activated primary tank operated with a sludge residence time estimated at 4 to 5 days the removal of suspended solids was 37% and total COD 19% (Hatziconstantinou, Yannakopoulos and Andreadakis 1996).

4.3.6 Impact of Freshness of Wastewater

The freshness of the wastewater can also affect the removal efficiency. Solids in stale wastewater generally settle less readily, due to biological degradation reducing particle size and the gel generated by biological reactions tending to cause particles to float (WEF 1996). The steep grades in the Selfs Point catchment result in relatively short sewer detention times and "fresh" sewage reaching the plant, favouring improved removal efficiencies.

4.4 Trickling Filters

4.4.1 Introduction

At Selfs Point there are two rock media trickling filters. The influent strength to the plant is greater than typical domestic sewage and the favourable BOD:TP and BOD:TN ratios indicated that there was BOD in excess of that required to achieve BNR without external carbon addition. This situation favoured the incorporation of trickling filters into the flowsheet of the upgraded plant (Morgan, Farley and Pearson

1997). All tenderers for the upgrade proposed use of the filters in the BNR upgrade and it is considered that the capital cost of the upgrade was reduced by over $1 million with their inclusion (HCC 1995b).

The retention of the trickling filters was seen to confer a number of actual and potential advantages (Morgan, Farley and Pearson 1997):

1. Reduced aeration tank volume as a result of a lower nitrification load;
2. Reduced aeration power costs;
3. Improved low temperature nitrification performance, due to a relatively smaller reduction in performance at lower temperatures of the trickling filters compared to the activated sludge system;
4. Increased ability of the plant to handle organic load shocks;
5. Possible reduction in sludge production;
6. Potentially better sludge settling properties compared to a process using only activated sludge;
7. Capability of treating high flows to secondary level prior to discharge; and
8. Increased flexibility of plant to handle future changes of influent characteristics.

A further potential advantage of trickling filters is that they may provide seed nitrifying bacteria for the downstream activated sludge process, which may reduce the minimum sludge age (USEPA 1993; Daigger et al. 1993). However, Parker and Tyler (1994) questioned the findings of a study assuming reliable nitrifier seeding of a downstream activated sludge process by a trickling filter. Trickling filters would not be the only source of seed as indicated by work noting that a significant quantity of nitrifying bacteria can be present in sewage entering a plant (Sandén et al. 1996).

Early experiences with the Selfs Point Plant with respect to these potential advantages have been described by Morgan, Farley and Pearson (1997). The reliable removal of BOD by trickling filters is acknowledged and in terms of the plant's nutrient removal performance the major impact of the trickling filters at dry weather flow is the nitrification capacity. Some of the factors affecting that capacity are discussed below.

4.4.2 Nitrogen Removal in Trickling Filters

4.4.2.1 Introduction

Trickling filters are noted as being very difficult to model and predictions of performance from theoretical considerations are only approximate (Harremoës, 1994;

Bell 1983). The sloughing of biofilm in a non-homogeneous manner, the diffusional limitations, changing flow paths and varying concentrations at different heights all contribute to the complexity (Schroeder 1983). While the understanding of biofilms is improving through microelectrode profiling and other techniques (for example Zhang and Bishop 1996), the models are still developing (Bishop 1997) and much of the information related to full scale performance is presently empirical.

The factors considered as having a major impact on nitrification performance of trickling filters are: organic load, hydraulic load, ammonia load, temperature, alkalinity, dissolved oxygen, presence of inhibitory substances and type of media (Henze 1995; Bitton 1994; USEPA 1993; Biddle and Wheatley 1992; Parker and Richards 1986).

4.4.2.2 Organic Load

Most of the literature indicates that nitrification rates in trickling filters drop with increase in organic load (USEPA 1993), though there are some instances where this has not been the case (O'Neill and Horan 1992). The reduction in nitrification is considered to be principally due to increased competition with heterotrophs. Higher organic load means faster growth in biofilm thickness and greater sloughing which disadvantages the slow growing nitrifying bacteria (Bitton 1994; USEPA 1993). The competition is reduced once the soluble BOD is reduced below 20mg/L, which can occur in the lower reaches of a trickling filter even if the feed has a high concentration (Mudrack and Kunst 1986). Full nitrification is noted as occurring when the soluble BOD is less than 10mg/L (USEPA 1993; Biddle and Wheatley 1992). Very low organic load, such as where tertiary nitrification follows a BOD removal only secondary process, can cause problems as the heterotrophs form biogrowth to which nitirifiers adhere and biogrowth formation and attachment is progressively more patchy at low load (USEPA 1993).

In terms of organic load it is noted by Arundel (1995) that at 0.1kg/m^3/day of BOD a high degree of nitrification can occur. A study summarised by the USEPA (1993) provides a wide range of results with a general reduction in nitrification with increasing organic load, which could be described approximately by a 15% drop in nitrification efficiency for each increase of 0.1kg/m^3/day in BOD load.

The above "rules of thumb" are based on the average daily load. Work by Zhang and Bishop (1996) indicates that it is not organic load per se, which reduces nitrification, but the competition for oxygen between heterotrophs and nitrifiers. Thus where carbonaceous load varies significantly during the day, but ammonium load is less variable, the nitrifiers may to some degree be advantaged, by having to compete less for oxygen during the low carbonaceous load periods

4 4 2.3 Hydraulic Load

The optimum hydraulic loading rates to promote nitrification are still unknown (WEF and ASCE 1992). Work by Okey and Albertson (1986a) noted that increasing hydraulic load increased ammonium oxidation rates, with application rates of greater than $1L/m^2/sec$ giving better results.

For trickling filters with low organic loads there generally appears to be an improvement in nitrification with recirculation, where the filter is hydraulically capable of accepting the higher flow without ponding occurring (USEPA 1993). Recycle is noted as increasing the ratio of dissolved oxygen to BOD applied, reducing the average BOD and providing additional scour (thus creating a more active surface) (USEPA 1993). Recirculation reduces the bulk ammonia concentration (that is in the liquid external to the biofilm), thus lowering the mass transfer driving force. and variations in hydraulic load are considered to be of lesser importance at lower NH_4^+-N concentrations (WEF and ASCE 1992). One potential benefit of regular high hydraulic load is that flushing can control predators, which may be consuming nitrifying bacteria and reducing the nitrifying mass (WEF and ASCE 1992).

4.4.2.4 Ammonia Load

The maximum ammonia load is often limited by the oxygen mass transfer rate and thus varies significantly depending on the organic load, trickling filter ventilation rates and biofilm thickness. Removal rates of up to $1.2gN/m^2/d$ are generally considered to be ammonium limited, while oxygen may be limiting above this load removal rates of up to $2.5gN/m^2/d$ have been observed (USEPA 1993; WEF and ASCE 1992). Values of 1.69 to 2 $56gN/m^2/d$ have been reported for nitrifying biofilms treating municipal wastewater at 20°C (Jansen, Harremoës and Henze 1995).

4.4.2.5 Temperature

Theoretically if temperature was a limiting factor for nitrification trickling filters should be more adversely affected by lower ambient temperatures than activated sludge due to the greater surface area for heat exchange with the atmosphere resulting in greater heat loss and wider temperature fluctuations (Parker et al. 1995). Plastic media with much lower heat capacity has been noted as being greatly affected by low ambient temperatures (Biddle and Wheatley 1992). However in practice the reduction in nitrification rates is much smaller than for activated sludge in the range 12-20°C (USEPA 1993), with Parker et al. (1995) finding a 24% reduction in nitrification when activated sludge equations predicted a 47% drop in performance for the measured temperature change. Arundel (1995) noted that a 15°C drop in temperature results in a 25% reduction in nitrification in trickling filters. Factors which are considered to be obscuring the effects of temperature on nitrification due to their greater relative impact include competitive heterotrophic activity, solids-sloughing cycles, predators, inhibitors in influent (WEF and ASCE 1992), diffusional effects (Okey and Albertson 1989) and alkalinity limitations (Zhang and Bishop 1996).

In very cold climates there is the problem of icing of the upper layer of the filters, but this tends to be more of a physical and operational problem which has to be overcome by appropriate measures, such as enclosure and forced ventilation of the filters (WEF 1996). Ambient temperatures are not low enough in Hobart for icing to occur. In mid-winter it has been noted that biofilm thickness tends to be greater due to lower grazing rates (Schroeder 1983; Mudrack and Kunst 1986), which may result in an increased mass of nitrifiers in the trickling filter during colder weather. This factor and the higher saturated dissolved oxygen concentration at colder temperatures may assist in maintaining nitrification performance at these temperatures.

4.4.2.6 Alkalinity and pH

The overall nitrification reaction can be described by (USEPA 1993):

$$NH_4^+ + 1.89O_2 + 0.0805CO_2 \rightarrow 0.0161C_5H_7O_2N + 0.952H_2O + 0.984NO_3^- + 1.98H^+$$

This equation indicates a substantial destruction of alkalinity through the production of hydrogen ions and is equivalent to 7.1g of alkalinity (as $CaCO_3$) per g of NH_4^+-N nitrified (USEPA 1993). The destruction of alkalinity reduces pH and if the pH is too low nitrification is inhibited. Szwerinski, Arvin and Harremoës (1986) found that decreasing the pH below 7 in a biofilm reactor led to a decrease in the ammonia oxidation rate. The theoretical equation they derived to describe their findings implies that bicarbonate becomes limiting if the mole ratio of $HCO_3^-:O_2$ drops below 2.4. Zhang and Bishop (1996) investigated this further and found that when this ratio was below 3 the pH could drop by 1.4-1.6 units within the biofilm and that nitrification in the biofilm stopped when a pH of 5.8 was reached. The cessation of nitrification was noted by the presence of oxygen and ammonium substrate fully penetrating the biofilm.

Thus even if there appears to be adequate alkalinity in the bulk liquid, there may be insufficient in the biofilm due to mass transfer limitations (Boller et al. 1994; Henze 1995). Siegrist and Gujer (1987) note that if the effluent alkalinity is 50mg/L (as $CaCO_3$) or greater significant effects due to low pH are less likely to occur.

4.4.2.7 Oxygen Concentration and Mass-Transfer

In trickling filters where the applied fluid has a high oxygen demand, oxygen transfer can be a limiting factor for nitrification as the dissolved oxygen in the influent wastewater is small compared to the oxygen demand. Temperature and humidity differences can provide sufficient air flow to provide the necessary oxygen if the filter has a low load and adequate open passageways (USEPA 1993) The air flow requirements are given by (USEPA 1993):

$$\text{Air flow} = 150 \times (1.25 \times BOD_5 + 4.6 \times TKN) \times (PF)/1{,}440\ m^3/min$$

where BOD_5 and TKN are in kg/day and PF is a peaking factor

Positive ventilation through blowers may be required, if oxygen demand is high, to reduce this limitation (Parker et al. 1989). The trickling filters at Selfs Point have natural ventilation and at high organic loads there could be a problem with insufficient ventilation.

4.4.2.8 Trickling Filter Medium

Rock media trickling filters generally appear to have greater nitrification rates for a given media surface area than vertical or randomly packed plastic media (USEPA 1993). This better performance does not necessarily hold on a volumetric basis as plastic media typically has a greater specific surface area. The longer liquid residence time (reducing BOD concentrations in the lower section of the filter), reduced shear stresses (reducing sloughing rates and thus favouring nitrifiers) and reduced propensity to short circuiting have been considered as contributing to the better performance on a surface area basis (Wilderer, Hartmann and Nahrgang 1983). Rock media also has greater specific heat capacity and filters using this material are thus less affected by ambient temperature fluctuations (USEPA 1993).

4.4.2. Denitrification in Trickling Filters

While not generally noted in the literature as being significant there is some evidence of nitrogen removal in excess of metabolic uptake in trickling filters, which are performing a combined carbonaceous removal and nitrifying duty. Hertle and Hartley (1994) found that denitrification efficiencies of 27-65% expressed as reduction of influent ammonia occurred in a low loaded trickling filter with thick biomass growth. Denitrification has been noted as only occurring in biofilms with a thickness of greater than 3mm (Mudrack and Kunst 1986). Denitrification can be performed in trickling filters if air flow to the filters is restricted and COD and nitrate are applied to the filter (Rüdiger and Sekoulov 1992). The design for Selfs Point predicted a drop in total nitrogen over the trickling filters from 34mg/L to 30mg/L (ANI-Kruger 1996), indicating that in the design virtually no denitrification was assumed.

4.4.2.10 Inhibition

Inhibitory substances are known to be toxic to nitrifying bacteria and can reduce or completely stop nitrification (Bitton 1994) Most of these substances are typically discharged by industrial sources and tests are available to assess the inhibitory effect of such substances, such as ISO 9509 (Grüttner et al. 1996). Trickling filters tend to be more robust than suspended growth systems due to the short contact time, but the instantaneous effluent quality is affected to a higher degree when a short duration, high concentration peak occurs in the influent (USEPA 1993).

4.4.3 Other Plants Using Trickling Filters in BNR Layout

Several flowsheets have incorporated both trickling filters and activated sludge in BNR plants. The Orange County Water and Sewage Association (OWASA) in North Carolina retrofitted an existing plant, which had trickling filtration in roughing duty followed by an activated sludge plant. One primary sedimentation tank was converted into a fermenter and the supernatant from this tank was bypassed to the head of the activated sludge process. The plant has demonstrated good phosphorus removal performance (Randall, Stensel and Barnard 1992). The treatment train has been patented as the OWASA™ process (WEF and ASCE 1992) and has been used in several other plants with trickling filters undergoing BNR upgrades, such as Mason Farm in the USA (Bundgaard et al. 1993). Barnard (1994) comments that bypassing some settled sewage around the trickling filters to the anaerobic zone of the activated sludge process would be more efficient for nutrient removal and that this method had been used in the Potchefstroom plant in South Africa some years earlier.

The Ebjy Mølle Wastewater Treatment Plant at Odense, Denmark (60ML/day capacity) uses this latter approach although the two sets of trickling filters are operated in series with intermediate settling. This flowsheet was selected from several options and was considered to have the lowest chemical consumption and least capital cost, due to the utilisation of many existing structures (Christensen 1990). The Ebjy Mølle plant uses the BioDenipho™ process, so is in some ways similar to Selfs Point, however at Selfs Point there is no intermediate settling for the trickling filters, there is sidestream prefermentation and the waste activated sludge is anaerobically digested. The Ebjy Mølle plant achieves an effluent of total nitrogen of 6mg/L and 1mg/L of total phosphorus, prior to tertiary filtration, with ferrous sulphate to support the biological phosphorus removal (Christensen 1990). The proposed flowsheet for the Bolivar Wastewater Treatment Plant is Adelaide, which is to be built by 2001, is very similar to that at Selfs Point for similar effluent criteria, with modelling indicating significant capital cost savings (Yerrell et al. 1997).

These process formats in a similar fashion to Selfs Point all sacrifice the RBCOD which goes to the trickling filters for the nitrification capacity of the filters, reducing both the BOD:TP and BOD:TN ratios and adversely impacting on biological nutrient removal potential (Ekama and Wentzel 1997). At Selfs Point the strong industrial component in the influent provides excess BOD and while the organic load to nutrient ratios are reduced using the trickling filters they are still adequate for nutrient removal.

One alternative to overcome this loss of BOD is the DEPHANOX process where a sidestream of the mixed liquor is settled and the overflow directed to trickling filters, which nitrify without consuming the RBCOD in the process (Bortone et al. 1996). It is considered that the process would reduce the required sludge age and significantly improve settleability. While the process shows some promise it has yet to be proven at full scale (Ekama and Wentzel 1997).

In summary, it appears that relatively few plants have incorporated trickling filters into a BNR flowsheet and that the process flowsheet for Selfs Point is used in few other locations around the world.

4.5 BioDenipho ™ - Phased Isolation Ditch Nutrient Removal Process

4.5.1 Introduction

The BioDenipho™ system is the BNR process used at Selfs Point. This system is generically described as phased isolation ditch technology (Tetreault et al. 1986). The process is a combination of oxidation ditch and sequencing batch reactor technologies (WEF and ASCE 1992). Some versions of the process use tanks both as reactors and for settling in a similar manner to a sequencing batch reactor (for example the Type T system) (USEPA 1993). The process configuration at Selfs Point has a system of three ditches with two alternately fed and the overflow from each of these two tanks passing to a third aeration tank from where the mixed liquor flows to separate secondary clarifiers (Morgan and Farley 1998). The installation at Selfs Point is the first time this particular configuration has been constructed. Numerical modelling of the configuration indicates that it has the potential to provide more efficient nitrogen removal than a parallel three ditch arrangement (Dines Thornberg, pers. comm., 1997).

4.5.2 Biological Phosphorus Removal

The biological phosphorus removal enhancement to the nitrogen removing BioDenitro™ to form the BioDenipho™ system is much the same as other single sludge systems not using chemicals. An anaerobic zone is incorporated where return sludge and primary influent are mixed to achieve phosphorus release (WEF and ASCE 1992). The phosphorus released in the anaerobic zone is then principally taken up under aerobic conditions (USEPA 1987), although phosphorus uptake in anoxic conditions has also been found to occur at about 50-60% of the aerobic rate (Kuba, van Loosdrecht and Heijnen 1996). The critical factors in ensuring good EPBR appear to be providing adequate VFA or other suitable RBCOD under strictly anaerobic conditions (i.e. no oxygen or nitrates) prior to the mixed liquor entering the anoxic/aerobic zones and ensuring that anaerobic conditions do not occur at other places in the process where the VFA is not available (USEPA 1987).

The BioDenipho™ process has proved capable of achieving low phosphorus effluents without the use of precipitants, as evidenced by the performance of the Viby plant in Denmark, where the effluent averages 0.4mg/L total phosphorus without precipitant chemicals (Sørensen et al. 1998).

In general the EBPR mechanism appears to be more tolerant of environmental factors, such as temperature, than nitrification (Stensel and Barnard 1992) and thus these factors are not discussed in this study. However, the sludge age of a single sludge BNR activated sludge plant is usually determined by nitrogen removal considerations (USEPA 1993) and this factor has been noted as affecting EBPR. A bench scale study has found that as the sludge age increases above 10 days there is a decrease in the proportion of PAOs and that the maximum phosphorus removal and VFA uptake rates decline (Rodrigo et al 1996). Thus an extended sludge age is likely to reduce EBPR performance.

The EBPR process is noted as being adversely affected by low pH and work on an acetic acid containing wastewater with a pH of 4.5 found the anaerobic zone to experience a pH of 5.5 and that EBPR could not be established (Randall, Stensel and Barnard 1992).

4.5.3 Nitrogen Removal

4.5.3.1 BioDenitro™ Process

In the BioDenitro™ process for a simple two tank system, the fed tank is usually in anoxic phase, undergoing denitrification, while the other tank is usually being aerated for nitrification. The proportions of aerobic and anoxic phases can be adjusted to suit conditions with the temporal fractions of nitrification and denitrification being variable from 0-100% (ANI-Krüger 1996). This permits the use of on-line measurements and advanced control strategies to optimise nitrogen removal on an almost real time basis as has occurred at several plants in Denmark (Thornberg, Nielsen and Eriksson 1998).

The process is similar to other oxidation ditches requiring no external recirculation, as there is high internal recycle of nitrates with a recycle ratio of about 100 to 200 (Hartley 1997; Sorensen, Thornberg and Nielsen 1994). Oxidation ditches can achieve nitrogen removals of over 90%, due to this internal recycle (WEF 1996), and this level of removal has been achieved at an intermittently decanted aerated lagoon plant at Sorell, Tasmania (Morgan and Farley 1998a). The BioDenitro™ process is noted as being capable of consistently achieving a total nitrogen concentration in the effluent of 6mg/L, but not 3mg/L on a reliable basis without post denitrification (USEPA 1993).

Nitrogen removal in the BioDenipho™ process is subject to many of the same factors as other single sludge suspended growth processes, including DO concentration, temperature, alkalinity, carbon to nitrogen ratio of the feed, presence of inhibitors, aerobic sludge age and diffusional limitations associated with mass transfer within flocs (USEPA 1993; Henze 1995; Stensel and Barnard 1992).

4.5.3.2 Nitrification Usually Limiting in Nitrogen Removal

Biological nitrification tends to be more sensitive than denitrification to environmental factors and is usually the limiting step in nitrogen removal, if there is sufficient carbon for denitrification (USEPA 1993). The anoxic and aerobic conditions required for denitrification and nitrification can be controlled by aeration, the sludge age by appropriate wasting rates and reactor volume and the carbon to nitrogen ratio is usually adequate in domestic wastewaters to achieve denitrification (USEPA 1993). The diffusional limitations in activated sludge, while important when considering simultaneous nitrification denitrification (for example de Beer et al. 1998), generally tend to be much less significant than in fixed film processes. With only one major industry in the Selfs Point catchment, there is considered to be limited external sources of inhibitory chemicals, thus the factors of most importance in achieving nitrogen removal at the Selfs Point Plant are considered to be temperature and alkalinity, while digester filtrate may be a source of inhibition (Bitton 1994).

4.5.3.3 Temperature

Nitrification rates are strongly affected by temperature, due to the reduction in maximum specific growth rate of nitrifying bacteria. The relationship has been described by Arrhenius type equations with the maximum growth rate halving with a 10°C drop in temperature (USEPA 1993). The minimum design mixed liquor temperature for Selfs Point is 12°C and an aerobic sludge age of 9 days has been selected for winter conditions (ANI-Krüger 1996). This value appears to be typical for 12°C (USEPA 1993). Denitrification rates also reduce with temperature but this issue is generally of less significance in nitrogen removal design considerations than nitrification requirements for sufficient sludge age (USEPA 1993).

4.5.3.4 Alkalinity and pH

Nitrification rates decrease rapidly as pH moves into the acid range, with a 50% reduction in rate for a drop in pH from 7 to 6 (USEPA 1993). For low alkalinity influent wastewater this is a concern because of the destruction of alkalinity in the nitrification reactions, as discussed in Section 4.4.2.5. Denitrification generates about 3.5g of alkalinity (as $CaCO_3$) per g of NO_3^--N denitrified, thus recovering about half of the alkalinity destroyed in nitrification (USEPA 1993) and this assists in maintaining pH. Once the alkalinity drops below 40mg/L (as $CaCO_3$) the pH tends to become unstable, leading to poor nitrification and there is a tendency towards bulking sludge (Ekama and Marais 1984). The relatively low alkalinity of the influent to Selfs Point was considered as having the potential to cause problems, particularly during periods of high ammonium load (Ray Farley, pers. comm., 1997). There is evidence that bacteria can acclimate to a pH in the range 6.0 to 6.5 under steady conditions, when abrupt changes would significantly reduce the nitrification rate (USEPA 1993).

4.5.3.5 Aerobic Sludge Fraction

Experience with laboratory scale nitrogen removal plants tends to support the opinion that at high unaerated mass fractions the process is prone to bulking, particularly at low temperatures, with DSVI > 150mL/g, whereas if the fraction is below 0.5 there are less problems (Ekama, Wentzel and Marais 1990; Ekama and Marais 1984). With a bulking sludge at peak flowrates there is the potential for sludge blanket to build up in the secondary clarifiers and thus increase the unaerated mass fraction of the BNR biomass. This is a concern as BNR plants appear to be prone to bulking with filamentous bacteria at lower temperatures, particularly when *Microthrix parvicella* is present (Wanner 1994).

4.6 Prefermentation

4.6.1 Description and Background

To achieve enhanced biological phosphorus removal, it is considered essential that there is adequate RBCOD, usually described as short chain volatile fatty acids (SCVFA) (Barnard 1984). There may be sufficient RBCOD in the plant influent due to the presence of a favourable industrial input to the sewer system as at Secunda WWTP in South Africa, where the plant received a wastewater rich in acetic acid (Randall, Stensel and Barnard 1992) or due to long sewer retention times resulting in fermentation of the organic content of the sewage (Barnard 1992), such as at the Klerksdorp Plant in South Africa, which is fed by a 3km long force main (Barnard

1990). Alternatively the substrate can be externally dosed, such as by the addition of acetic acid (Henze 1995a).

At Selfs Point the sewer retention times are short and the industrial input is not favourable. Milk waste, with its high protein and fat contents tends to be more difficult to ferment than typical domestic wastewater (Hanaki et al. 1987). Thus it was anticipated that fermentation of primary sludge would be required to achieve enhanced biological phosphorus removal (ANI-Krüger 1996). Prefermentation or sludge hydrolysis has been shown to significantly increase EBPR (Ekama and Wentzel 1997; Randall et al. 1997) The aim of prefermentation is generally to achieve about 50mg/L of VFA (as acetic acid) to ensure sufficient is available to remove the typical phosphorus load (Barnard 1992).

Prefermentation is the anaerobic conversion of more complex substrates into simple organic molecules. This conversion can be considered as two steps, hydrolysis (or depolymerisation) and fermentation (Gujer and Zehnder 1983). The first step is the conversion of particulate organic matter, such as proteins and carbohydrates into amino acids and sugars, while the second step is the conversion of these simpler molecules into VFAs and byproducts. The second step can be split into acidogenesis and acetogenesis, for which volatile acids and acetate are the products (Bitton 1994). With typical municipal wastewater sludge the conversion of these steps proceeds if the sludge is held for adequate time. Methane formation only occurs at extended sludge ages (Bitton 1994), though prefermenters may require occasional sludge replacement to prevent buildup of the slower growing methanogenic bacteria (Barnard 1992).

One method to promote fermentation is to increase the size of the anaerobic zone (Sudiana et al. 1997), but a large anaerobic zone has now found to generally be counterproductive at an anaerobic fraction of greater than 0.15 (Randall, Stensel and Barnard 1992) and recent work indicates that the anaerobic zone should be kept as small as possible to reduce secondary release of phosphates of which only 40 to 60% are taken up under aeration (Barnard 1992; Stephens and Stensel 1998). Relative to the rate of VFA uptake during phosphorus release by PAOs the rate of fermentation is about an order of magnitude lower and unable to provide fermentation products sufficiently rapidly (Danesh and Oleszkiewicz 1997). The anaerobic zone at Selfs Point is about 0.10 of the total BNR reactor volume (ANI-Kruger 1996).

The use of supernatant from a separate acid phase digester to promote EBPR was suggested by Pitman et al. (1983). Barnard (1984) described the concept of activated primary sedimentation tanks, in which a sludge blanket is built up to provide anaerobic conditions suiting the formation of VFAs Prefermenters have now become a recognised method of supporting BNR processes and are in use on many plants in a number of different countries (Rabinowitz and Barnard 1997)

An important factor is the timing of the addition of any hydrolysate. At Frederikssund WTP in Denmark, some of the hydrolysate carbon source is retained for Sundays when the influent has a reduced soluble COD (SCOD), where it has been found to support EBPR during the day of low load (Andreasen et al. 1997). It has also been noted that short periods of low organic load can deplete internal PHB stores of PAOs and that while the anaerobic phosphorus release process responds virtually immediately, the phosphorus uptake response is delayed as the polyhydroxybutyrate (PHB) stores rebuild at a slower rate (Temmink et al. 1996). Thus it would appear that hydrolysate addition would have a more marked effect if it occurred during periods of low organic load.

4.6.2 Types of Prefermenters

There is a range of prefermentation methods, which can be placed in one of two categories:

1. Sidestream fermenters fed with primary settled sludge; and
2. Activated primary sedimentation tanks.

These two categories provide the same function with primary sludge in anaerobic conditions for sufficient time to create short chain volatile fatty acids (SCVFA). The sidestream fermenters remove the sludge which is collected in the primary sedimentation tanks into one or more separate reactors, while the activated primary tanks build up a sludge blanket, some of which is usually mixed with influent to elutriate any soluble COD or VFAs which have been generated (Christensson et al. 1998; Jönsson et al. 1996). The sidestream reactors can be further categorised as:

1. Complete-mix;
2. Static prefermenter/thickener; and
3. Complete-mix with dedicated thickener.

The last of these three categories is considered to be the most efficient (Oldham 1994), although work by v. Münch (1998) indicates that based on actual operating results there is little difference in performance. The activated primary tank system can be an inexpensive retrofit and may only require a change to operating procedures to increase the sludge blanket in the primary tanks and installation of a system to transfer of some sludge to the influent for elutriation of VFAs (Raper et al. 1997). Christensson et al. (1998) note that there appears to be significantly fewer studies of activated primary operation than for off-line or sidestream fermenters. Some disadvantages of activated primary tanks compared to a sidestream fermenter are that the:

- process can be adversely affected during high flows caused by wet weather (Stevens and Oldham 1992);
- environmental factors such as temperature, HRT and pH are more difficult to control (Stevens and Oldham 1992; Danesh and Oleszkiewicz 1997); and
- primary effluent concentration of VFAs varies with flow, thus at high flows the additional VFA concentration is low (Christensson et al. 1998).

Oldham (1994) asserts that in general activated primary tanks operation is less efficient than sidestream fermentation, although there are plants which achieve good phosphorus removal with the aid of activated primary tanks (Raper et al. 1997; Christensson et al. 1998).

4.6.3 Temperature and Solids Residence Time

These two factors are discussed together as the VFA yield for a given temperature appears to be related to the solids residence time and vice versa. The rate of VFA generation appears to be significantly affected by temperature. Oldham et al. (1992) found that rates of fermentation were 60% higher at an ambient temperature of 28°C compared to 15°C at the Penticton Plant in Canada; the reactor temperatures were not provided. Kristensen et al. (1992) found that at an HRT of 2 days soluble COD production at 30°C was 13% while at ≤15°C the rate was 8% as a proportion of total COD entering the fermenter. Results by Bundgaard et al. (1993) indicated that at 30°C, the yield of soluble COD reached a maximum after 2 days at 11-15%, while at 15°C the yield was about 9% after three days and still increasing.

Barnard (1992) notes that SCVFA production increases with SRT for at least 10 days, but that 6-8 days appears to be optimum for EBPR, though no mention is made of the temperature at which this applies. Wentzel et al. (1988) found in batch tests that the maximum yield of VFA (0.125 kg VFA as COD/kg initial VSS as COD) occurred at a sludge age of 6-9 days, with a 60% yield at 3-4 days. Raper, Crockett and Glover (1994) indicate that a sludge age of 3-7 days appears to be typical from the literature. Work by Bliss, Barnes and Evans (1989) indicates that at a sludge age of 8 days VFA generation was still significant, with the average rate during the second 4 days about 60% of that in the first 4 days. At Nambour STP in Queensland 8 to 12 days was found to be the optimum SRT (Cusack, Gloag and Stevens 1997). While not directly related to SRT the average reactor solids content may be affected by SRT design considerations. Eastman and Ferguson (1981) noted that the solubilisation of particulate organic carbon did not appear to be affected by varying solids concentrations of up to 6% volatile solids.

Thus for the typical reactor temperatures occurring at Selfs Point of 12-20°C an SRT of about 6 days or slightly longer would appear to provide close to optimum VFA production. The original process design of the complete mix fermenter at Selfs Point called for an average SRT of about 1-2 days (ANI-Krüger 1996).

4.6.4 Effect of Hydraulic Residence Time

At the Penticton plant in Canada, although the effluent VFA concentration dropped slightly, the mass rate of VFA production was increased by 15% and the soluble COD by 19% when the HRT was dropped from 28.4 to 20.8 hours (Oldham et al. 1992). In a laboratory scale study of acidogenesis Elefsiniotis and Oldham (1994) found that the optimum HRT for VFA production was 12 hours. In a study of prefermenters in Canada and Australia, v. Münch and Koch (1997) found that on average that Canadian prefermenters tended to have better performance compared to those in Australia. It is interesting to note that the HRTs for the Canadian prefermenters were all in the range of 6 to 16 hours, while the Australian prefermenters tended to longer HRTs. The original process design for Selfs Point included a complete-mix fermenter of 1.5 days HRT, same as the SRT. Christensson et al. (1998) considered that the short HRT of an activated primary may assist in minimising methane generation and Eastman and Ferguson (1981) found that maintaining a short HRT was an important factor in reducing methanogenesis in their work on fermentation.

4.6.5 Impact of pH

Eastman and Ferguson (1981) found that the rate of solubilisation of particulate organic carbon into soluble COD was 42% greater at pH 6.67 compared to that at a pH of 5.15. Gas production was noted as being relatively small in both cases, although a lower rate occurred at the higher pH. A study by Danesh and Oleszkiewicz (1997) found that VFA production was about 30% lower at a pH of 6.1-6.4 compared to that at 7.0-7.6 for SRTs of 8 and 13 days, but was 90% less for an SRT of 4 days. The higher of these pH ranges may only be relevant for fermentation in activated primary tanks due to their high dilution keeping the pH near neutral. Product inhibition has been observed due to pH depression and high concentrations of VFAs (Zoetmeyer et al. 1982). Thus it appears that VFA production rates decrease as pH drops and the production of VFAs can be self limiting. Shorter HRTs, which would result in greater dilution of the liquid phase and remove the acids being generated, would thus appear to favour greater specific VFA generation rates.

4.6.6 Impact of Mixing

Both static thickener/prefermenters and complete mix fermenters are used on full scale plants. Due to the trapping of liquid and thus fermentation products in the sludge blanket reducing concentration gradients and locally lowering the pH it would appear that the static fermenter would be less effective. A comparison between the well performed static fermenter at Kelowna and the complete mix fermenter at Penticton by Stevens and Oldham (1992) appeared to indicate that the complete mix fermenter produced about 50% more VFAs under similar ambient conditions. Work by v. Münch and Koch (1997) indicates that by some measures the static fermenter at Kelowna is more efficient than some complete-mix systems. Generally mixing is considered to have a beneficial impact on VFA production (v. Münch 1998).

4.6.7 Other Factors Impacting on Prefermenter Performance

The prefermentation of primary sludge is noted as causing a buildup of fibrous sludge (Barnard 1992), which may require periodic flushing of the system of the sludge. In work by Cha and Noike (1997), operating an anaerobic reactor over an extended time, bacteria with long slender shapes were noted at an HRT of 48 hours at both 15°C and 30°C and also at an HRT of 6 hours at the higher temperature but were much shorter at 15°C. If the fibrous sludge is related to this bacterial morphology, then a shorter HRT may assist in preventing the buildup of these fibres, though whether the sludge form and bacterial shape are related may be problematic.

As well as providing VFAs the prefermentation process may assist downstream processes by releasing metal ions such as magnesium and calcium, which are implicated in phosphorus removal mechanisms (Christensson et al. 1998).

The additional VFA has also been demonstrated to enhance denitrification rates, thus if the VFA production is greater than that required for bio-P removal or phosphorus removal is not required the hydrolysate can be used to enhance denitrification (Brinch et al. 1994; Hatziconstantinou, Yannakopoulos and Andreadakis 1996).

Fats are noted as being difficult to degrade and proteins only partially degraded in the conditions existing in acid phase fermenters (Fox and Pohland 1994; Elefsiniotis and Oldham 1994). This is an important consideration at Selfs Point where a significant proportion of the BOD is from the milk processing facility

4.6.8 Effects of Prefermentation on Anaerobic Digestion

The fermentation reactions are considered to increase the process stability and improve the biodegradation kinetics of the downstream methanogenic anaerobic digestion process (Fox and Pohland 1994). A potential source of the additional stability is the modification of the more complex influent constituents, such as those compounds containing double bonds into simpler compounds (for example Komatsu, Hanaki and Matsuo 1991). It has been observed by Andreasen et al. (1997) that gas production in anaerobic digesters dropped by 25% with the advent of primary sludge hydrolysis.

4.7 Secondary Clarification

4.7.1 Introduction

Secondary clarifiers are the settling tanks used to separate activated sludge and effluent. The theory of settling has developed over many years and settling theory as it applies to secondary clarification is well covered in the literature (see Ekama et al 1997; Albertson 1992; WEF and ASCE 1992).

As this study is focused on nutrient removal, there are three issues which have been selected for discussion which apply to BNR plants in general and Selfs Point in particular. These issues are:

1. The applicability of the different sludge volume index (SVI) measurements to assessing settleability and bulking;

2. The problems associated with *Microthrix parvicella* and factors which may be important in its proliferation; and
3. Denitrification in the clarifier sludge blanket.

4.7.2 Measurement of Settleability and Bulking

Secondary clarifier design generally uses the SVI as one of the selected design parameters (WEF and ASCE 1992). While flux theory is used in design of the clarifiers, its detailed use requires the extensive determination of zone settling velocities over a range of solids concentrations and SVI is more commonly used instead as it is far more readily measured (Bye and Dold 1998). The SVI has been incorporated as one of the parameters used in clarifier operating diagrams (Hermanowicz 1998; Daigger and Roper 1985).

The SVI is calculated from measurements of the suspended solids concentration and the settled sludge volume (APHA and WEF 1995). There are however several options for measuring the settled sludge volume, including different time periods, using dilution, using stirring and by combinations of these methods (Bye and Dold 1998; Albertson 1992a). There is some controversy as to which SVI measurement best approximates the conditions in the clarifier sludge zone (Albertson 1992a). For mixed liquor suspended solids (MLSS) of greater than 1200mg/L at an SSVI of 80-85mL/g, the unstirred procedure was found to provide unreliable results, whereas the stirred SVI compared well to the maximum free settling velocity (Rachwal et al. 1982). Thus the unstirred procedure, if undiluted, would not appear to be useful as the MLSS encountered in BNR plants is usually higher than 1200mg/L. The DSVI and SSVI are both recommended as acceptable for an MLSS in the range 1500 to 5000mg/L (Albertson 1992a).

Bye and Dold (1998) have reviewed the stirred SVI, diluted SVI and stirred SVI at 3.5g/L. They note that the test conditions (column height, solids settleability and compactability) can have a marked effect on the final result and that the stirred SVI can be affected by the starting solids concentration. They also found that the diluted SVI removes the effect of solids concentration, but the measurement becomes one of compactability rather than settleability, but that it may be a better measure to assess bulking conditions.

At Selfs Point the SVI measurements have generally been used as a relative indicator of settling properties over time, rather than an exact quantitative assessment which could be used to compare with the performance of other plants (Ray Farley, pers. comm., 1998).

4.7.3 Bulking, Foaming and Microthrix parvicella

4.7.3.1 Bulking

To achieve very low suspended solids effluent with secondary clarifiers the prevalence and type of filamentous bacteria are important factors. While bacteria forming flocs provide larger particles with greater settling velocities, without filamentous bacteria to provide a "backbone", smaller flocs which have been fragmented, may not settle out (Arundel 1995). This condition is known as pin floc and can be indicative of very low filamentous bacteria numbers (Wanner 1994). However large number of filamentous bacteria can cause a condition known as bulking, where sludge has poor compactability. There is a range of filamentous bacteria types which, in excess, can result in settling problems in the secondary clarifiers (Wanner 1994).

4.7.3.2 Foaming

Biological foaming at wastewater treatment plants is usually less of a problem than bulking but can impact on effluent quality, cause health hazards, odours and anaerobic digester problems (Bitton 1994). Pagilla, Craney and Kido (1997) noted that excessive levels of *Nocardia* in the feed sludge increased the foam layer in anaerobic digesters, particularly if the digester was gas mixed. It has been noted that at least some of the foam causing organisms probably cannot survive in the anaerobic digester environment, but that nonviable filaments still have the ability to form foam (van Niekirk et al. 1987).

While a number of mechanisms have been proposed for foam formation none has been proven, though it does relate to hydrophobicity of the cell walls and biosurfactants produced by the associated organisms (Stratton and Seviour 1997; Lemmer 1986). *Microthrix parvicella* is noted as being one of the two most common bacteria observed in foams (Stratton and Seviour 1997).

4.7.3.3 Factors Affecting the Abundance of *Microthrix parvicella*

Early results of filamentous bacteria identification from Selfs Point following the upgrade to BNR, indicated an early dominance by a rosette form of type 0803, but this was replaced by *Microthrix parvicella* in early winter of 1997 (Farley, Lucas and Morgan 1997). *M. parvicella* is found in many BNR plants around the world and can cause both severe bulking and foaming problems (Pitman 1996; Eikelbloom, Andreadakis and Andreasen 1998).

The conditions supporting proliferation of *M. parvicella* are complex, but there appears to be a number of factors which favour its growth to the level where it causes problems. These factors include:

1. Aeration tank temperatures of less than 15°C (Pitman 1996; Eikelbloom, Andreadakis and Andreasen 1998);
2. Primary sedimentation prior to the biological process (Andreasen and Sigvardsen 1996);
3. Primarily domestic wastewater influent with a low industrial fraction (Bitton 1994; Eikelbloom, Andreadakis and Andreasen 1998);
4. Alternating anoxic and aerobic conditions (Wanner 1994);
5. Longer sludge ages and low F/M ratios (Westlund, Hagland and Rothman 1996; Knoop and Kunst 1998), and
6. Complete mix reactors such as oxidation ditches appear to be more favourable than plug flow configurations (Mamais et al. 1998)

In a study of substrate uptake by filamentous organisms, Andreasen and Nielsen (1997) found that *M. parvicella* had taken up oleic acid, but none of the simple substrates tested, which included acetate, glucose and ethanol Eikelbloom, Andreadakis and Andreasen (1998) noted that *M. parvicella* uses fatty acids and that these are not usually present in fresh wastewater, but are generated by hydrolysis as is often used at BNR plants to generate VFAs. Work by Hagland, Westlund and Rothman (1998) on sewage treatment plants in the greater Stockholm area concluded that there was a clear relationship between high sludge retention time in the secondary clarifiers, VFA production in the RAS and a high abundance of *M. parvicella*. Thus the use of prefermentation would appear to favour the proliferation of *M. parvicella*.

At Selfs Point all of the foregoing conditions apply occur due to either environmental conditions during the winter months or as an integral part of the process design or are catchment characteristics, thus most of the options which are usually available for control of *M. parvicella* can not be used Keeping the sludge blanket at a low level does appear to be a control method available which may reduce proliferation.

Work by Pitman (1996) suggests that the removal of scum traps by modifying the plant physically can reduce the prevalence of *M. parvicella*. Cha et al. (1992)

demonstrated that foam trapping was a significant factor in proliferation of *Actinomycetes spp.* Removal of the scum or foam from the process by dewatering or disposal is recommended due to the potential for reseeding if the scum is returned to the activated sludge system (Pitman 1996; Wanner 1994; Bitton 1994).

During the colder months at Selfs Point it would appear that conditions are close to ideal for the proliferation of *M. parvicella* and that it will be necessary to consider this factor in winter operation of the plant.

4.7.4 Denitrification in Secondary Clarifiers

The presence of nitrates in the anaerobic zone of a EBPR plant is known to adversely affect the phosphorus removal mechanism by permitting anoxic conditions, which allow the denitrifying organisms to outcompete the PAOs for readily available substrate (Stensel and Barnard 1992). An anoxic zone can be used ahead of the anaerobic zone to eliminate nitrates from the return activated sludge through endogenous denitrification (Barnard 1990), or alternatively this can be carried out in the clarifier sludge blanket (Shehab et al. 1996).

At Selfs Point there is no pre-anoxic zone and there is a reliance on sufficient denitrification occurring in the clarifier sludge blanket. A study by Thomas, Ostarcevic and Bliss (1997) describes the development of controlled denitrification using the clarifiers. If there is insufficient sludge blanket depth, nitrates can enter the anaerobic zone, while excessive depth can result in phosphate release without carbon source uptake reducing the efficiency of the EBPR mechanism (Thomas, Ostarcevic and Bliss 1997). At the Vereeniging plant near Johannesburg the sludge blanket was kept high to reduce nitrates, but resulted in a release of up to 12mg/L phosphorus (Ekama et al. 1997). The depth of the blanket is thus critical and requires good control.

Shehab et al. (1996) found at the Ann Arbor WWTP in the USA, which uses the A/O process, that a blanket depth of 2-3 ft (0.6-0.9m) gave adequate denitrification without excessive phosphorus release. Thomas, Ostarcevic and Bliss (1997) calculated a denitrification potential for the clarifier sludge blanket in their study of St Marys BNR Plant in New South Wales, which was shown to increase with depth and decrease at higher RAS flowrates. A sludge blanket depth of 1.7 to 2.25m was found to be effective.

Work by Ridgely (1998) models the impact of sludge blanket level on nutrient removal plant performance and notes that in plants designed for full nutrient removal the sludge blanket has to be taken into account in the design. The modelling in the paper is based on similar design parameters to those which occur at Selfs Point of 10ML/day and 15°C. For a carbonaceous removal or nitrification only plant the sludge blanket level is shown to have relatively little impact particularly if there is no primary settling. Where reactor volume is small relative to clarifier area however the assessment predicts a very high unaerated mass fraction if the sludge blanket is 1 metre deep, due to the large mass of sludge in the clarifier. In the example provided a 1 metre sludge blanket would increase the unaerated mass fraction to 0.68 and the actual sludge age to 21.5 days and the resulting aerobic sludge age of 7 days would be insufficient to sustain nitrification.

Ridgely (1998) notes that this effect is likely to be significant where:

1. There is pre-treatment such as primary settling or other organic load reduction;
2. There is 'weak' sewage due to infiltration or household pre-treatment;
3. Minimum sludge age, nutrient removal facilities are used in warm climate conditions,
4. Plant design provides for full treatment to a high hydraulic peaking factor; and
5. Plants have been designed to run under high MLSS conditions, where solids flux is limiting and the surface loading rate is conservative.

At Selfs Point both primary sedimentation and trickling filtration provide organic load reduction, the plant is designed for a 2.5 times hydraulic peaking factor and the winter design MLSS of 4500-5000mg/L results in the solids flux being the limiting factor. Thus three of the above five points are relevant to Selfs Point. An additional factor is that trickling filter nitrification occurs externally to the activated sludge system and thus increases the ratio of denitrification to nitrification in this system compared to a plant without trickling filters.

In the example given the ratio of a reactor volume of 3,900m^3 to a clarifier area of 1,136 m^2 is considered as potentially causing problems. The equivalent values at Selfs Point are 4,900m^3 and 1,064m^2 at a design aerobic fraction of 55-60% (unaerated mass fraction 0.4-0.45) and is thus slightly less likely to result in problems. However, a deep sludge blanket at Selfs Point in winter, which occurred for much of the 1997 winter, could provide an excessive unaerated mass fraction. At fractions above 0 6 it is noted that sludge yield can increase and the sludge can be less stable (Ekama and Marais 1984)

4.8 Chemical Phosphorus Removal

4.8.1 Introduction

Phosphorus limits for wastewater treatment plant effluent are becoming more strict. The question of whether to select biological or chemical phosphorus removal has been raised (Videon 1996; USEPA 1987; Barnett 1994) and there is some evidence to suggest that to achieve very low phosphorus effluents chemical removal may be more cost-effective (Hamilton and Griffiths 1997). The 2mg/L total phosphorus effluent requirement for the Selfs Point Plant is generally accepted as being achievable by biological removal methods at cheaper net present values than chemical removal (for example Michael and Lewis 1997), though chemical backup would be recommended to ensure effluent phosphorus limits are always met (USEPA 1987; WEF and ASCE 1992). Plants in the Danish municipality of Aarhus all had precipitant dosing facilities to ensure compliance with an effluent phosphorus concentration of 1.5mg/L. By improving EBPR performance during the period 1993-1996 however the average molar ratio of iron to phosphorus was reduced from 0.8 to 0.4 (Sørensen et al. 1998). There is some evidence that chemical dosing into the BNR process over time can adversely impact on EBPR (Lötter 1990).

4.8.2 Use of Alum as a Phosphorus Precipitant

At the Selfs Point Plant, anaerobic digestion of the WAS was expected to result in return of some phosphorus to the BNR section of the plant, which could require chemical dosing to capture. While lime and iron salts were considered, due to factors described below and practical operational issues such as occupational health and safety, alum was selected as the most appropriate precipitant chemical.

Alum combines with soluble orthophosphate to form a variety of insoluble precipitates, though it is usually described by (USEPA 1987):

$$Al_2(SO_4)_3 \cdot 14H_2O + 2\,PO_4^{3-} \rightarrow 2AlPO_4\downarrow + 3SO_4^{2-} + 14H_2O$$

Alum also reacts with bicarbonate to form carbonates and analysis of the precipitate when alum dosing can usually be described by $Al_aCa_b(H_2PO_4)_c(OH)_d(HCO_3)_e$ (adapted from Arvin and Henze 1995). The stoichiometric reaction with bicarbonate used for design is considered as (USEPA 1987):

$$Al_2(SO_4)_3 \cdot 14H_2O + 6\,HCO_3^- \rightarrow 2Al(OH)_3\downarrow + 6CO_2 + 14H_2O + 3SO_4^{2-}$$

As indicated by this equation the alum reactions consume alkalinity and this reduction can be a problem for low alkalinity wastewaters (Arvin and Henze et al. 1995).

4.8.3 Location of Precipitant Dosing

There are several potential dosing points for precipitant chemical in a treatment plant. It is however recommended that dosing does not occur in the anaerobic zone or aeration tanks if biological phosphorus removal being performed (WEF and ASCE 1992). The contract specification for the Selfs Point upgrade called for sidestream dosing only (HCC 1995a), however HCC officers accepted that dosing at other points should be considered to maximise the phosphorus precipitated per unit of alum used (Ray Farley, pers. comm., 1998). It is considered by the USEPA (1987: 60) that "full scale experimentation with various points of addition will likely be necessary" to optimise chemical use.

4.8.4 Factors Affecting Precipitant Consumption

The addition of precipitant chemical at Selfs Point was expected to be well below the concentrations used for chemical only phosphorus removal (ANI-Krüger 1996). Two dosing points were initially installed, one after the belt filter press and the other at the inlet to the secondary clarifiers (ANI-Krüger 1996). The quantity of precipitant would be based on that required to precipitate the phosphorus in the digester filtrate sidestream, but the precipitant could be added where the use was most efficient.

To achieve an effluent of 1-2mg/L with simultaneous or post precipitation dosing of the mainstream flow a molar ratio of aluminium to phosphorus of 1:1 is considered typical (Arvin and Henze 1995). When post precipitating, both the pH and alkalinity of the stream being dosed are important, with low alkalinity and pH in the range of 7 to 8.5 providing the lowest soluble phosphorus in the effluent (Arvin and Henze 1995).

In dosing digested sludge filtrate or digested sludge the results are considered as not being realistically predictable due to the variety and concentration of the chemical species present in these streams (USEPA 1987). As both anaerobically digested sludge and its filtrate have relatively high alkalinity concentrations it is anticipated that only a small pH drop would occur when compared to dosing the effluent.

Information provided in Arvin and Henze (1995) indicates that where phosphorus concentrations in wastewater are lower the Al:P ratio has to be higher to achieve the same mass removal rate, though this effect appears to be of most significance at orthophosphate concentrations of less than 5mg/L. More efficient chemical use is expected where the phosphorus concentration is high, as long as the precipitant does not preferentially react with other species (van Loosdrecht, Brandse and de Vries 1998).

Chemical dosing of a flow which returns to the BNR process does have the effect of increasing the fixed suspended solids proportion in the mixed liquor, which thus reduces the biologically active proportion of the mixed liquor solids (Kilpatrick Green 1995). To achieve the same effective sludge age for nitrification the mixed liquor suspended solids has to be increased, which puts additional load on the secondary clarifiers. The dosing is however expected to reduce the SVI more than compensating for the additional solids flux (Kilpatrick Green 1995).

4.9 Disinfection: Impacts of Nutrient Removal

4.9.1 Introduction

The incorporation of nutrient removal into a wastewater treatment plant can impact on disinfection. Chlorine and ultraviolet light (UV) disinfection are considered as these are both installed at the Self's Point Plant.

Chlorine is perhaps presently the most widely used wastewater effluent disinfection process in use at large plants (WEF and ASCE 1992). There is however increasing interest in alternatives in the face of concerns about the toxicity of chlorine species in the environment, in particular the trihalomethanes which can be produced in reactions between chlorine and organic compounds (Bull 1993). In the USA many plants are required to dechlorinate after chlorination to reduce the chlorine residual due to concerns about its toxicity to aquatic organisms (WEF 1996). Recent advances in ultraviolet light disinfection have resulted in this process usually being less expensive than chlorination/dechlorination, but not as cheap as chlorination (Darby et al. 1995). The quality of effluent from nutrient removal plants is usually very high in terms of suspended solids and transmissivity, which tends to improve the competitiveness of UV as it is more sensitive to these parameters than other disinfectants (Darby et al. 1995; USEPA 1986).

UV was selected as the disinfection system for Selfs Point, but chlorine was retained to provide a residual as required. Lund and Ormerod (1995) reported the results of a study into regrowth in potable water distribution systems with three different disinfection systems - chlorination, UV and ozonation. They found that ozonated water produced the greatest biofilm production, with a significantly lower rate in UV disinfected system. No biofilm was formed with chlorinated water maintaining a free residual of 0.04-0.05mg/L. Biofilms in effluent reuse have been shown to increase the potential for elevated numbers of pathogenic bacteria being generated in the distribution system (USEPA 1992) Thus it was anticipated that the addition of a small flow of chlorine would be required to achieve appropriate reuse standards even with high UV dosage.

4.9.2 Disinfection Mechanisms for Chlorine and Ultraviolet Light

There have been a number of theories as to the mechanisms by which chlorine kills microorganisms. These theories, which are supported by observations, include: oxidation, reactions with chlorine, protein precipitation, modification of cell wall permeability, enzyme inhibition, viral protein reactions causing inactivation and hydrolysis and mechanical disruption (Darby et al. 1995; WPCF 1986, Bitton 1994). The disinfecting action of chlorine principally occurs through three chemical species: hypochlorous acid, hypochlorite ion and chloramines, each of which have different disinfection rates (WPCF 1986). The disinfecting power of chlorine is related to the concentration of each of these species, thus any reactions or conditions which adversely alter the proportions of the species or convert the chlorine to inactive forms will reduce the effectiveness of disinfection for a given chlorine dose.

The germicidal effects of ultraviolet light (UV) involve photochemical damage to ribonucleic acid (RNA) and deoxyribonucleic acid (DNA) within the cells of microorganisms. The nucleic acids absorb light in the range 240 to 280 nm and can be damaged, inactivating the cell (Darby et al. 1995). In this wavelength range the pyrimidine thymine can undergo dimerisation, thus distorting the normal helical structure of the parent molecule and preventing the molecule from replicating itself (USEPA 1986). Much of the mercury lamp output is at 253.7 nm, which is near the optimum for disinfection and this lamp has become the preferred UV source (WPCF 1986). The light has to reach the DNA and RNA of an organism to kill or inactivate it, thus absorption or reflectance of UV light which shields the nucleic acids will prevent disinfection. The transmissivity and number and size of suspended particles are important factors in the effectiveness of UV as a disinfectant (Loge et al. 1996)

4.9.3 Impacts of Nutrient Removal on Disinfection

4.9.3.1 Chemical Phosphorus Removal

In achieving desired effluent phosphorus concentrations, chemical dosing can be used as the primary method or it can be used as a supplement to biological removal. The precipitants used most widely are alum and iron salts (USEPA 1987). From a review of the literature alum does not appear to have a major impact on the effectiveness of either chlorination or UV disinfection (USEPA 1986; WPCF 1986).

Iron in the form of ferrous (Fe^{2+}) ions can react with chlorine and converts to ferric (Fe^{3+}) ions, which reduces the chlorine available for disinfection (Darby et al. 1995). This could be a concern if excess precipitant chemical was added and some of the iron left the biological treatment processes in the ferrous form. The chlorine residual could be maintained by increasing the chlorine dosing rate. High iron concentrations are of greater concern with UV disinfection as iron is a strong absorber of UV light at the wavelength emitted by the mercury lamp. Darby et al. (1995) describe three mechanisms by which iron can affect UV disinfection:

1. Soluble iron absorbs UV;
2. Iron can adsorb on to the surfaces of particles and prevent the UV radiation entering the particle; and
3. Iron can precipitate onto quartz lamps and reduce transmission of UV light.

The concentrations of iron in the effluent from dosing with an iron salt precipitant supporting EBPR were not considered to be high enough to present a problem (Fischer and Porter undated).

4.9.3.2 Reduction of Ammonium Nitrogen in Effluent

When there is little or no nitrification in the treatment of municipal wastewater effluent ammonia concentrations are typically 20mg/L or more (USEPA 1993). Chlorine reacts with ammonia to form chloramines, which have a disinfecting power one fiftieth that of chlorine (Bitton 1994). While this factor should result in low ammonia effluents requiring lower chlorine dosages to achieve desired disinfection levels, the converse appears to be the case. It appears that the "free chlorine" is less stable and breaks down more rapidly, reducing the impact it has on microorganisms

(WPCF 1986). In general the disinfection of nitrified effluents by chlorination is considered less reliable than for those with significant ammonia concentrations (WEF and ASCE 1992). One other factor of note, indicated from work by Rebhun, Heller-Grossman and Manka (1997), is that there is a significantly higher rate of disinfection by-product (for example trihalomethanes) formation when ammonia concentrations are low, such as with a nitrified effluent. The final concentrations of the by-products also appears to be related to the dissolved organic carbon content, which tends to be low for BNR effluents (Rebhun, Heller-Grossman and Manka 1997). UV disinfection appears to be relatively unaffected by changes to ammonia concentrations (Darby et al. 1995).

4.9.3.3 Impacts of pH Changes

The effluent from a treatment plant which removes nitrogen has a lower pH than if nitrification had not occurred, due to the consumption of alkalinity during nitrification (USEPA 1993). Denitrification replaces some but not all of the alkalinity destroyed, resulting in a net reduction in alkalinity (Henze 1995). In addition chlorine dosing reduces alkalinity at a rate of 1.4mg/L (as $CaCO_3$) per 1mg/L of chlorine added (WPCF 1986). Lippy (1986) has reported that lower pH requires a greater Ct (concentration time product) to achieve a given kill rate. This increase in either contact time or disinfectant concentration could be due to the equilibrium at lower pH favouring the formation of hypochlorous acid (HOCl) over that of the hypochlorite ion (OCl^-). While the hypochlorous acid is 80 times more effective than hypochlorite ion for inactivating *E. coli* (Bitton 1994), the HOCl can be more readily dissipated by organic reactions, thus reducing its disinfection effect (WPCF 1986). The preferred operating pH is in the range 6.5 to 7.5 to ensure consistent performance (WPCF 1986). Municipal wastewater treatment plant effluent is typically within this range, but the low alkalinity concentration of the Selfs Point influent and the incorporation of nitrogen removal in the process could potentially result in the effluent having a pH of 6.5 or less. The pH of the effluent does not directly affect disinfection rates of UV systems, but can alter chemical equilibria involving metals which may absorb UV light or build up on lamp surfaces and impair effectiveness (Darby et al. 1995).

4.10 Thickening of Prefermenter and Waste Activated Sludges

At the Selfs Point Plant both the prefermenter sludge and waste activated sludge are thickened prior to transfer to the anaerobic digesters. Thickening of both streams is required to achieve the digester design solids concentration of 5.5% (ANI-Krüger 1996). The high solids concentration was selected to ensure sufficient residence time in the digester for good stabilisation without having to construct additional digester volume (ANI-Krüger 1996) The use of this method of increasing digester volumetric efficiency in Germany has been reviewed by Buer, Roos and Pecher (1996). They found that a reduction of up to 35% in digester volume could be made by using this technique with increases in gas production and the degree of stabilisation of the sludge. It was noted that with thickened sludge feed there was greater wear on equipment. The higher solids concentration also has the benefits of reduced heating requirements and volume of supernatant, though thickening to more than 6% can cause mixing problems (WEF 1995).

The rotary drum thickeners used at Selfs Point have a similar thickening performance to gravity drainage decks at lower capital cost, power consumption and space requirements (WEF and ASCE 1992). To achieve a higher solids content in the thickened sludge polymers have to be used to condition the inlet sludge (WEF 1996). While dry powder polymers have typically been used in Australia many plants in Europe use liquid polymers, which are often considered to have better performance, particularly on waste activated sludge (Per Nielsen, pers. comm., 1996).

Waste activated sludge from secondary clarifiers in typical activated sludge plants has a solids concentration of about 0.5 to 1.5% (WEF and ASCE 1992). The thickening of waste BPR sludges should have a small residence time to minimise the potential for phosphorus release that could occur in anaerobic conditions (Barnard, Randall and Sen 1992). The detention time in a rotary drum thickener is short and is expected to result in minimal phosphorus release. While waste activated sludge is generally considered difficult to thicken, with the aid of polymers Buer, Roos and Pecher (1996) found that centrifuges could achieve 6.6% dry solid matter content and sieve drums (or rotary drum thickeners) could produce 6.1% solids content.

4.11 Digestion and Dewatering

4.11.1 Introduction

Biosolids produced by wastewater treatment usually require "stabilisation" prior to use or disposal, unless the biosolids are to be incinerated or are to be direct injected into the ground (WEF 1995). Stabilisation has been defined as "the processing of biosolids to reduce or eliminate the potential for putrefaction and which, as a result, reduces pathogens, vector attraction and offensive odours" (NSW EPA 1997: 106). The criteria for stabilisation usually include one or more of: percentage of volatile solids destruction, reduction in specific oxygen uptake rate, aeration for a minimum time at a given temperature, alkaline treatment and moisture content (Smith and Brobst 1998; NSW EPA 1997; WEF 1995).

The method of stabilisation is particularly important in processes incorporating EBPR, due to the potential return of phosphorus. The WAS from EBPR processes has a significantly higher phosphorus content than non-nutrient removal sludges (Kempton and Cusack 1997). The phosphorus content of a sludge with normal metabolic uptake of phosphorus is in the range 1 to 3% dry weight basis (Bitton 1994), while EBPR sludges with luxury uptake of phosphorus typically have phosphorus concentrations of 3.5-6% by weight (Kempton and Cusack 1997).

There are a variety of methods of stabilisation, including aerobic digestion, anaerobic digestion, alkaline stabilisation and drying (WEF 1995). Examples of the first three are noted in a summary of Australian BNR plants by Hartley (1995).

4.11.2 Aerobic Stabilisation

Perhaps the simplest method of biosolids stabilisation is for extended aeration plants to have a sufficiently long sludge age to meet stability criteria. Tonkovic (1997) found that even with a sludge age of 30 days acceptable stability could not be guaranteed, which is in accordance with USEPA studies. The process design for Selfs Point incorporates an aerobic sludge age of 9 days in winter (ANI-Krüger 1996) and thus the sludge does not meet typical stability criteria. The direct dewatering of waste activated biosolids with such a short sludge age was considered to be inappropriate as the resulting product is transported through the city centre and undergoes further processing by vermiculture (Ray Farley, pers. comm., 1998). The direct dewatering of waste activated sludge without further stabilisation treatment has the obvious advantage that there is little if any release of phosphorus from the sludge into the filtrate (for example see Hartley 1995).

4.11.3 Selection of Anaerobic Digestion at Selfs Point

In the original design for the Selfs Point Plant upgrade aerobic digestion had been proposed (AWT 1993). Aerobic digestion, particularly if aeration is intermittent, can achieve relatively low returns of phosphorus to the main process (Tonkovic 1997). In a survey of plants in the USA, it was found that aerobic digestion was preferred for plant capacities of less than 5 ML/day and anaerobic digestion for plants with greater than 20 ML/day of flow (WEF and ASCE 1992). At 10.4 ML/day Selfs Point is of an intermediate capacity, and if it had been a green field development the capital cost of aerobic digestion would have been lower than for anaerobic digestion. However there were existing anaerobic digesters, which were in good condition and similar to the experience at some plants in Queensland the re-use of existing anaerobic digesters has been assessed as the most cost effective option, despite the potential problems of phosphorus return in the filtrate sidestream (Kempton and Cusack 1997).

4.11.4 Anaerobic Digestion

4.11.4.1 Background

Anaerobic digestion is the "solubilisation and reduction of complex organic substances by microorganisms in the absence of oxygen" (WEF 1995: 50). While it is more capital intensive than aerobic digestion the process does not require aeration, it generates less sludge, it produces usable energy in the form of biogas and it can degrade some of the more resistant organics such as lignin (WEF 1995; Bitton 1994). The chemical reactions can be described by (adapted from Bitton (1994: 230)):

$$\text{Organic matter} \rightarrow CH_4 + CO_2 + H_2 + NH_3 + H_2S + \text{residue}$$

Bitton (1994) describes four categories of bacteria: hydrolytic, fermentative acidogenic, acetogenic and methanogenic, which operate in a synergistic relationship, with the "waste" products from each group providing substrate for other groups. Usually the most crucial group for achieving anaerobic digestion of wastewater sludges is that of the methanogenic bacteria. These are the slowest growing and most sensitive to changes in environment. The maximum specific growth rate of methanogens is about 0.04 hr^{-1} compared to about 1 hr^{-1} for the acetogens (Hammer 1986).

Anaerobic digestion occurs over a wide range of temperatures, however the temperature ranges of practical importance are the mesophilic (30-40°C) and thermophilic (50-65°C), as reactor volumes at lower temperatures become excessive (WEF 1995). By far the majority of full scale anaerobic digestion is performed in the mesophilic range (Chynoweth and Pullammanappallil 1996) and this is the case for the Selfs Point digesters In the mesophilic range a sludge residence time of 20 days is generally accepted as providing adequate digestion to meet stability criteria (WEF 1995). The process design for Selfs Point is for a 35 day sludge residence time, with pre-thickening to reduce the hydraulic load on the digesters (ANI-Krüger 1996).

While a variety of other factors can affect digester performance including pH, chemical composition of the wastewater and presence of toxicants, the following discusses some of the factors relevant to treating biological phosphorus removal sludges.

4.11.4.2 Anaerobic Digestion and Biological Phosphorus Removal

In the EBPR process the PAOs release phosphorus in anaerobic conditions. When the waste activated sludge enters the anaerobic conditions of the digester there is a release of phosphates which has been variously reported as being up to 60% (Kempton and Cusack 1997), up to 75% (Barnard, Randall and Sen 1992) and up to 80% (Mavinic et al. 1996) of that contained in the waste sludge. These values would appear to be equivalent to the proportion of luxury uptake of phosphorus compared to normal metabolic needs in EBPR sludges.

Operating results from full scale plants indicate that a significant proportion of the released phosphorus is precipitated out as inorganic phosphates, such as struvite ($MgNH_4PO_4$), brushite ($CaHPO_4 \cdot 2H_2O$) and vivianite ($Fe_2(PO)_4 \cdot 8H_2O$) (Barnard, Randall and Sen 1992) and also being fixed by aluminium (Jardin and Pöpel 1996).

Work has shown that potassium, calcium and magnesium are important in the accumulation of polyphosphate (Lockwood, Lindrea and Seviour 1990) and polyphosphates are assumed in Activated Sludge Model No. 2 to have the composition $(K_{0.33}Mg_{0.33}PO_3)_n$ (Gujer et al. 1995). The release of these cations in conditions prevalent in an anaerobic digester appears to lead to the formation of struvite from the magnesium released from the PAOs (Jardin and Pöpel 1996).

A study of the York River WWTP anaerobic digester prior to and during biological phosphorus removal, indicated that while the total phosphorus in the digester increased by 650mg/L the soluble phosphorus only increased by 250mg/L. Batch studies with the sludge indicated that significant precipitation of struvite and brushite was occurring and it was inferred that the remainder of precipitated phosphorus was in the form of vivianite. Only 27% of the total phosphorus in the digester was in soluble form (Barnard, Randall and Sen 1992). This compares favourably to the 20%

return of phosphorus from dewatering of anaerobically digested sludge typically experienced in Denmark, where the potable water typically has relatively high concentrations of calcium and magnesium, and digester filtrate does not cause major problems with phosphorus return (ANI-Krüger 1998; Sørensen et al. 1998). A plant at Pontiac, Michigan was noted as having little phosphorus return from anaerobic digesters treating a BPR sludge, with precipitate thought to be struvite (USEPA 1987). By contrast, at the Ballarat South Plant in Victoria, it has been estimated that the return of phosphorus to the process is close to 50%, with the orthophosphate concentration in the digesters typically being 320mg/L (Price 1990). In comparing an EBPR plant anaerobic digester sludge with that of a non-EBPR plant the soluble fraction of magnesium was markedly lower, whereas for the other cations measured the soluble fractions were similar (Carliell and Wheatley 1997). The proportion of phosphorus precipitated would thus appear to be closely related to the concentration of specific cations in the influent wastewater.

The potable water in the Hobart City Council area has a low total dissolved solids concentration and a very low hardness (Ed Kleywegt, pers. comm., 1998) and limited data of the influent wastewater indicates calcium concentrations of less than 20mg/L (Selfs Point plant records, 1998). Based on this information it would appear that the proportion of phosphorus remaining soluble in the digesters at Selfs Point is likely to be of the same order as at the Ballarat South plant, rather than the other plants described. If the proportion of phosphorus returning to the process is too high the biological system may not have sufficient capacity to take up the load (Kempton and Cusack 1997).

Elsewhere in Australia there are plants where digested sludge filtrate has been mixed with lime slurry or pickle liquor to precipitate the phosphorus from the filtrate and then undergone settling to remove the phosphorus-rich sludge for further dewatering (Hartley 1995; AWWA 1997; Kempton and Cusack 1997). At Nambour STP the digester supernatant and filtrate undergo aeration to strip CO_2 and raise the pH prior to mixing with lime. The CO_2 stripper reduces the quantity of lime required to achieve the desired pH range (Kempton and Cusack 1997).

4.11.4.3 Impact of Precipitant Chemicals on Anaerobic Digestion

If precipitant chemicals are added to a process to achieve phosphorus removal, depending on the addition point they may have to pass through the digestion system.

In a summary of work and operating experiences carried out by the USEPA (1987) it appears that anaerobic digester performance can be adversely impacted by the addition of the precipitant chemicals, alum and ferric chloride The concentrations of precipitant chemicals were relatively high as the summary was dealing with chemical only phosphorus removal plants. Gossett et al. (1978) concluded from laboratory studies that ferric chloride doses of 150mg/L and alum doses of 200mg/L to wastewater were required to cause noticeable drop in digester performance. A recent study by Ghyoot and Verstraete (1997) did not find any drop in digester performance with the introduction of chemically assisted primary tank operation with $FeCl_3$ being added at a rate of about 100mg/L The study also noted that there did not appear to be any release of the chemically bound phosphorus during anaerobic digestion, while the USEPA (1987) summary found that there may be some release from ferric ions reducing to ferrous ions

4.11.4.4 Ammonia Return From Dewatering Digested Sludge

Anaerobic digestion of solids streams in a wastewater treatment results in a significant return of ammonia to the liquid processes (WEF 1995) As the waste activated sludge (WAS) from BNR processes has a similar nitrogen content to non-BNR processes for the same sludge mass flowrate the same nitrogen mass load would be transferred to the digestion system In a nitrogen removal plant WAS has undergone a degree of aerobic stabilisation (due to the sludge age required to achieve nitrification) less of these solids are solubilised in the digester and thus this sludge typically generates less ammonia in the digestion system than primary solids (USEPA 1993). Thus upgrading a wastewater treatment plant to BNR operation may result in a only small change to the ammonia return to the process. For Selfs Point, it was estimated that the increase could be up to 20% (Hans Regnersgard, pers. comm., 1996). At York River WWTP the ammonia concentration in the primary anaerobic digester increased from 316mg/L to 390mg/L following the introduction of BPR (Barnard, Randall and Sen 1992). Even though the increase may be small, as the nutrient removal process has to nitrify ammonia, this load is usually significant and must be taken into account in the design and operation of the plant (USEPA 1993).

It is noted by Barnard, Randall and Sen (1992) and Parker et al. (1989) that ammonia return from digested solids dewatering to the liquid process is an important consideration and should be performed over an extended period, preferably during low load conditions. If this is not undertaken there may be ammonia breakthrough to the plant effluent, when the load exceeds the maximum nitrification rate of the process or there is inadequate alkalinity (USEPA 1993).

Where anaerobic digester supernatant is returned to the process, the nitrification rate can be reduced. Gujer (1977) noted that if the supernatant recycle increased the ammonia nitrogen concentration by 5mg/L in the mainstream process, the growth rate of *Nitrosomonas* could be reduced by 20%. The study assumed the inhibition was due to one or more by-products of anaerobic digestion. Work by Æsoy, Ødegaard and Bentzen (1998) on septic wastewaters entering a treatment plant found that even small concentrations of sulphide (about 0.5mg/L as sulphur) could inhibit nitrification. Sulphide concentration may be a significant cause of nitrification inhibition by digested sludge filtrate, although other components may also be a factor (Bitton 1994). Digester supernatant has been successfully nitrified in a sequencing batch reactor pilot plant in Sweden (Mossakowska, Reinius and Hultman 1997) and the work was undertaken due to concerns that the high ammonium concentration in anaerobic digester filtrate may create inhibitory products such as nitrous acid and hydroxylamine and that severe foaming had on occasions been observed in the full scale plant.

4.11.4.5 Impact of Increased Solids Content on Digester Performance

At Selfs Point one of the changes in operations during the upgrade which is likely to have had a major impact on the nitrogen equilibria of the digestion system, is the increase in digester solids concentration as a result of pre-thickening the sludge feeds (ANI-Krüger 1996) Prior to the plant upgrade ammonium concentration in the primary digester was in the range 300 to 500mg/L NH_4^+-N at a total solids content of 2 to 2.5% (Selfs Point plant records).

For anaerobic digestion ammonium concentrations of up to 700mg/L appear to be beneficial, but ammonium toxicity can be a concern if the digester is operated at high solids content and the ammonium-nitrogen concentration is 3000mg/L or greater (Chynoweth and Pullammanappallil 1996). Higher ammonia concentrations can be tolerated at longer solids retention times (Bhattacharya and Parkin 1989). A detailed study by Lay, Li and Noike (1998) indicates that the methane production rate of an anaerobic digester begins to drop as ammonium nitrogen concentration increases above the range 1000-3000mg/L, with the concentration at which the drop commences being pH dependent. With the design solids content of the digesters having increased by about 120%, if the ammonium nitrogen concentration increases in proportion to the solids loading, then the ammonium concentration would still be at a level well below that where ammonium toxicity may be a problem, though there may be a slight drop in proportion of volatile solids destruction.

4.12 Vermiculture and Biosolids Quality

4.12.1 Background

The biosolids generated at the Self's Point Plant are transported to the Hobart City Council Worm Farm, where the material is treated by vermiculture to produce a high grade soil conditioner. This method of downstream processing has placed a number of limitations on the processes used at Selfs Point, particularly those used in nutrient removal. The main issues are:

1. The requirement for biosolids to have undergone a "Process to significantly reduce pathogens (PSRP)" (WEF 1995), prior to reaching the Worm Farm;
2. The biosolids must be relatively easy to handle with mobile machinery;
3. The sludge should be relatively odourless and have low vector attraction; and
4. The material must have chemical properties in which earthworms can thrive, including an appropriate pH and low ammonia concentration.

In Australia many flowsheets of BNR plants the WAS is either direct dewatered or undergoes aerobic digestion with lime dosing of the filtrate sidestream to capture any phosphorus which has been released during the digestion process (AWWA 1997).

The first of these two options was not considered acceptable for Selfs Point due to the inadequate pathogen removal levels, greater volumes of difficult to handle sludge and insufficient stabilisation at winter temperatures. The relatively low degree of stabilisation was expected to result in an odorous sludge with significant vector attraction potential (Kanak 1994). The second option was considered as having greater capital and operating expenditure than using the existing anaerobic digesters, which were still in good condition (Ray Farley, pers. comm., 1998). Use of lime was not preferred due to potential dust emissions from materials handling.

Anaerobic digestion does remove a significant proportion of pathogens in the biosolids, but may not always be sufficient to comply with the numeric limit of 2,000,000 cfu/g of total solids for faecal coliforms of the USEPA 40 CFR 503, class

B pathogen criterion (Stukenberg et al. 1994). However, further pathogen reduction occurs during the vermiculture processing through the antibiotic action of worms and post composting (Andrew Baker, pers. comm., 1998).

4.12.2 Organic Substrate Factors Affecting Earthworm Productivity

While earthworms are ubiquitous where soil contains adequate organic matter and moisture content, there are a variety of factors which can affect growth rate, reproduction rate and whether they will feed on a particular material. The use of earthworms to actively process organic wastes has been a relatively recent phenomenon, with growing interest in the 1980s (Murphy 1993). Commercial scale treatment of biosolids has only recently occurred (for example in Poland see Anon 1997a). Thus there has only been a relatively short history of research into the factors critical to the vermiculture processing of such materials.

4.12.3 Impact of Substrate pH

From the research to date it would appear that pH of the organic material is one of the critical parameters for earthworm survival. In a summary of work carried out on temperate climate species it was found that the preferred range of pH was 5.0-7 4 and that very alkaline wastes such as pulverised fly ash can be toxic to soil flora and thus destroy the earthworm population (Curry 1998). It is noted by Edwards (1998) that although the species often used for processing organic waste *Eisenia fetida* could tolerate a pH range of 5 to 9 the worms moved towards the more acidic material in soil with a pH gradient. Earthworms may be able to tolerate some variations in pH due to the buffering capacity of their mucus secretions (Edwards and Shapitalo 1998), which could explain the relatively wide range of tolerance.

Fungi have been shown to be an important food source for earthworms (Doubé and Brown 1998) and many species of fungi are favoured by slightly acidic conditions rather than alkaline (Bitton 1994). This relationship could contribute to earthworm's preference for slightly acidic conditions.

4.12.4 Impact of Ammonia Content

It has been noted that earthworms are very sensitive to ammonia and die quickly if ammonia concentrations are greater than 0.5mg/g, such as occur in fresh poultry manure (Edwards 1998). Digested sludge has relatively high concentrations of soluble ammonium ions (WEF 1995). As pH is raised the equilibrium shifts towards undissociated ammonia, such that at pH 10 approximately 80% of total ammonia will be present as NH_3 (WEF and ASCE 1992). Thus lime amendment would be expected to increase ammonia concentrations in the biosolids and if this ammonia is not released to the atmosphere the biosolids would not be suitable for vermiculture treatment. The loss of this nitrogen also reduces the fertiliser value of the biosolids.

4.12.5 Heavy Metals and Phosphorus Precipitant Chemicals

Earthworms appear to tolerate quite high levels of heavy metals, though metals of inorganic form appear to pose greater risks than organically bound metals (Curry 1998). Eijsackers (1998) has summarised research with shows that combinations of metal salts (including $FeCl_3$) can impact and reduce earthworm reproduction rates, but also noted that earthworms actively avoid higher concentrations of heavy metals. From the work undertaken to date it appears that the heavy metal content and form found in biosolids from domestic sourced wastewater results in it being acceptable for vermiculture (Edwards 1998).

The iron concentrations in the biosolids from both of Hobart City Council's wastewater treatment plants is relatively high at 6000 to 15,000mg/kg dry solid basis (HCC unpublished data; Kinhill 1993). The high iron concentration can reduce the availability of other nutrients, such as copper and manganese and thus lower the agricultural value of the end product (Handreck and Black 1994), though trials in the USA have not shown any significant decrease in growth compared to non-chemical sludges (USEPA 1987). Use of an iron based phosphorus precipitant chemical has been shown to increase the iron concentration in digested sludge significantly, with one study which analysed digested sludge from a plant using iron salts to precipitate phosphorus finding an average total iron content of 44,000mg/kg (Kyle and McClintock 1995). This suggests that using iron salts to precipitate phosphorus could impact on the value of the final vermicast product.

It is noted by Kyle and McClintock (1995) that the phosphorus in both iron and alum precipitated sludges had reduced solubility compared to biological phosphorus removal and non-nutrient removal plant digested sludges and that this "may offer a positive environmental effect because the fixed P would be available for later growing seasons, and eroded soils may be less likely to release P when in the low pH, reducing conditions of benthic environments" (Kyle and McClintock 1995: 288). Thus while iron or alum precipitated phosphorus is less readily bioavailable, it is also less leachable and would act more as a slow release fertiliser than the phosphorus in non-chemical sludges.

4.12.6 Vermiculture Summary

The selection by Hobart City Council of vermicomposting as its downstream value adding process for biosolids for Selfs Point limited the options for chemical phosphorus removal. Due to the substrate requirements of the earthworms and the need to maximise the value of the final product both lime and iron salts were considered inappropriate, leaving alum as the only viable option of the readily available chemicals. Some inorganic chemical binding of phosphorus may improve the environmental impact of the vermicompost by reducing the leachability.

4.13 Chapter Summary

The Selfs Point Plant has an atypical flowsheet for a BNR plant. Many of the processes interact through sidestreams or constrain operational options for other processes. The incorporation of the trickling filters increases the degree of flexibility but also the complexity of the plant. The nitrification performance of the trickling filters is crucial in achieving the required nitrogen removal rates. Prefermentation is required to support EBPR at Selfs Point and optimising the performance of this system will improve phosphorus removal. Anaerobic digestion creates a nutrient rich sidestream which can have a major impact on plant nutrient mass balances. A review of theory, laboratory research and observations of plant performance elsewhere has provided the basis for assessing the BNR performance of the Selfs Point Plant.

CHAPTER 5: PROCESS DESCRIPTION AND METHODS

5.1 General Design and Catchment Description

5.1.1 Introduction

Hobart City Council is responsible for three separate sewage catchments, which, in the main, coincide with natural drainage basins (see Figure 5.1). The northern catchment, including the suburbs of New Town, Mt. Stuart and Lenah Valley generates mainly domestic quality wastewater, but stronger waste is discharged from a milk processing plant. The sewage from this catchment flows by gravity to the Selfs Point Plant. Wastewater from the central area, including the Hobart city centre, is treated by high rate trickling filters at the Macquarie Point Plant The southern catchment, including the suburbs of Sandy Bay and Mount Nelson, has an inflow which is virtually all generated by either residential or light commercial sources (Morgan and Farley, 1998).

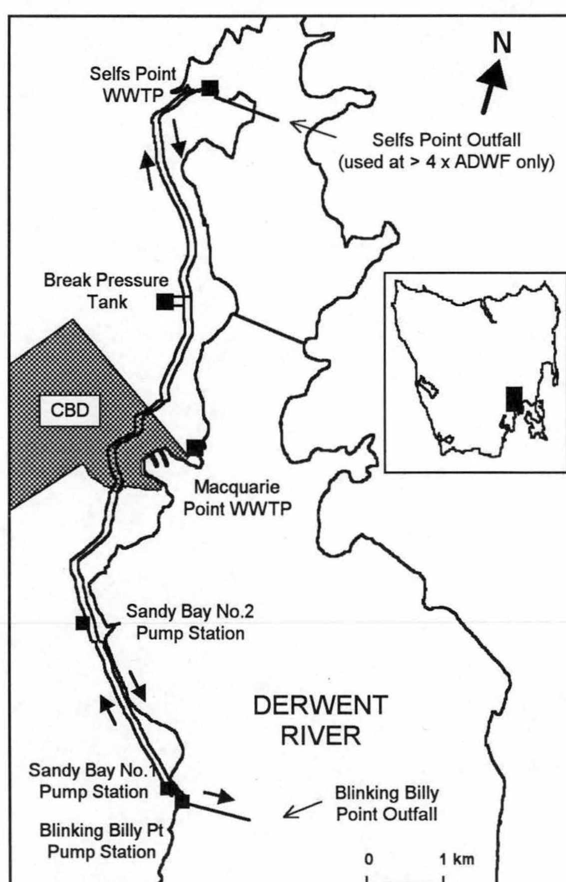

Figure 5.1: Map with Location of Selfs Point Plant and Pipelines to Sandy Bay

The sewage from the southern or Sandy Bay catchment is transferred through a rising main to the Self's Point Plant. There are two major pump stations, the southern station pumping approximately half the flow to a second station, which transfers the total Sandy Bay flow approximately 8 kilometres to the Self's Point Plant. For the last 3 kilometres flow is by gravity from a vent tank at the high point in the pipeline. The Sandy Bay sewage is then combined with the New Town catchment inflow before being treated to nutrient removal standard at the upgraded plant and pumped through a return effluent pipeline for discharge through the Blinking Billy Point outfall. The same route is followed by the raw sewage and return effluent pipelines, as indicated in Figure 5.1. The pipelines are each 10.5 kilometres long (Morgan and Farley 1998).

5.1.2 Design Criteria

The major process design parameters for the Self's Point Plant upgrade were determined by the requirements of the new environmental licence for the Selfs Point Plant. The concentrations given in Table 5.1 indicate the design influent composition selected by ANI-Krüger , the previous Self's Point licence conditions, the new licence conditions and the effluent concentrations specified in the contract documents.

Table 5.1: Influent and effluent parameters for Selfs Point Plant

Parameter	Units	Design Influent Concentration	Previous Licence	New Licence	Design Effluent
BOD_5	mg/L	265	40	15	15
Suspended Solids	mg/L	200	60	20	20
Ammonia as Nitrogen	mg/L	39	no limit	2	2
Total Nitrogen	mg/L	41	no limit	10	10
Total Phosphorus	mg/L	9	no limit	3	2
Oil and grease	mg/L	25*	no limit	5	5
Faecal coliforms	cfu/100mL	10^6-10^8	100	750	10

* This value was based on very limited data

The licence conditions are set at 'never to exceed' concentrations (DELM 1995). However in preparing the specification for the design and construct contract these conditions were considered as being unrealistic for designers to meet and the never to exceed limits were replaced by a performance evaluation requiring 95% compliance. Both phosphorus and faecal coliforms design standards were reduced below licence limits to enhance the range of options for effluent reuse (Morgan and Farley 1998).

The specified minimum design temperature was 10°C in the aeration tank, with a maximum value of 22°C. No particular process was specified by Hobart City Council in the contract documents, only performance criteria and minimum standards for equipment (HCC 1995a). As the site is a former landfill with highly corrosive groundwater and Council required efficient use of space to maximise area for potential future expansion, the footprint and layout of the major process units were significant factors in the design. The biosolids quality was required to meet Class B stability criteria of the USEPA *Part 503 Regulations* (HCC 1995a).

5.1.3 Combined New Town and Sandy Bay Catchment Characteristics

The Sandy Bay and New Town catchments are predominantly residential with some light commercial areas and few industries discharging to sewer. A milk processor in the New Town catchment is the only significant industrial effluent contribution, providing 2-3% of the total flow to the plant and 10-15% of the organic load. The New Town catchment is a gravity collection system in which the average grade is relatively steep (except for close to the plant) resulting in a mean sewage residence time of about 2-3 hours. The Sandy Bay catchment is slightly larger in size with steeper topography. The transfer to Self's Point by pumping increases the average sewer residence time before treatment to 4-10 hours for the Sandy Bay flow (Morgan and Farley 1998).

The specification for the plant upgrade required a design for an average dry weather flow of 10.4 ML/day, while the dry weather flow for the combined catchment is typically in the range 9.0 to 9.5 ML/day. The peak diurnal dry weather flowrate for the combined catchment usually falls in the range 180-240 litres/second, while the minimum daily flow is about 40 litres/second. At the time of the BNR process startup the BOD_5 concentrations were measured as about five times higher at peak flow (400-500 mg/L) compared to overnight levels (80-100mg/L) and this resulted in an influent organic load fluctuation of 20:1 over a typical day during the early stages of the commissioning (Morgan, Farley and Pearson 1997). The milk processing factory has recently installed wastewater storage, reducing the diurnal BOD variations. The impact of this change is discussed in Chapter 6.

5.2 Description of Upgraded Treatment Plant

5.2.1 Equipment and Process Design Summary

The previous Self's Point Plant flowsheet incorporated inlet works, primary sedimentation, trickling filters, secondary sedimentation tanks, chlorine disinfection, anaerobic digestion and drying beds The tanks were all in good structural condition and the upgrade incorporated all of the existing tanks and the trickling filters. Basic data for the main processes in the upgraded Plant is provided in Table 5.2 (ANI-Krüger, undated).

Table 5.2 Self's Point Plant Technical Data Summary

Structures	Dimension	Equipment	Description
Mechanical screen	480 L/s	Screening	Step, 10 mm spacing
Raw influent pumps	700L/s	Influent pumping	3 variable speed pumps
Grit Chamber	$120m^3$	Grit chamber	Aerated
Primary settling tanks	2 x $340m^3$	Primary tanks	Rectangular
Primary effluent pumps	240 L/s	Primary effluent	3 variable speed pumps
Stormflow tanks	$650m^3$	Stormflow pumps	2 x 30L/s, fixed speed
Trickling filters	2 x $1,285m^3$	Trickling filters	Variable speed, 4 arm
Anaerobic tanks	$550m^3$	Anaerobic tanks	4, vertical shaft mixers
Aeration tanks	3 x $1,450m^3$	Aeration tanks	6 rotors, 3 mixers
Clarifiers	2 x 26m dia	Clarifiers	Scraper bridges
UV disinfection	300L/s	UV disinfection	32 lamps, self cleaning
Final effluent pumps	300L/s	Effluent pumps	4 dry mounted
Prefermenter	$130m^3$	Prefermenter	Vertical shaft mixer
Prefermenter thickener	$20m^3$/hr	Polymer systems	Two dry, one liquid
Excess sludge thickener	$30m^3$/hr	Process water	Recycled effluent
Anaerobic digesters	2 x $700m^3$	Digesters	Gas mixed, in parallel
Gas fired heating	175kW	Cogeneration	2 x 15 kW gas engines
Belt filter press	$30m^3$/hr	Skip bin	$8m^3$ capacity

A summary of the basic process design parameters for the upgraded Self's Point Plant is provided in Table 5.3

A graphic showing the process for the liquid streams of the plant is illustrated in Figure 5.2 (from Morgan and Farley 1998), while an overall process flow sheet for the upgraded plant is given in Figure 5 3.

Table 5.3 Self's Point Plant Process Design Parameters at August 1998

Design Parameter	Value or Description
Design ADWF	10.4ML/day or 44,000 EP
Design PWWF	480L/sec or 4 times ADWF
Primary Settling Tank	$340m^3$ with HRT of 0.8hr at ADWF, SRT of 2 days
Trickling filters	Rock media, $2570m^3$ at 1.8m deep
Anaerobic zone	$550m^3$ with two selector tanks and two main tanks
Aeration tanks	Total volume $4350m^3$, max. aeration rate $265kgO_2/hr$
Secondary clarifiers	4.5m side wall, weir rate $0.40m^3/m^2/hr$ at ADWF
Anaerobic digestion	$1400m^3$, to be operated at 38°C and 5 5% solids
Maximum effluent flow	300L/sec (2.5 ADWF) to Blinking Billy Point outfall
RAS flow rate	70-80% of influent flowrate
Split to trickling filters	50% at >ADWF, 10% at min flow, 50L/sec recycle
Design MLSS	3000-3500mg/L @ >15°C, 4500mg/L @ 12°C
Minimum temperature	12°C
Aerobic sludge age	9 days at minimum design temperature
Wasting rate of WAS	1200 to 1600 kg dry weight per day
Design SVI	DSVI of 150mL/g (max.)
Design MLVSS:MLSS	0.67 (Actual 0.75-0.80)
Design F/M ratio	0.075kgBOD/(kgMLSS)/d
Typical mass fractions	0.10 anaerobic/0.3-0.45 anoxic/0 45-0 6 aerobic
Nitrification rate	$80gNH_4$-N/(kgNMLVSS x h)*
Denitrification rate	1 $8gNO_3$-N/(kgMLVSS x h)

* - NMLVSS denotes the nitrifying proportion of the mixed liquor solids

5.2.2 Inlet Works

After flows from the two catchments combine, raw sewage passes through a 6mm spacing self cleaning bar screen prior to the inlet wet well The screenings are washed and pressed to a design solids content of greater than 35% before being bagged for disposal (HCC 1995a). The inlet pumps are driven by variable speed drives and are controlled by an algorithm using both flow and wet well level to eliminate rapid fluctuations in the influent flow to the plant.

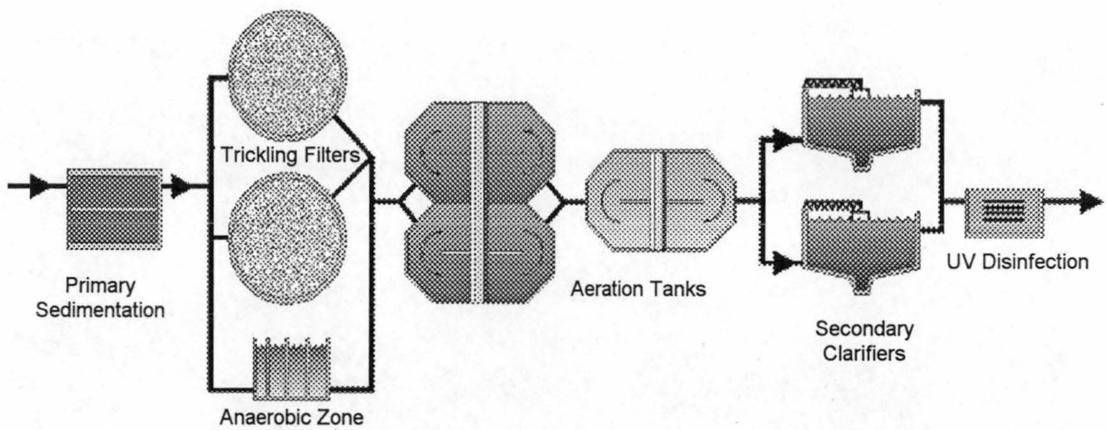

Figure 5.2 Selfs Point Liquid Process Flow Diagram

The pumps lift the screened sewage to an aerated grit chamber. Grit is removed from the chamber by air lift pumps and separated in a grit classifier. While the aerated grit removal does increase dissolved oxygen (DO) concentration slightly and strip some volatile organic compounds, the slight negative impact on phosphorus removal is considered to be outweighed by the reliability and effectiveness of the system. Both grit and screenings are removed for disposal to landfill (Morgan and Farley 1998).

Subsequent to grit removal raw sewage flows to the existing rectangular primary sedimentation tanks, only one of which is being used as discussed in Chapter 6. The tank has a sidewall depth of 2.75 metres, with a volume of $340m^3$. The primary tank is operated in "activated primary" mode with a sludge residence time of about 2 days. A submersible pump is used to transfer sludge from one of the collection hoppers to the tank inlet to assist elutriation of soluble COD generated by hydrolysis in the tank. At the discharge of this tank, actuated adjustable weirs split the flow between the trickling filters, the BNR section of the plant and, at times of high influent flow, the storm flow retention tanks (Morgan and Farley 1998).

The retention tanks, are the old secondary sedimentation tanks, and have a combined volume of $650m^3$. Flows in excess of 300 L/sec are diverted to these tanks for later reintroduction to the process, to prevent sludge washout from the BNR section of the plant. During extended periods of high influent flow, once the tanks are full the overflow leaves the plant having undergone settling treatment in two sets of tanks. These tanks are also capable of storing effluent or digested sludge in emergency situations (Morgan and Farley 1998).

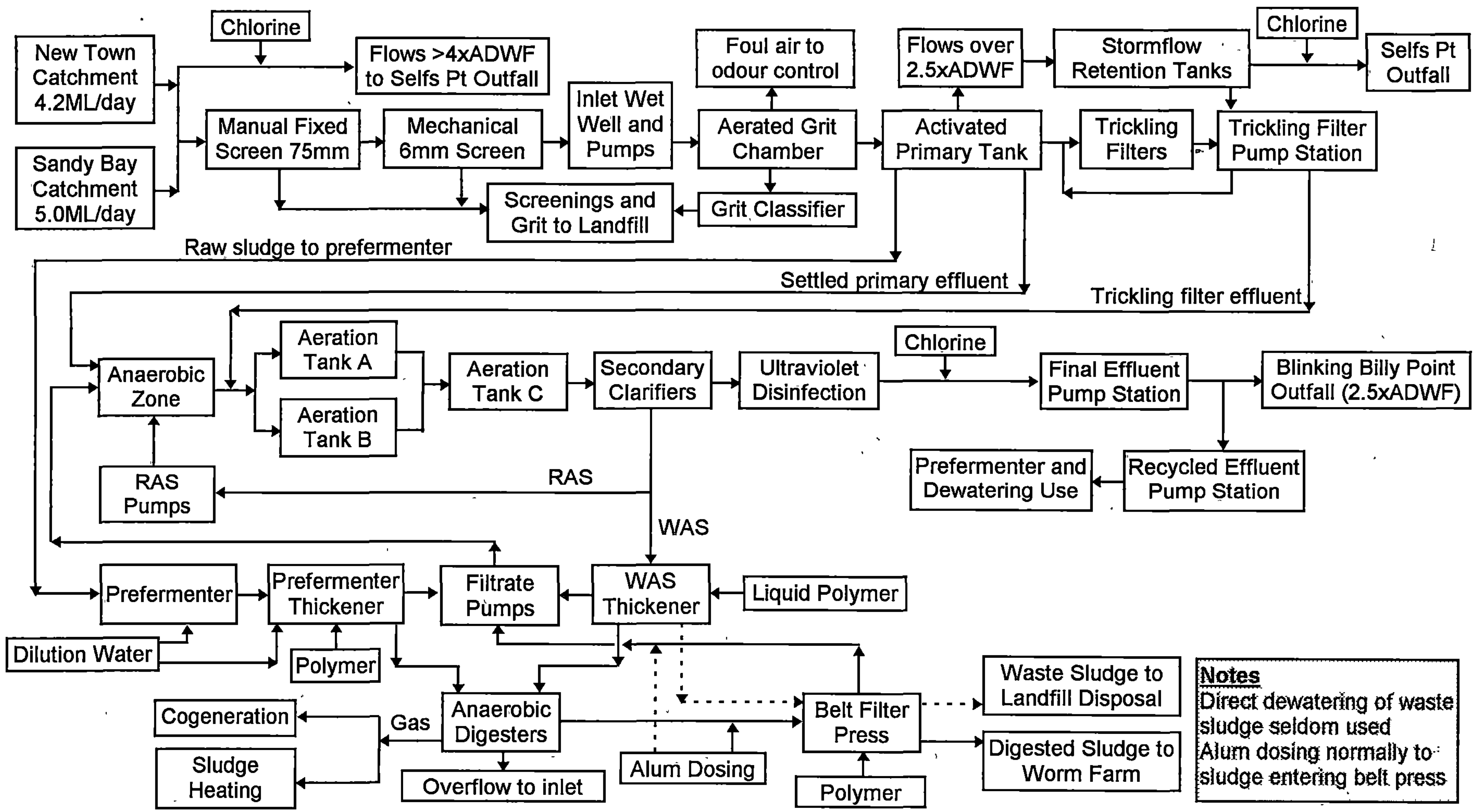

Figure 5.3 Process Flowsheet for Upgraded Self's Point Plant in August 1998

5.2.3 Trickling Filters

The two existing trickling filters have been retained with only minor refurbishment. The filters are each 30 metres in diameter, approximately 2 metres in depth with rock media and have natural ventilation only. The filters are operated in parallel, fed from a splitting box at the primary sedimentation tank weir and the arms have variable speed drives. The effluent from the filters is pumped, without settling, direct to the inlet of the BNR aeration reactors. About 50 litres/second can be recirculated to the trickling filter feed to maintain hydraulic load during low influent flow and for maintenance flushing of the media. Trickling filter effluent bypasses the anaerobic zone of the BNR system as the high nitrate concentration of this stream would reduce the zone's effectiveness. The filters provide good BOD removal and a significant degree of nitrification, thus significantly reducing the aeration energy required in the BNR process (Morgan, Farley and Pearson 1998).

5.2.4 Biological Nutrient Removal Process and Clarifiers

The BNR system incorporates the phased isolation ditch Bio-Denipho™ process developed by Krüger in Denmark. The system at Selfs Point includes an anaerobic zone which is followed by three oxidation ditch aeration reactors and two circular clarifiers.

Primary settled effluent, the prefermenter thickener filtrate, return sludge from the clarifiers and filtrate from the dewatering systems are combined at the inlet of the anaerobic zone. The anaerobic zone has two small mixing tanks followed by two larger tanks, which provide about a total of about one hour residence time at average dry weather flow. There is no supplementary external carbon addition to the anaerobic zone (Morgan and Farley 1998).

Trickling filter effluent is added prior to the flow entering one of the first two aeration reactors. The flow is switched between these tanks on a timed cycle. The tank being fed is operated in the denitrification cycle without aeration, while the other tank is in nitrification mode and is being aerated. The third aeration reactor operates in series with whichever inlet reactor is being fed and is switched between nitrification and denitrification as controlled by settings made on the supervisory control and data acquisition (SCADA) system (Morgan and Farley 1998). The tank arrangement is a relatively novel configuration for the BioDenipho™ process, but theoretically has the potential to provide greater efficiency than a configuration of three tanks in parallel (Dines Thornberg, pers. comm., 1997).

Aeration is achieved by surface rotors, which also provide good mixing. While the rotors are generally considered less energy efficient than fine bubble diffused air systems, they are far simpler to access for maintenance and provide reliable performance. One rotor in each reactor is dual speed to increase the turndown ratio of the aeration system. Propeller mixers are installed in each of the tanks to prevent settling and ensure good mixing when the rotors are off. An actuated weir at the outlet to the third tank is used to control the level, which permits some control over the aeration efficiency of the rotors. Foul air collected from the more odorous processes is bubbled through a diffuser in the third aeration tank to remove odours (Morgan and Farley 1998).

Two clarifiers operate in parallel to settle the effluent from the third aeration reactor They are each 26 metres in diameter with a sidewall depth of 4.5 metres, are centre fed, have a peripheral effluent weir and have surface scum collection. The return sludge is controlled by two variable speed drive submersible centrifugal pumps, one for each clarifier. The flowrate is usually controlled to a setpoint which is a selected ratio of the influent flow averaged over fifteen minutes, with options for control including on/off times and minimum limits to pump speeds.

5.2.5 Disinfection

The effluent from the clarifiers undergoes ultraviolet light (UV) disinfection prior to being pumped to the Blinking Billy Point outfall. The UV system has been designed to achieve <10cfu/100mL concentration of faecal coliforms at two times average dry weather flow (240L/s). The UV unit is a Trojan UV4000 system with 32 medium pressure, high intensity lamps, which have variable intensity control. The system's programmable logic controller (PLC) controls the UV power output (using an algorithm incorporating transmissivity and flowrate measurements), cleaning frequency and cycling of lamps (Morgan and Farley 1998).

The existing chlorine disinfection system has been retained with a reduced storage volume of chlorine to dose any stormflows not passing through the UV system, as a backup to the UV system, to reduce biofilm growth in the effluent wet well and pipeline and to provide a chlorine residual, if required, to meet effluent reuse standards (Morgan and Farley 1998).

5.2.6 Sludge Thickening and Prefermentation

The sludge collected in the primary sedimentation tanks is transferred from the tank hoppers under timed sequence control to the prefermenter. The prefermenter provides conditions to allow partial hydrolysis of the sludge generating a significant increase in concentration of readily biodegradable chemical oxygen demand (RBCOD). The prefermenter has a volume of $130m^3$ and was originally designed as a complete mix reactor with a 1.5 day hydraulic and solids residence times. The addition of dilution water and reduced flow from the primary sedimentation tank have increased process options and the system can be operated with a maximum solids residence time (SRT) of about 5 days and hydraulic residence time (HRT) of about 2 days. From the prefermenter the sludge passes through a rotary drum thickener. The thickened sludge of about 8-9% solids content is transferred to the anaerobic digesters. The thickener can be operated automatically at times to suit process requirements for addition of filtrate, though preference is for start up during operator attendance. The filtrate from the thickening process, rich in RBCOD, is transferred to the inlet of the anaerobic zone (Morgan and Farley 1998).

Activated sludge is wasted from the return sludge stream through a rotary drum thickener of the same size as the prefermenter thickener. The solids content is increased from 0.8-1.2 % to 3-4%. To achieve a greater solids concentration in the thickened sludge a liquid polymer dosing system has been installed in addition to the solid polymer make up units. Once thickened, the sludge is pumped to the anaerobic digesters for further stabilisation, while the filtrate is returned to the anaerobic zone of the BNR plant (Morgan and Farley 1998).

5.2.7 Digestion and Dewatering

The two thickened sludge streams are co-digested in the two existing anaerobic digesters. The gas mixed digesters, each $700m^3$ in volume, were refurbished as part of the plant upgrade, including repair and recoating of the floating gas storage covers. The two digesters are now linked and operated in parallel with a design detention time of 35 days at mesophilic temperatures The design operating solids concentration is 5.5%, however neither the solids content nor detention time have been achieved to date, with 20-25 days detention at 3-3.5% solids having been the typical operating parameters. The digested sludge is dewatered by belt filter press. A sludge cake of 20-26% solids is conveyed by a shaftless screw conveyor to a skip bin for transport offsite to the Council Worm Farm or other use. The plant has also been configured such that waste activated sludge can be directly dewatered by thickener and belt press, though this system is used only infrequently. The sludge dewatering system is fully automatic and can be operated unattended for several hours, the only limitation being the $8m^3$ capacity of the skip bin (Morgan and Farley 1998).

The anaerobic conditions in the digesters results in a significant release of phosphorus into the liquid phase. A precipitant chemical dosing system has been installed so that an aluminium salt can be added to the belt press filtrate sidestream, clarifier inlet or digested sludge prior to dewatering to ensure sufficient released phosphorus can be chemically bound to achieve effluent licence limits (Morgan and Farley 1998).

The increase in digesters solids concentration compared to previous operation, which was about 2% solids, has resulted in gas production excess to that required for digester heating. Two 15kW cogeneration units have been installed to use the excess gas to generate electricity. The digesters are heated by a combination of cooling water from the cogeneration units and a biogas fired water heating system (Morgan and Farley 1998).

5.2.8 Control, Instrumentation and Power Supply

All automatic operations in the plant are controlled through PLCs and the operator interface is through a Citect™ SCADA system. The Selfs Point Plant, Macquarie Point Plant and the Sandy Bay raw sewage pump stations can be monitored and controlled remotely, when there are no operators in attendance.

Online instruments measuring suspended solids, turbidity, phosphorus, ammonia, nitrate, BOD and COD have been installed to monitor these process parameters. The details of these instruments are described in Section 5.6.

The upgraded plant required an increase in the mains electric power supply from 500kVA to 1500kVA, comprising two 750kVA transformers. In addition, due to the sensitivity of the process to upsets of even a few hours, a backup diesel generator has been installed. This generator has a capacity of 200kVA which is sufficient to maintain aeration and all major pumping duties within the plant at up to two times average dry weather flow. This is to ensure that the biomass will remain active during the power outage and the plant will be able to meet all licence conditions as soon as mains supply is resumed. The control system computers and PLCs are operated from an uninterruptible power supply incorporating sufficient battery backup to maintain power until the diesel generator starts up (Morgan and Farley 1998)

5.3 Effluent and Biosolids Reuse

The effluent passes 10.5 kilometres through Hobart giving relatively ready access to a variety of potential users. A number of opportunities have been identified for reusing the high quality effluent along the pipeline route. It is considered that the low nutrient concentrations and high disinfection level will result in the effluent being acceptable for a range of reuse applications. One factor counting against large-scale reuse is the relatively low cost of potable water in Hobart and economic viability is expected to be the major determinant of reuse volumes. During winter months there is considered to be potential for up to 3ML/day of reuse, while in summer a maximum consumption of 6ML/day may be realisable (Morgan and Farley 1998).

A novel use of effluent is as a heat source (or sink) for heat pump systems. Two systems have recently been commissioned, using treated effluent from Self's Point. One system is used to heat swimming pool water and to provide space and hot water heating at the new Hobart Aquatic Centre and the other system provides space heating at a retirement village. Apart from minor commissioning problems these systems have worked well. During winter the effluent is expected to have a minimum temperature of 10°C, which is several degrees greater than can be sourced geothermally as was originally proposed for both sites. Heat pump systems can readily use this temperature as a heat source providing a very energy efficient heating system, with a coefficient of performance in the range 6 to 8kW of heating per kW of electrical power (Morgan and Farley 1998). A minimum flowrate of 30L/s is required to provide sufficient heating capacity at the Aquatic Centre and this is a major factor in any maintenance which impacts on effluent flow (Ray Farley, pers. comm., 1998). A backup heating system is to be installed at the Aquatic Centre and eliminate this reliance on the provision of this flow (Andrew Tompson, pers comm., 1998).

The biosolids generated at the augmented Selfs Point Plant will have undergone mesophilic anaerobic digestion and the design production rate is about 40 m^3 per week at 22-25% solids (ANI-Krüger 1996) The nature of the catchment results in relatively low concentrations of heavy metals and problem organics such as pesticides, thus creating the opportunity for reuse. The biosolids are monitored for a range of heavy metals monthly (Ray Farley, pers. comm., 1998).

Biosolids from the plant are transported to a Council-operated vermiculture operation. After addition of about the same dry weight of conditioning agents, the biosolids are processed into vermicast.

5.4 Process Monitoring and Experimental Methods

5.4.1 Sampling Methods

Sampling of the plant process streams was performed by plant operators and laboratory personnel in accordance with *AS 5667 Water Quality - Sampling: Parts 1 and 10* (Standards Australia 1998 & 1998a) and accepted industry practice (WEF 1996). Early results, particularly those of samples taken from the aeration tanks, were considered to have a significant degree of inconsistency. The variability was principally attributed to slightly different sample collection techniques used by different persons taking the samples. Some of the sampling procedures in locations such as the aeration tanks were trialled and reviewed to verify the representativeness of the samples and consistency of the results. Procedures, which were developed as a result of this review, were subsequently followed by all persons taking samples. (Ray Farley, pers comm., 1998).

The laboratory analysing most of the samples is located at the Selfs Point Plant site. Analyses, such as mixed liquor nitrate in which the concentration can change rapidly, were processed at the laboratory within ten minutes of being sampled. Several refrigerated and unrefrigerated automatic samplers were used to monitor different process streams Both ISCO 3700R and American Sigma 900R automatic samplers were used for those analyses requiring refrigerated collection. The samplers both have the capability of collecting 24 discrete samples at times programmed into the sampler controller The daily composite samples of influent, primary sedimentation effluent, trickling filter effluent and final effluent were taken as 24 discrete samples and then combined in the laboratory in proportions relative to the average hourly flowrate for each sample provided by the SCADA system. The use of 24 discrete samples enables hourly profiles of analytes to be determined when desired.

The sludge blanket levels in both the primary sedimentation tanks and secondary clarifiers were taken by operators, at designated locations in each tank, with the aid of graduated 50mm perspex tubes with a vent valve.

5.4.2 Laboratory Methods

The methods used in the laboratory were in accordance with *Standard Methods for the Examination of Water and Wastewater* (19th edn.) (APHA and WEF 1995) except for those noted below. Total phosphorus was measured by Standard Method 4500-P B 5(2) which uses persulphate digestion in an autoclave. This method was determined through comprehensive laboratory testing to correlate well with other digestion method results both at the Selfs Point laboratory and the procedure is used extensively in other laboratories. Some of the BOD tests were carried out with Merck Oxitops to reduce laboratory costs. Tests were carried out at the Selfs Point laboratory which confirmed work elsewhere that the results obtained by this method correlated well with the Standard Method procedure.

The combined oxidised nitrogen anions (nitrite (NO_2^-) and nitrate (NO_3^-)) and total nitrogen concentrations were measured with a dual channel nitrogen analyser using a chemiluminescent reaction with ozone (O_3) to detect nitric oxide (NO) (Martin, Takahashi and Datta 1996). The method is in accordance with ASTM D5167 and the analyses were performed on a Tekmar-Dohrmann DN-1900 supplied by Ai Scientific Pty Ltd. The volatile fatty acid (VFA) analyses were performed on a Perkin-Elmer gas chromatograph (Model: Autosystem XL) with flame ionisation detection (FID). The colorimetric determinations were performed on a Perkin-Elmer UV-visible spectrophotometer (Model no. Lambda 10). The gas chromatograph and UV-visible spectrophotometer were both supplied by Perkin-Elmer Pty Ltd.

The unstirred settled sludge volume (SSV) and diluted unstirred settled sludge volume (DSSV) were both measured using a standard glass one litre graduated measuring cylinder. The stirred settled sludge volume (SSSV) was determined with a 2 L perspex settlometer of 100mm inside diameter with a stirring rod operating at 3.5 rpm All measurements were based on a settling period of 30 minutes, with the exception of some unstirred SSVs which were also taken at 60 minutes and 120 minutes, to assess whether these determinations were of value. The sludge volume index (SVI), diluted sludge volume index (DSVI) and stirred sludge volume index

(SSVI) were calculated using the appropriate settled sludge volumes and the mixed liquor suspended solids concentration in the secondary clarifier influent.

A nitrification inhibition analyses in accordance with *ISO 9509 Water quality - Method for assessing the inhibition of nitrification of activated sludge micro-organisms by chemicals and waste waters* (ISO 1989) have been performed on the plant influent and milk processor effluent samples following an apparent nitrification inhibition event at the Plant. This test was incorporated into the offer by ANI-Krüger, in the event that nitrification inhibitory influent was potentially contributing to poor nitrification (HCC 1995).

The microscopic identification analyses and enumeration were performed in accordance with methods described by Jenkins, Richard and Daigger (1993), Lindrea, Seviour and Seviour (1993) and Eikelbloom and van Buijsen (1983).

5.4.3 Laboratory Scale Experimental Work

A variety of laboratory scale tests mimicking treatment plant processes have been performed to explore or predict outcomes which may occur in the full scale processes. These tests have used sludges and liquids drawn freshly from the process. All test work was either undertaken or supervised by laboratory personnel in consultation with operations staff and ANI-Krüger engineers. As there were numerous tests individual methods most are not detailed in this study, but all were designed to match actual plant conditions as closely as possible at bench scale (Fran Gilroy, pers. comm., 1998).

One example of the laboratory experimental work undertaken was that of testing the suitability of treating biosolids with lime to precipitate phosphorus. This was performed in an attempt to predict the effects of adding lime slurry to digested sludge prior to dewatering in removing phosphorus and the impact of the pH of the final biosolids. As discussed previously the biosolids are processed by vermiculture after leaving the site and there were concerns regarding the high pH of the amended sludge. Five litres of digested sludge was drawn from the process and mixed with a lime slurry in a ratio previously determined as being adequate to reduce phosphorus return in the belt press filtrate by about 80%. The resulting mix was centrifuged to mimic the dewatering process. One of the samples thus treated was placed outside in conditions simulating stockpiling and was monitored for pH weekly. Another sample was placed in a domestic size Reln Worm Factory worm farm with worm bedding and earthworms. A control worm farm was fed with unlimed digested sludge. Earthworm numbers, activity and appearance were monitored in both worm farms weekly for 6 weeks following a single application of sludge

Other bench scale work has included:

- aeration of digested sludge to determine the impact on reducing soluble phosphorus prior to dewatering;
- assessment of possible phosphorus release in clarifier sludge blanket,
- acetate dosing of return activated sludge;
- determination of activated sludge nitrification rates;
- assessing potential inhibition of nitrification by influent wastewater;
- oxygen uptake rates of the activated sludge, and
- a variety of trials on alum dosing to estimate performance of alum addition at different locations in the process.

Only some of the results from these trials are presented in this study.

5.4.4 Online Instrumentation

The process at Selfs Point is continuously monitored with the aid of several online instruments. The instruments of most relevance to the results presented in this study are the flowmeters, online nutrient and oxygen demand analysers and the dissolved oxygen meters. In addition the mixed liquor and return sludge are monitored for suspended solids and the effluent for turbidity with online instruments. The influent and mixed liquor in the third aeration tank have continuous pH and temperature measurement.

Most of the critical flows in the plant are measured with electromagnetic flowmeters. These include the influent, trickling filter effluent, final effluent and all sludge transfer flows The return activated sludge flow is measured by ultrasonic level over a sharp edged weir.

There is online COD and BOD measurement of the primary sedimentation tank effluent to assess load on the secondary treatment processes. The analysers are located about 10 metres from the sample point. The COD instrument is a STIP Phoenix-1010, which uses ozone as an oxidant. The instrument calculates COD by controlling the flows of dilution water and sample through a reactor so as to maintain at a constant ozone differential. Ozone provides slightly different results to the

Standard Method for COD analysis, which uses dichromate (WEF and APHA 1995), but once calibrated there is a reasonable degree of correlation between the analyser and laboratory. The BOD instrument is a STIP Biox-1010, which maintains a constant dissolved oxygen differential between the inlet and outlet of a submerged media fixed film reactor. The sampling flow and dilution water flows are adjusted to maintain a constant flow through the reactor. The results from the instrument has been shown to reflect the BOD_5 obtained by Standard Methods (Köhne et al. 1986; Riegler 1984). Some results of the online monitoring of the Selfs Point Plant influent BOD from the New Town catchment are presented by Couper and Pearson (1996).

Three nutrients are analysed online. The feed for the nutrient analysers can be drawn from the anaerobic zone, aeration tank B, aeration tank C or final effluent, with the sample point selected by manually operated valves. The analysers are approximately 40 metres from the sample point with the sample supply pipeline residence time of about 60 seconds. Ammonium and nitrate are measured by a STIP Genion 4, which has ion selective electrodes for both analytes. The maximum analysis rate is once every five minutes. The online phosphorus meter is a STIP Photom-15, which can monitor either orthophosphate or total phosphorus. In total phosphorus mode a microwave digestion system is used to convert phosphorus to orthophosphate for analysis. The analysis is based on the Standard Methods colorimetric method 4500-P C (APHA and WEF 1995). The maximum sampling rate is once every 15 minutes.

The initially poor analyser performance was traced to an inadequately conditioned power supply. Particularly when analysing nutrient concentrations in mixed liquor, there has been a requirement to maintain the instruments on a frequent basis. Outputs from all of the analysers have shown a reasonable degree of correlation with laboratory results when suitable time delays compensating for sample transport, pre-treatment and reactions times are included. The analysers are presently considered as providing a good relative measure of the process nutrient concentrations, such as changes over time, while not necessarily reflecting the results obtained in the laboratory of samples drawn from the process. One of the possible problems with the operation of the nutrient units is the 50 metre pipe run and number of bends which create blockage points restricting flow. Some trial work with the STIP ammonia and nitrate buoy online meters indicated quicker response times and values which were better correlated with laboratory measurements.

There is a Danfoss Evita (sensor type Oxy 1100) dissolved oxygen meter in each of the three aeration tanks and a self cleaning Züllig meter with a S12 sensor in Aeration Tank C adjacent to the Danfoss meter. The Danfoss meters provide the dissolved oxygen measurements used to control the aeration rotors in each tank.

Data from the online instruments is collected on the plant SCADA and is archived on a writable CD-ROM system. The Citect™ software can be used to display graphed trends of the data. Much of the earlier data has been lost as a result of computer problems and regular archiving of plant results has only been achieved from April 1998 (Ray Farley, pers. comm., 1998).

5.4.5 Operator Log Books

Log books have been kept by the process operations staff, describing events which are out of the ordinary or which are major process changes. The information contained in the log books includes:

- Times and dates that process units are offline and reasons for shutdown;
- Process set point changes;
- Significant events requiring operation in manual mode rather than in sequence control;
- Sludge blanket levels in the secondary clarifiers; and
- Unusual events in the catchment such as spills of trade waste or wet weather conditions.

These log books have been reviewed for events and information relevant to the interpretation of the results obtained from the laboratory and online instruments.

5.5 Chapter Summary

The Selfs Point Plant has a number of elements which are novel in the Australian context for wastewater treatment and are worthy of review. The plant has a high degree of flexibility, with the incorporation of trickling filters into the BNR process, potential for some flow equalisation, options for controlling dissolved oxygen concentrations and the proportion of anoxic and aerobic conditions in a diurnal pattern and a complete-mix and thickener prefermentation system with dilution water addition. To achieve the optimum performance from this flexibility a variety of online instruments have been installed and a new laboratory has been constructed at the Selfs Point site to provide rapid results for process samples.

In this study a variety of sources of information have been used to assess the operating performance of the plant and review proposed modifications with the potential to optimise process outcomes. These sources have included discussions with the operations personnel and review of operating log books, assessment of laboratory results and data from online instruments and discussions with personnel from ANI-Krüger Pty Ltd.

CHAPTER 6: RESULTS AND DISCUSSION

6.1 Introduction

This chapter presents results from the monitoring of plant performance and laboratory scale studies undertaken to assess or evaluate potential changes to the process. The results and discussion have been presented together due to the wide range of results and related difficulty of presenting and discussing the results in separate chapters. The results are noted in relation to the most appropriate unit process operation, but due to the interrelatedness of the various processes there is a degree of overlap between sections.

The chapter starts with a brief report on the startup of the BNR section of the plant and a summary of the major events since, which were likely to have an impact on the plant performance The chapter steps through the liquid stream processes and the solid handling processes as they follow in the plant, presenting results and discussing the factors considered relevant to these observations.

6.2 BNR Process Startup

Prior to startup of the new BNR process operators undertook a training programme including the topics of activated sludge and biological nutrient removal, both of which were new to Hobart City Council. Several Council employees were provided training at Krüger A/S designed plants in Denmark. A new laboratory designed to meet the greater testing requirements of the new plant, while physically ready some months before the startup, was still in the process of developing methods to ensure consistency and repeatability of results at the time of process startup

A number of potential seed sludges from other plants in the Derwent Estuary area were evaluated for use in assisting the startup of the BNR section of the plant. Microscopic analysis indicated that all either had excessive numbers of filamentous bacteria or were of very short or very long sludge ages (Ray Farley, pers. comm., 1998). The trickling filters were expected to provide an adequate supply of seed nitrifying bacteria.

To initiate biomass buildup solids were retained in both the primary and secondary sedimentation tanks for several days prior to the commencement of feed to the activated sludge system. Wastewater was introduced to the new tanks on March 5 1997 (Morgan and Farley 1998). The rapid conversion of the plant from trickling filter to activated sludge operation was tentatively confirmed by early results of a study using fluorescent in situ hybridisation (FISH), where the proportion of beta proteobacteria were noted as having markedly increased by 18 March 1997 (Blackall et al. 1997). By the end of April the plant effluent was meeting ammonia and total nitrogen concentration requirements. A brief chronology of major events since the startup of the BNR system is provided in Table 6.2.

During the operation of the Selfs Point Plant from startup to the present ANI-Krüger engineers have been responsible for all major process control changes and HCC personnel have in effect been working to implement instructions provided by ANI-Krüger. Many of the process changes have been a result of joint consultation, following reports to ANI-Krüger staff of observed results. The HCC laboratory has where possible maintained a 24 hour turnaround for all results (except BOD_5), which are faxed to the ANI-Krüger office in Sydney for review as soon as they have been checked. The ANI-Krüger personnel also can observe plant operations remotely through the SCADA system, including the online nutrient analysers.

6.3 Summary of Operating Results

6.3.1 Introduction

Since commissioning of the BNR process the effluent COD, BOD, total suspended solids (TSS) and turbidity have all been low. The BOD and TSS are typically less than 5mg/L, COD in the range 25-30mg/L and turbidity 1.5-2 NTU. Low values for these parameters are usually the case with nutrient removal wastewater treatment, though these are probably at the lower end of the ranges typically expected (WEF and ASCE 1992). Data presented in Table 6.1 summarises the average influent, primary tank effluent and final effluent concentrations since the BNR startup. From these averages it can be seen that phosphorus has been the only effluent analyte which has regularly exceeded specification.

Table 6.1 Summary of Influent and Effluent Concentrations Data

Measurement	Average Concentrations (mg/L)							
	SS	COD	SCOD	BOD	NH_4^+-N	NO_x^--N	TN	TP
Influent	297	605	164	314	27.6	0.15	33.9	8.7
Post-Primary	133	400	180	205	28.6	-	33.8	8 0
Effluent	5	29	20	5	1.6	5.1	8.2	4.7

Table 6.2 Chronology of Major Events since Startup of BNR Process

Date	Event
5 Mar 1997	Sewage first enters aeration tanks, startup of BNR section
Early Apr 1997	One of the primary sedimentation tanks taken offline
Apr 1997	Nitrification and denitrification established
Jun & Jul 1997	Difficulties in optimising thickening systems, resulting in excess biomass in the BNR system and bulking sludge
11 Aug 1997	Spill in catchment of 4 m^3 of cream and 1 m^3 of custard
Aug & Sep 1997	Use of offline primary tank for flow equalisation
Early Sep 1997	System to permit direct dewatering of WAS commissioned
19 Sep 1997	Fire at milk processor, receipt of 117 m^3 of milk for processing
Oct 1997	Milk processor shut down due to damage in fire
Nov & Dec 1997	Prefermenter off several times due rag buildup on mixer
27 Dec 1997	Received 7 m^3 of waste skim milk by tanker
Early Jan 1998	Start of trial of primary tank in activated mode
7 to 9 Jan 1998	Fire at pump station, stopping flow from Sandy Bay
12 Jan 1998	Received 26 m^3 of waste milk by tanker
Late Jan 1998	Milk processor commissions wastewater holding tank
Early Feb 1998	Start of sludge withdrawal from digester supernatant ports
12 Mar 1998	Alteration of timing of milk processor holding tank discharge
Early Apr 1998	Further change to milk processor tank discharge
Early May 1998	Trial of acetate dosing into anaerobic zone
Early Jun 1998	Trial of Cadbury's wastewater DAF sludge in prefermenter Primary tank elutriation pump commissioned Dilution water for prefermenter and thickener commissioned
6 Jun 1998	First trial of alum addition to digested sludge before filter press
18 & 20 Jun 1998	Control fault caused alum dosing to operate while not dewatering
21 Jun 1998	Receipt of 28 m^3 of waste skim milk from milk processor
26 Jun 1998	Treated 26 ML in 24 hours due to rain, 14.2 ML through BNR
Late Jun 1998	Proportion of flow to trickling filter reduced overnight Digested sludge dewatering moved from peak flow to later in day
6 Jul 1998	First trial of liquid polymer on waste activated sludge thickener
11 Jul 1998	Possible nitrification inhibition event
Jul 1998	Thickener dilution water fault, poor prefermenter performance
Jul & Aug 1998	Digester foaming overflowing to inlet wet well
24 Aug 1998	Diversion of digester foam to offline primary settling tank

The specification for the plant required effluent limits for ammonia nitrogen, total nitrogen and total phosphorus and the results for each of these analytes are provided in Figures 6.1, 6.2 and 6.3.

6.3.2 Effluent Ammonia Nitrogen

Ammonia removal has generally been good with effluent typically having a concentration of about 1mg/L NH_4^+-N Consistent performance to the specified limit of 2mg/L appears achievable in winter though the criterion does require an increase in MLSS to ensure sufficient sludge age. There have been several periods of high ammonium in the effluent since BNR startup and these are discussed.

In July 1997, the biomass quantity in the BNR system exceeded design parameters due to several factors restricting the thickening and dewatering throughput, including control system faults. In combination with high SVIs the focus of the process control focused on sludge retention at peak flow. Ammonia removal probably declined as a result of the high unaerated mass fraction and significant endogenous respiration causing aeration capacity to be a limiting factor.

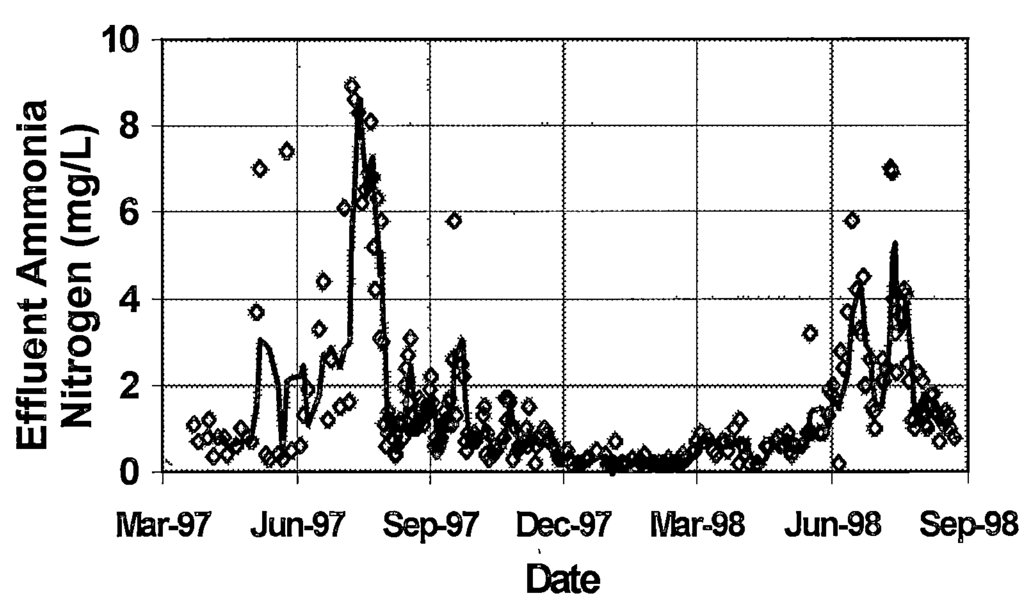

Figure 6.1 Effluent Ammonia Nitrogen for March 1997 to August 1998

In late September the milk processing facility had a major fire and requested the disposal of 117,000 litres of raw milk. The waste milk was originally fed to the trickling filters and BNR at a slow rate, however this soon overloaded the aeration system. Treatment of the remaining 70,000 litres of milk included a major portion being pumped directly to the digesters, reducing the aeration requirements. The effluent ammonia rose as the aeration capacity was exceeded.

In June 1998 the effluent ammonia rose as the temperature fell. This was found to be due to the reversion of dewatering digested sludge during the hours of peak influent load. The additional ammonia from the digested sludge filtrate was in excess of the process nitrification capacity at peak load. The additional load was estimated at 4kg/hr of NH_4^+-N or 30-40% of the peak daily load entering the BNR system. The dewatering was moved back to a late afternoon start moving the ammonia load to a time when there is spare capacity in the BNR system and ammonia concentrations dropped in response to this change.

In early July 1998 the effluent ammonia nitrogen rose rapidly over about one day. The reason for this rise is unknown, but was considered as possibly being due to influent containing inhibitory substances. The influent is undergoing tests for nitrification inhibition, but none of the samples tested to date have been proved to cause nitrification inhibition as tested in accordance with ISO 9509.

Nitrification in the BNR process appears to be less than optimum, possibly due to alkalinity limitations or a high unaerated mass fraction, both of which are discussed below. The trickling filters have provided a higher degree of nitrification than was estimated in the BNR upgrade design.

6.3.3 Effluent Total Nitrogen

Denitrification has generally been adequate except for the period of several weeks following the fire at the milk processor. Several of the peaks in high effluent total nitrogen are as a result of high ammonia nitrogen concentrations, such as those occurring in July 1997 and June 1998.

In the week immediately following the fire in late Sep 1997 denitrification was good while the waste milk was being processed. The processor however was offline for about 6 weeks while repairs were made to the factory. The reduction in organic load to the plant reduced the BOD available for denitrification and effluent daily composite total nitrogen concentrations reached a maximum of 14mg/L, principally comprised of nitrate nitrogen.

The delayed discharge of wastewater from the milk processor and improved fermentation of the primary sludge have reduced the variations in denitrification performance. The trickling filters have also contributed a degree of denitrification in excess of the process design.

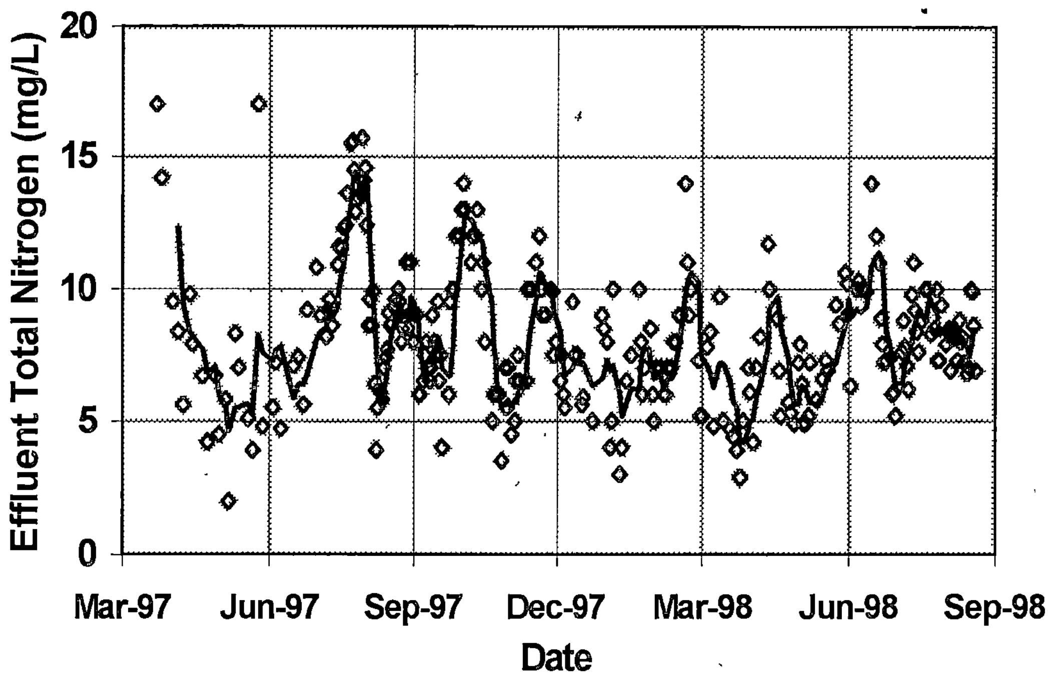

Figure 6.2 Effluent Total Nitrogen for March 1997 to August 1998

6.3.4 Effluent Total Phosphorus

The BPR process has performed at an inadequate level for most of the period. Some of the low values have been due to waste milk deliveries or spills and normal metabolic uptake had been sufficient to remove the influent phosphorus

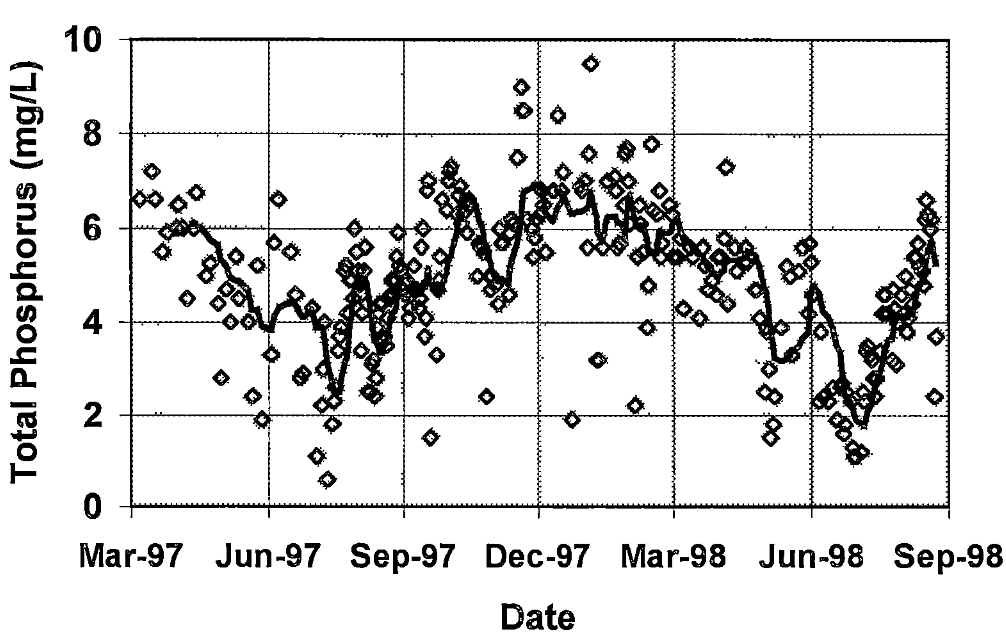

Figure 6.3 Effluent Total Phosphorus for March 1997 to August 1998

From January 1998 onwards more stable operation, the commencement of activated primary operation and commissioning of the milk processor balancing and storage tank has resulted in a gradual reduction in effluent phosphorus concentration. In early May an external carbon source dosing trial temporarily reduced the effluent phosphorus to low levels.

In early June modifications to the prefermenter and activated primary tank increased the RBCOD supply to the BNR process and the effluent phosphorus dropped to about 2mg/L for 2 weeks. A ruptured recycled effluent pipeline resulted in the prefermenter being taken offline for over a week in early July. During July and August the digesters were foaming and as the overflow passed to the inlet works untreated the concentration of phosphorus entering the BNR system rose. It is estimated that this effectively increased the influent concentration to the BNR process by 2-3mg/L based on total plant flow. An effluent result of 2.4 mg/L total phosphorus was obtained following piping modifications to store the overflow liquor for reintroduction to the digesters.

6.4 Catchment Impacts

6.4.1 Trends in Influent Characteristics

While the catchments for the Selfs Point Plant have seen little development, during the period since startup of the BNR system, there have been some trends in the influent characteristics with changes in concentrations of several analytes.

Regression analysis of the influent data over the most recent 12 months has shown significant increases in BOD, TSS and TP; virtually no change in COD and a drop in TN. The estimated changes during that 12 months are that BOD has increased from 240mg/L to 350mg/L, TSS from 235mg/L to 290mg/L, TP from 8.4 to 9.1mg/L and TN has dropped from 34 to 33mg/L. It is difficult to understand from these values what may be occurring in the catchment and this needs further investigation.

The rise in BOD when there has been no significant change in COD could be a function of the storage tank being operated by the milk processor, in which the wastewater undergoes a degree of fermentation prior to discharge thus increasing the more biodegradable fraction as would be indicated by the reduction in the COD:BOD ratio. This however would not explain the greater total suspended solids, unless some of the organics are being converted from soluble organics or emulsions not picked up by the suspended solids laboratory procedure into particulate matter which can be measured.

The increase in total phosphorus in the influent does not appear to be realistically explainable in terms of changes at the milk processor and it is considered that the commercial laundries and other major detergent users in the catchment will have to be reviewed for changes in throughput or in the types of chemicals they have been using. The decrease in the influent total nitrogen is small but appears to have continued a trend since Sandy Bay data was gathered in 1995, but again there is no ready explanation for this trend and it is likely that a detailed review of the catchment would have to occur for the factors for this trend to be determined

6.4.2 Major Events in the Catchment

6.4 2.1 Fire at Milk Processor

In late September 1997 a major fire at the milk processing facility halted milk production for about 6 weeks, reducing the organic load on the Selfs Point Plant during this period Milk deliveries had already been received at the Plant prior to the fire and the factory manager requested that Council accept 117,000 litres of milk as waste. The milk was delivered to Selfs Point and transferred to the stormflow retention tank. With a total organic load estimated at about 15,000kg of BOD, the load was about five times the normal daily influent load. During the first 24 hours after the delivery about 30,000 litres was pumped into the plant, which resulted in dissolved oxygen levels of <0 5mg/L at full aeration. The plant did however meet specified effluent requirements except for ammonia nitrogen of 2.6mg/L, with what in effect was a greater than 150% organic overload. There was however a concern that the additional sludge production would exceed the thickening capability. The addition rate was dropped to about 10,000 litres per day. To reduce the activated sludge production and the load on the BNR process the decision was taken to transfer a portion of the milk direct to the anaerobic digesters and direct dewater some of the activated sludge so as to be able to process the milk within a reasonable timeframe All of the waste milk was treated within 7 days.

6.4 2.2 Fire at Sandy Bay Sewage Pump Station

On the evening of 7 January 1998 the No. 2 Pump Station at Sandy Bay, which transfers Sandy Bay catchment sewage to Selfs Point, there was a fire in one of the variable speed drives. Due to concerns that the carbon soot from the fire had entered other electrical equipment and could potentially cause other problems the restart of the pump station did not occur for 48 hours until all critical equipment was fully cleaned.

This event reduced the influent flow to the Selfs Point Plant to about 4 2ML/day or a 55% reduction compared to normal flow. The influent COD concentrations rose by over 50% showing the dilution effect of the primarily domestic quality Sandy Bay flow. The effluent quality improved with the effluent total nitrogen dropping from 8mg/L to 4mg/L. The effluent phosphorus concentration was unchanged, indicative of the poor EBPR performance at that time.

6.4.2.3 Commissioning of Storage Tank at Milk Processor

As part of liquid trade waste improvements the milk processor converted an existing cellar into a balancing/storage tank for controlled discharge of their stronger wastewater, during the latter stages of 1997. The tank, with a capacity of 60m^3 or about 30% of the daily discharge from the site, was expected to balance out most of the pH swings which had previously occurred Monitoring in 1996 had shown factory effluent with pH of up to 12 5, and peaks of over pH 10 at the Selfs Point Plant. Following the commencement of flow from Sandy Bay the peaks had reduced to about a pH of 9.5, which were still an obvious concern. The commissioning of the storage tank has resulted in the pH being far more uniform at the Selfs Point Plant influent with pH peaks now less than 8.5 confirming the pH balancing performance of the tank.

In February the automatic pumpout of the tank commenced. Council officers had previously assessed the residence time of the sewer from the milk processor to the Plant as about 2 hours during normal working hours. Thus the request was made to the milk processor to discharge the tank during the period from 12am to 4am. However, the actual residence time of the sewer during low flow was found to be closer to 6 hours and the concentrated discharge arrived in the BNR tanks at peak daily flow and the dissolved oxygen levels were depressed and remained so for several hours, indicating an overload of the aeration system. The milk processor had also set the system to pump down the pit fully when the high limit was reached, which compounded the problem with the Plant receiving two loads of concentrated waste within a short period. The impact of this peak load is indicated by the online analyser results in Figure 6.4 The figure shows a peak in COD online analyser of the flow leaving the primary settling tanks and this is followed by dissolved oxygen concentration depression in Aeration Tank A.

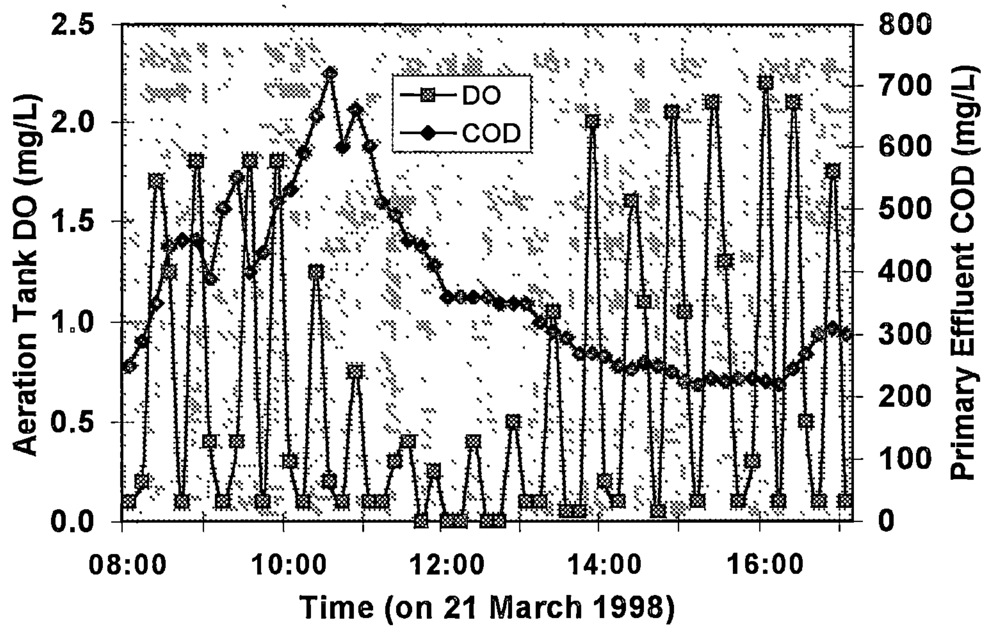

Figure 6.4 Online COD and Aeration Tank DO Readings after Milk Discharge

The control of the tank operation was modified in early April so that only small quantities were discharged at one time once the tank was full and the controlled pump down of the tank was programmed for 9pm to 1am. The offpeak discharge has significantly improved the diurnal load profile, with about 4-5% of the daily BOD load having been transferred from the peak morning load to the low flow period after midnight

6.5 Primary Sedimentation

6.5.1 Initial Primary Sedimentation Tank Performance

The upgraded plant was designed to include the two existing primary sedimentation tanks with a hydraulic residence time of 1.55 hours at average dry weather flow (ADWF), based on a design flow of 435 m^3/hr and a tank volume of $680m^3$. Performance during the first 6 to 8 weeks of these tanks following startup of the BNR process indicated that the removal was greater than anticipated and that the greater BOD removal was adversely impacting on the BOD:P and BOD:TN ratios. One of the tanks was taken offline and emptied, reducing the HRT to about 0.85hrs and resulting in an overflow rate of about 75 $m^3/m^2/d$ at ADWF. The removal efficiencies for design and actual operating for two and one tank operation are given in Table 6.3.

Even with one tank online the removal efficiencies are greater than design. The design values were based on "experience with actual operation of approximately 15 full size wastewater treatment plants in Denmark" (ANI-Krüger 1996:10).

Table 6.3 Primary Sedimentation Operation with One and Two Tanks

Settling Tank Parameters			Removal Efficiencies (%)				
Mode	HRT (hr)	Weir rate ($m^3/m^2/d$)	COD	BOD	TSS	TN	TP
Design-2 tanks	1.55	42	25	25	45	5	10
Two tanks	1.7	38	39.9	36.4	62.8	7.1	12.8
One tank	0.85	76	33.8	30.3	53.4	4.6	8.5

The BOD:TN ratio which dropped from 9.3 at the plant influent to 5.2 for the primary effluent with two tank operation, was 6.1 with one tank in use. The BOD:TP ratio showed similar trends with an inlet BOD:TP of 36 dropping to 23 and increasing to 26 with one tank operation. The two tank operation was getting close to the lower commonly accepted limits for successful nutrient removal of about 5:1 for BOD:TN and 20:1 for BOD:TP (Randall, Stensel and Barnard 1992). These typical values are lower than that required at Selfs Point due to the incorporation of the trickling filters which remove BOD but not phosphorus and only denitrify to a small degree. A further factor is that the BOD proportionately drops to a greater degree during the hours 12am to 7am than TP, thus the ratio is more unfavourable during this period of the day than is indicated by the daily composite concentrations.

The reason for the greater than anticipated removal rates is not known, but could include factors as discussed in Section 4.3, such as:

- the relative freshness of the sewage (i.e. less than 12 hours sewer residence time);
- the effect of the aerated grit chamber in pre-aerating the sewage;
- the relatively high length to width ratio of the rectangular tanks of 6:1;
- that some of the flow is transferred by force main; and
- the short solids residence time in the tank(s), which was less than 0.5 days prior to conversion to activated primary operation.

6.5.2 Settling Removal Efficiency with Activated Primary Operation

In January 1998, due to concerns about there being insufficient RBCOD for EBPR the primary tank was converted to "activated" operation by building up a sludge blanket of an estimated 2 days SRT. This operation has been maintained apart from a brief period when the elutriation pump was installed in May 1998. This change was expected to reduce the removal efficiency due to the higher sludge blanket and biodegradation of the sludge under anaerobic conditions resulting in a poorer settling sludge.

Table 6.4 Removal Efficiencies of Usual and Activated Primary Tank Operation

Settling Tank Operation		Removal Efficiencies (%)				
Mode	SRT (days)	COD	BOD	TSS	TN	TP
Settling only	<0.5	33.8	30.3	53.4	4.6	8.5
Activated	about 2	25.2	29.9	50.9	-4.1	3.6

The results in Table 6.4 (based on over thirty results for each mode) indicate that there was a reduction in the removal efficiencies for all the analytes, with the most significant change being that of COD. This drop was not reflected in the BOD results, which is somewhat surprising in that an increase in BOD from degradation of some COD would have been expected due to fermentation within the tank.

Factors which may have impacted on these results is that the milk processor was offline for a significant portion of the period in settling mode (May to December 1997), and was controlling discharge during the period of activated mode (Jan 1998 onwards). The first factor would have resulted in a change in influent characteristics for that period and the second in more organic load reaching the plant during periods of lower flow, with the milk processor organic load having undergone several more hours of retention prior to reaching the plant.

The reason for the negative total nitrogen removal rate in activated mode (-4.1%) is not known and was not reflected in ammonia nitrogen removal efficiency which was 0% ±1.5% in all modes. The only potential source of additional nitrogen is overflows from the digester, but these were not noted as a frequent occurrence during the period. While this anomalous result may be due to the analysis method, further work is to be undertaken to assess the factors contributing to this result.

Insufficient data has been gathered since the installation of the primary sludge elutriation pump to fully assess the impact of this operation on the primary sedimentation tank performance. The reason for this is that digester overflows enter the plant as a sidestream between the two monitoring points, and this has been occurring for much of the period since the pump has been commissioned as discussed below, greatly increasing the load on the system.

6.5.3 Activated Primary Tank Prefermentation Performance

The performance of the activated primary tank operation as a prefermentation system has to date only been assessed through the measurement of soluble COD, with further work to be undertaken assessing the VFA generation rate. While limited data is available from prior to conversion to activated operation the soluble COD showed little change over the primary tank, in settling only mode.

In the period January 1998 to beginning of June 1998 the average influent soluble COD was 162mg/L, while that of the primary effluent was 188mg/L. The increase of 26mg/L in soluble COD indicates that there is a degree of fermentation occurring in the primary tank. Only limited VFA measurements of the primary effluent have been taken to date (5 samples). These samples had a mean concentration of 13 mg/L of VFA (as acetic acid), of which virtually all was acetic acid.

A comparison of the increase in soluble COD (26mg/L) over the primary tank with the VFA content of the primary effluent (equivalent to about 14mg/L COD), there would appear to be virtually no VFA in the influent to the plant as discussed in the following paragraph. No samples of the influent have yet been analysed for VFA content and this work is to be undertaken in the near future to confirm the low RBCOD content of the influent. The low value for VFA is expected as a result of the catchment characteristics as discussed in Section 4.2.3.

From work performed on the prefermenter noted in Section 6.9.4 the proportion of VFA COD to SCOD in the prefermenter effluent at lower solids concentrations is 55%. The conditions are very different in the prefermenter where the soluble COD concentrations are much greater and pH is lower. If however, for the sake of discussion, it is assumed that 55% of the additional SCOD in the primary effluent is VFA COD generated in the primary tank (55% x 26 mg/L = 14.3 mg/L), this would indicate that virtually all of the VFA was generated in the tank i.e. none in the influent. The volumetric efficiency (after v. Münch and Koch 1997) based on this assumption about 18mg/L/h.. If this is actually the case then this value would appear to be very high for an activated primary (the Salmon Arm Plant has a value of 1.4 mg/L/h (v. Münch and Koch 1997)). If this reflects the actual situation the high value is almost certain to be due to the very good settling performance of the tank at Selfs Point requiring only a small HRT and thus a small volume. The Salmon Arm Plant activated primary tanks averages an HRT of 6 hours, approximately 7 times that of Selfs Point, but at 3-5 days had only a slightly longer SRT. Further work is required to assess whether this efficiency value does reflect the actual performance.

The impact of the activated operation has also been reflected in a reduction in the diurnal swing in soluble COD (SCOD) in the primary effluent and higher SCOD in the period 4am to 8am. Samples were composited for six 4 hour periods over the day and measured for SCOD. There was a wide spread of results on a limited data set, however the results considered typical for the peak and low periods for SCOD are provided in Table 6.5.

Table 6.5 Diurnal SCOD for Influent and Primary Effluent in Different Modes

Sample	Soluble COD (mg/L)	
Load Period and Time	Peak (12pm-4pm)	Low (4am-8am)
Influent	280	100
Primary Effluent-Settling Only	270	120
Primary Effluent-Activated Mode	280	180

There were insufficient results from prior to activated mode operation to undertake statistically meaningful comparison of the operating modes. However as can be seen from Table 6.5 the averages of the results that were taken, indicated that there was a higher soluble COD in the primary effluent during the low flow period of 4am to 8am in activated operation (180mg/L) than that during settling only operation (120mg/L), This difference was further evidence that fermentation was occurring. At peak daily flows there was no noticeable change in concentration, which is likely to be due to the much shorter HRT during this period and effective dilution.

The increase in SCOD during the 4am to 8am period is significant in that it indicates the provision of additional RBCOD to assist EBPR during this period of the day, when there is low load. While the work by Temmink et al. (1996) related to a period of low organic loading of 20 hours, it may have been the case that a low organic load for about 6-7 hours overnight had a regular adverse impact on EBPR in the BNR process and converting to activated primary operation may have assisted in overcoming this factor.

As discussed above the pump to elutriate the solids in the primary sedimentation tank was commissioned at the end of May 1998, but due to digester supernatant from foaming entering the primary tanks, the effect of this pump on prefermentation efficiency is unknown at this time.

6.6 Trickling Filters

6.6.1 Introduction

As described previously the trickling filters were incorporated into the BNR plant, to provide BOD removal and nitrification. The process design for the plant assumed an effluent from the trickling filters of 75mg/L of BOD and 30mg/L of total nitrogen, with no stated assumption of nitrification rates. Table 6.6 summarises the influent and effluent data for the trickling filters for 8 hour composites taken from 8am to 3pm. During this period the applied organic and nitrogen loads are greater and thus the removal efficiencies are less than occurs over a full day. However, the future operation of the trickling filters is for perhaps 50% of the flow to be applied to them during the high flow period and a reduced proportion at low flow, thus the peak flow performance is more critical.

Table 6.6 Organic and Nutrient Removal Performance of Trickling Filters

	Concentration (mg/L)						
Flow	COD	BOD	TSS	NH_3-N	NO_3-N	TN	TP
Influent to Filters	395	200	127	28.5	0	34.0	8.1
Effluent from Filters	198	78	79	7.8	14.7	25.6	8.1
Removal Rate	50%	61%	38%	73%	-	25%	0%

The filters have provided close to the expected removal performance during the peak load of the day and thus have performed better than design expectations. The filters have achieved close to full nitrification and have also performed a significant degree of denitrification (25% of total nitrogen has been removed). This degree of denitrification may be possible due to the relatively thick biofilm the rock media can support. The relatively heavy applied organic load may also creating anoxic conditions in portions the biofilm.

The nitrification performance of the filters dropped only slightly with temperature. Although individual results are a little erratic during the warmest 3 months the nitrification rate averaged 78% for a mean aeration tank temperature of 17.5°C and

68% for a mean temperature of 14.0°C for the coldest 3 months. The nitrification performance during the winter months of 1998 has been about 74% reflecting the warmer ambient temperatures of this winter (a mean of about 15.0°C and a low of 13.8°C in the aeration tanks) and a reduced organic load during the peak flow, with more flow passing to the BNR system and the milk processor holding back organic load for offpeak load discharge.

6.6.2 Oxygen and Alkalinity Limitations

Results from an hourly profiling of the trickling filters taken on 2 July 1998 (shown in Figure 6.5) appear to indicate that removal of COD is consistent and independent of variations in hydraulic and organic load over the daily peak flow (Note: In Figure 6.5 the flows shown are the plant inlet flowrate, approximately double the flow passing to the filters). The nitrification performance however drops significantly over the hours of highest flow, which is most likely to be due to the higher applied organic load and either the heterotrophs outcompeting the nitrifiers for oxygen or the capacity of the nitrifiers being exceeded. The trickling filter effluent ammonia concentration shows a virtually linear relationship with flow over the measured flowrates.

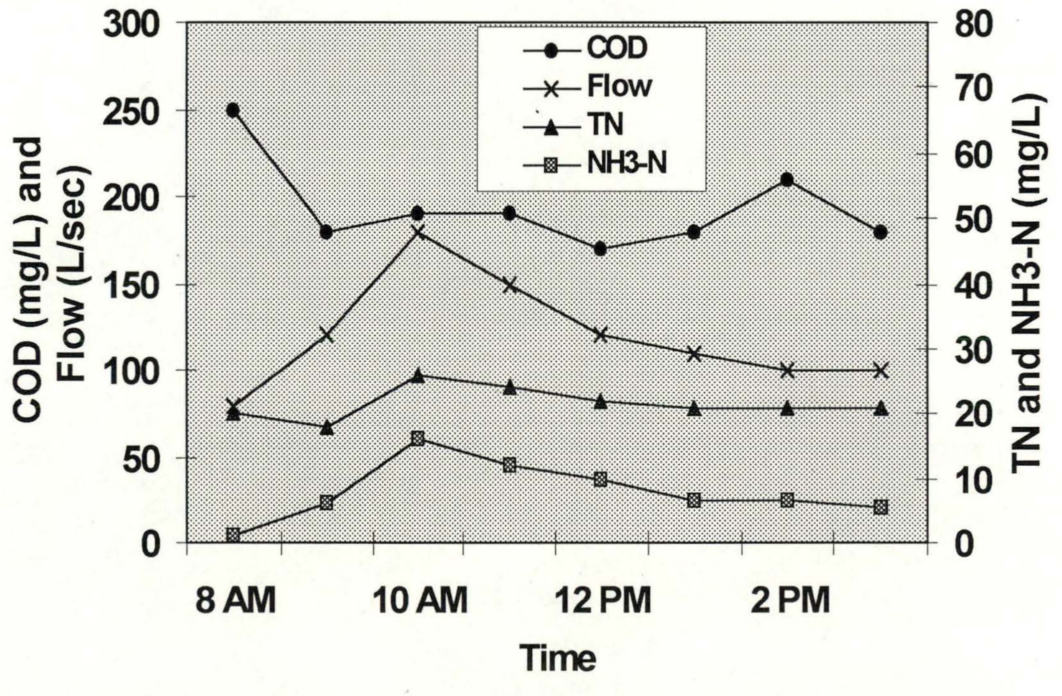

Figure 6.5 Typical Hourly Profile of Trickling Filters Effluent

The total nitrogen concentration of the trickling filter effluent remains relatively constant indicating increased denitrification rates at the higher flows, which may occur due to the greater BOD providing additional organic material for the denitrifying bacteria and also the creation of anoxic conditions, also indicative of oxygen limitations for the nitrifiers.

From testing undertaken at lower flow periods of the day when the ammonia nitrogen in the effluent from the trickling filters is below approximately 5mg/L the alkalinity in the effluent is less than 40mg/L (as $CaCO_3$) Thus at high percentages of nitrification the conversion of ammonia nitrogen appears to be alkalinity limited. The alkalinity of the wastewater entering the trickling filters is typically about 150 mg/L (as $CaCO_3$). Approximately 7.1mg of alkalinity is destroyed per mg of ammonia nitrogen nitrified (USEPA 1993). Dissolved oxygen (DO) measurements of the influent to the trickling filter typically been 3mg/L. If an average bulk liquid DO throughout the filter of 5mg/L is conservatively assumed and an effluent of 5mg/L of NH_4^+-N occurs, then the molar ratio of oxygen (as O_2) to ammonia nitrogen (after Jansen, Harremoës and Henze 1995) is (5/32)/(5/14) = 0.44, which is <1.4 and thus oxygen is considered to be the rate limiting substrate. Due to the diffusional limitations a residual alkalinity of above 2.4 x 5/32 x 50mg/L = 19mg/L is considered as being necessary (after Jansen, Harremoës and Henze 1995). Thus only 130mg/L of alkalinity is available which is only sufficient for nitrification of 18mg/L of NH_4^+-N. This value can be compared with values of up to 24mg/L of ammonia nitrogen being nitrified. The additional alkalinity to nitrify the higher value would appear to be sourced primarily from the denitrification process. The alkalinity limitation is also confirmed by the lower NH_4^+-N concentrations which can occur in the trickling filter effluent overnight such as 1.2mg/L at 7am on 2 July 1998, when the applied ammonia concentration is low (11mg/L) and alkalinity is no longer limiting.

Thus the nitrifying capacity of the trickling filters appears to be oxygen limited at high flows and organic loads and alkalinity limited at low organic loads. The trickling filters also provide a significant degree of denitrification which appears to assist in providing additional alkalinity for nitrification.

6.6.3 Organic and Nitrogen Load Impacts

The flow split to the trickling filters under recent operation has been varied from about 50% of the primary effluent during higher loads to 10-15% during the lower flow and load periods. This operation has been performed so that the BNR system receives a more uniform load during the day, with a decrease in the peak BOD and ammonia loads which would otherwise be experienced. This mode of operation appears to have assisted the operation of the BNR system, reducing ammonia peaks in the system, though detailed assessment of impact is to await the installation of an actuated gate to be automatically controlled by the SCADA system.

With the organic load of the primary effluent varying by a factor of greater than 5:1 between high and low load periods the change in flow split proportion results in the organic loading on the filters varies over a range of approximately 20 to 1 during every 24 hour period. Averaging the load and performance over a whole day under such circumstances would not appear to be appropriate in comparing the trickling filter performance with data in the literature The performance based on hourly composites has thus been used to determine equivalent daily rates for comparison with results elsewhere.

The range of organic loading rate considered as usual to achieve 75% nitrification is 0.1-0.25kg/m^3/d for rock media, and for 50% nitrification is in the range 0.2-0.4kg/m^3/d (based on data in USEPA 1993). During peak flow, which occurs at about 10am daily, the organic loading rate based on an influent BOD of 220mg/L is typically equivalent to about 0.6-0.7 kg/m^3/d, while in the early afternoon the BOD loading rate drops to about 0.4-0.5 kg/m^3/d. From a typical 7am to 3pm profiling of inlet and outlet from the filters (Figure 6.5) it can be seen that the ammonia removal is strongly impacted by the increased organic load from the higher flows, with nitrification efficiency dropping to about 50% at peak flow and recovering to 80% when the load drops. However the nitrification efficiency achieved is over twice that which is expected from the literature.

The ammonia load also varies with flow during this peak period. For the calculated estimates given below the rock media specific surface area is assumed to be 50m^2/m^3 (USEPA 1993). At 10am the ammonia loading is estimated at 1.7gNH_4^+-N/m^2/d (with a removal of about 0.8gNH_4^+-N/m^2/d), dropping to 1.1gNH_4^+-N/m^2/d in the early afternoon (with a removal of 0.9gNH_4^+-N/m^2/d). At loading rates of above 1.2gNH_4^+-N/m^2/d, in trickling filters performing only nitrifying duty oxygen is considered to be the limiting substrate (WEF and ASCE 1992). Thus the level of nitrification efficiency is greater than half of that which would typically be expected from filters performing only nitrifying duty.

Thus it would appear that the trickling filters are performing at greater efficiency than the literature would suggest for a combined carbonaceous and nitrifying duty A possible reason for these results is that the flow to the filters is very low overnight and thus the organic loading is much lower during this period, which may be disadvantaging the faster growing heterotrophs in comparison with the nitrifiers. The reduction in applied organic load overnight is likely to be reducing sloughing rates of the biofilm which again would be expected to favour nitrifiers. If the total daily loading rates are considered, including both high and low load periods, the average organic loading is about 0.15-0.2kg/m^3/d of BOD and the ammonia loading is 0.5gNH_4^+-N/m^2/d and a nitrification efficiency of 75% fits well within the results reported elsewhere.

6.6.4 Sludge Production

The sludge production from the filters is relatively small compared to the applied load. The suspended solids of the effluent is typically 80mg/L of which a small portion is applied solids load which has not been treated. If all of this is considered to be sludge production and the filters treat a third of the flow (3.1ML/day) as has recently been the case the estimated daily sludge production is 250kg/day dry weight. As typically 130mg/L of BOD has been consumed the effective sludge yield is 0.62kgSS/kgBOD destroyed. The design quantity of sludge production was 560kg/day, however this was based on a flow of 6.25ML/day passing over the trickling filters and a reduction of 100mg/L of BOD, giving an effective sludge yield of about 0.85kgSS/kgBOD destroyed, indicating that the sludge yield is about 25-30%% lower than predicted in the process design. This lower value may have been contributed to by the reductions in organic and hydraulic loads, which in turn lower biofilm growth rates and sloughing rates and thus permit increased grazing by organisms of a higher trophic level.

The quantity of this biomass is important when it comes to calculations concerning the BNR activated sludge system as it is extremely unlikely that it can contribute to EBPR having not undergone the selective pressures of anaerobic conditions. The other sludge in the BNR system thus has to be more efficient at taking up phosphorus than would be the case with an activated sludge only process required to remove the same quantity of phosphorus.

The addition of the trickling filter sludge to the BNR process was considered to be likely to "contribute to the healthy formation of an activated sludge with good settling properties" (ANI-Krüger 1996), based on experience in Denmark. As discussed in Section 6.8 the settling properties during winter at Selfs Point have been relatively poor, but this is due to a proliferation of *Microthrix parvicella*. During the warmer months the settling was very good and the return sludge pumps had to be turned off several times each day to ensure that there was some sludge in the clarifiers for denitrification. Thus when the numbers of *M. parvicella* are sufficiently low the sludge settles extremely well and this could be contributed to by the presence of the trickling filters.

6.7 BNR Reactors

6.7.1 Low Alkalinity and Nitrification

The influent alkalinity to Selfs Point is typically in the range 160-200mg/L as $CaCO_3$. Thus there has been a concern about there being sufficient to achieve nitrogen removal. With an influent ammonia nitrogen of about 30mg/L, if the plant was nitrifying only, there would be insufficient alkalinity to fully nitrify as nitrification consumes about 7.1mg of alkalinity per mg of ammonia nitrogen nitrified. At Selfs Point denitrification is also used and this generates some alkalinity and per mg of nitrogen regains about 50% of the lost alkalinity. For an effluent of 1mg/L ammonia nitrogen and 6mg/L of nitrate nitrogen from an influent of 30mg/L approximately 140mg/L of alkalinity is theoretically required. A concentration of 50mg/L of alkalinity is considered as sufficient in the effluent to ensure stable operation. Thus at Selfs Point the situation is marginal with respect to alkalinity.

Performing denitrification with influent BOD prior to nitrification does assist the situation by increasing alkalinity prior to nitrification. By comparison the trickling filters appear to be alkalinity limited at low load without the alkalinity gained from denitrification.

Results to date indicate that the average alkalinity in Aeration Tank C is 68mg/L as $CaCO_3$. This value being the average then it is likely at times the alkalinity drops below 50mg/L and nitrification could be inhibited by low pH. In a test on 22 June 1998, the alkalinity averaged 50mg/L for the period 10am to 4pm. The pH in aeration tank C has been noted as low as 6.1 on several occasions. The process does appear to be at least partially acclimatised to the low pH as indicated by the continued high degree of nitrification during the low pH events.

Nitrification rates however do appear to be relatively low in winter, and in performing a nitrification inhibition test on the wastewater entering the BNR process, the sludge sample was noted as having a nitrification rate below the level required to ensure accuracy of the results. Another factor which may be impacting on the nitrification rate is the high clarifier sludge blanket levels occurring in winter. When the blanket level reaches 1-1.5 metres, depending on the control settings for the aeration tanks the unaerated fraction could be over 0.6 and the aerobic sludge age getting to the lower limit of what is required to support a nitrifier population. The seeding by the trickling filters of nitrifiers may assist in retaining sufficient population to provide nitrification.

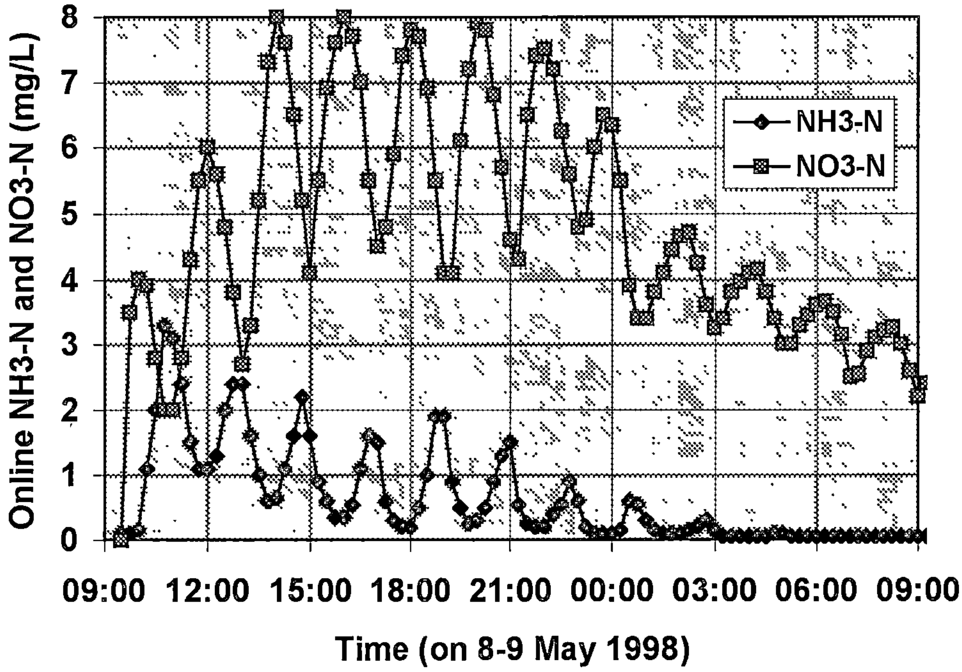

Figure 6.6 Typical Diurnal Ammonia and Nitrate Concentrations in Tank C

The situation in the BioDenipho system is very dynamic and parts of each aeration cycle could reach alkalinity limited conditions. Figure 6.6 provides a typical daily profile from the online ammonia and nitrate analysers indicating the swings in concentration through each aeration cycle in Tank C (the swings are wider in the other two tanks).

If organic load for denitrification is lost then there could be insufficient alkalinity to ensure full nitrification. At Selfs Point this could at times be overcome by restricting flow to the trickling filters to ensure enough BOD reached the BNR system.

6.7.2 Phosphorus Content of WAS

The phosphorus content of the BNR sludge is a good indication of how well the EBPR mechanism is working. Values of above about 4% phosphorus by dry weight are indicative that there is significant luxury uptake of phosphorus. The results for

Selfs Point from January to June 1998 are shown in Figure 6.7. The phosphorus content of the sludge slowly rose during the period. This increase is considered as being partially in response to the activated primary operation, increased consistency in dosing the belt press filtrate for phosphorus and possibly due to the shorter sludge ages used in March and April. While there are no recent measurements, low effluent phosphorus concentrations in June are indicative of good EBPR and it is expected that the activated sludge phosphorus content would be high to reflect this situation.

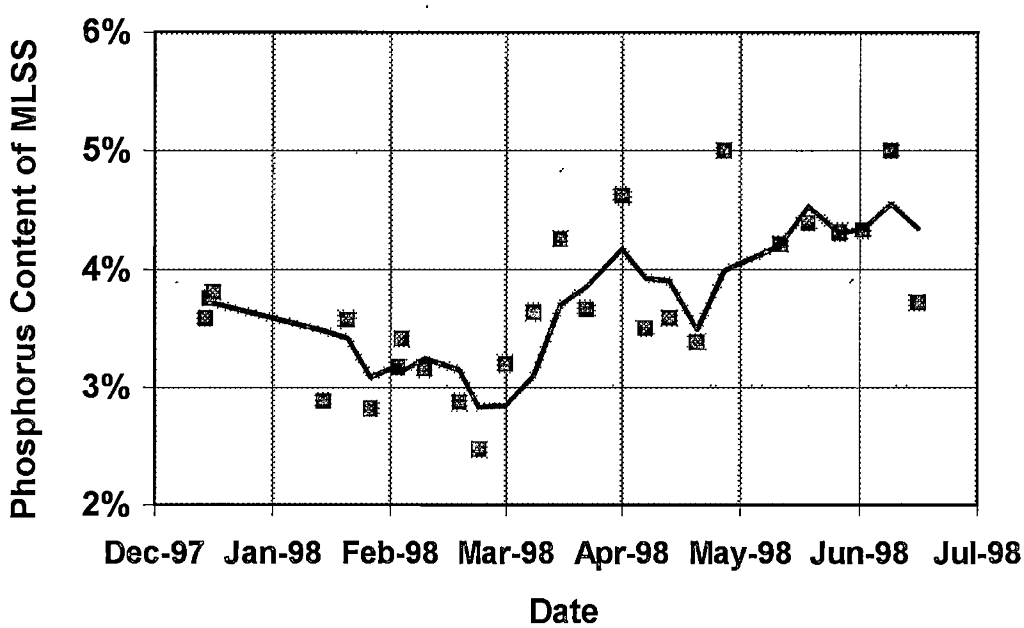

Figure 6.7 Phosphorus Content of Waste Activated Sludge

The most recent results from June 1998 of about 4.3% phosphorus dry weight show that there is a significant degree of EBPR occurring, particularly when it is noted that about 20% of the sludge is non-EBPR sludge from the trickling filters. If it is assumed that the trickling filter sludge has a P content of 1.5% then the EBPR sludge has an equivalent phosphorus content of about 4.9%. These values do however include alum precipitated phosphorus from dosing of the digested sludge filtrate, which at the dosing rate prevalent during the period was equivalent to about 1% of the 4 3% (i.e. 16-18kg/day of chemically precipitated phosphorus).

The estimated wasting rate is 1600kg dry solids per day. Thus it is estimated that about 68kg of phosphorus is removed per day through the waste activated sludge. This is equivalent to about 7 1mg/L of the plant influent. The phosphorus removal in the primary tank is about 0.4mg/L, of which about 0 3mg/L remains in the sludge after prefermentation and is transferred to the digesters. Thus about 7mg/L of phosphorus is removed when EBPR is working well and there are no sidestreams returning significant quantities of phosphorus For the present influent average phosphorus concentration of 9mg/L this is sufficient for the plant to meet specification.

6.7.3 Aeration Requirements for Phosphorus Uptake

During trials of operating Aeration Tank C with dissolved oxygen at about 1mg/L to determine if nitrogen removal could be improved by encouraging simultaneous nitrification/denitrification it was noted that phosphorus removal appeared to be somewhat poorer. One example of phosphorus release occurring at this lower dissolved oxygen level is presented in Figure 6 8 which plots the online soluble phosphorus with dissolved oxygen in the tank. As can be seen the DO depression has resulted in an increase in soluble phosphorus.

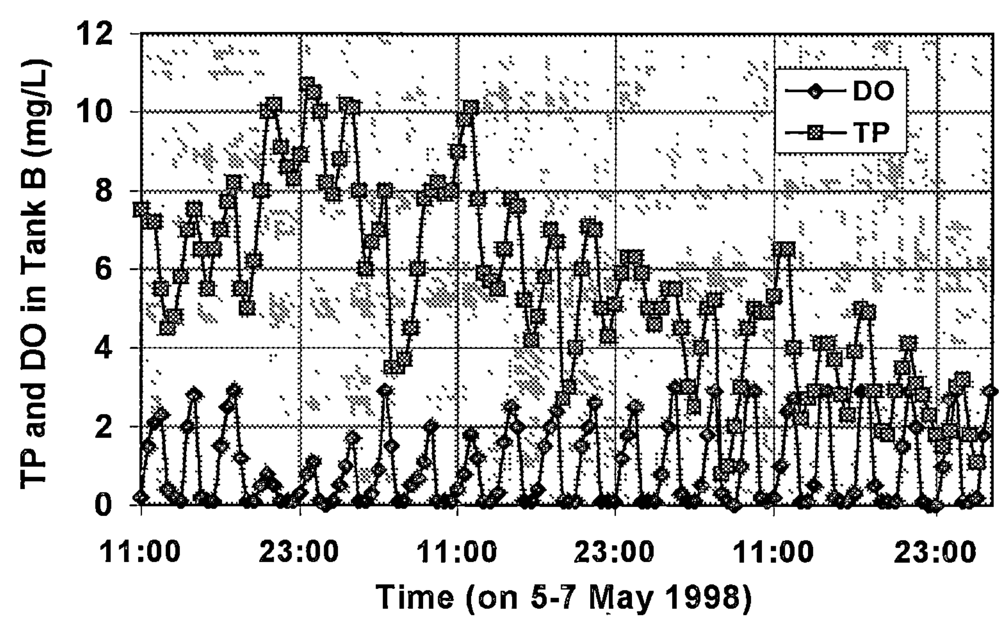

Figure 6.8 Release of Phosphorus at Low Dissolved Oxygen

In an effort to encourage better phosphorus uptake the DO setpoints were raised to 4mg/L for a period. The phosphorus removal did not however appear to be any better than at 2mg/L and the setpoint was returned to 2mg/L.

6.7.4 Volatile Fraction of Mixed Liquor Suspended Solids

The proportion of MLVSS to total suspended solids in the mixed liquor has averaged close to 80%. The original design was for 67% (ANI-Krüger 1996). The relatively high proportion of VSS is thought to be related to the low total dissolved solids concentration of the influent in comparison to the organic load received. With less dissolved solids there is less inorganic ions to precipitate out. The value of 80% is greater than that typically experienced in Denmark (Per Nielsen, pers. comm , 1998).

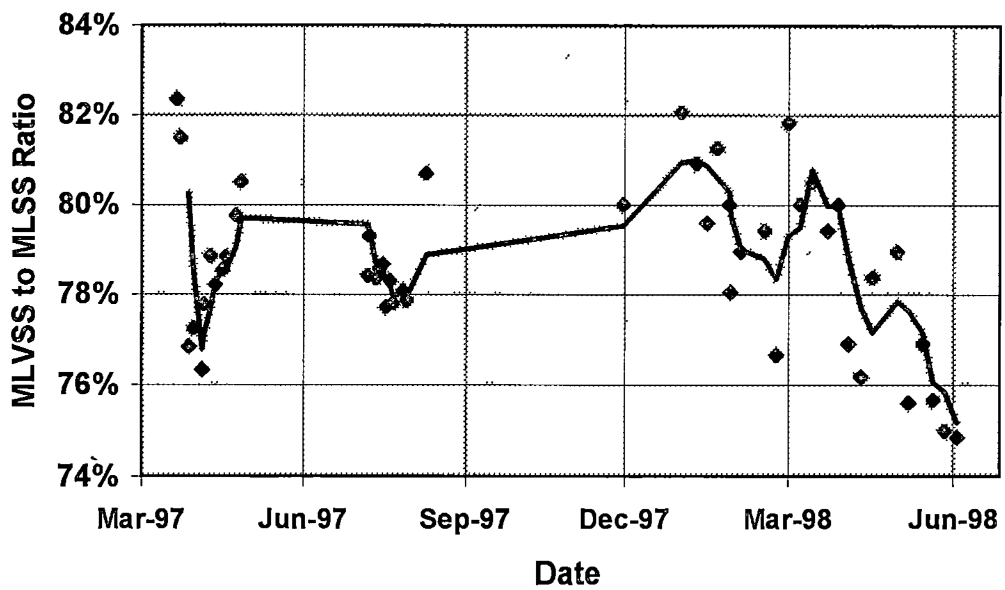

Figure 6.9 Changes in Mixed Liquor Volatile Suspended Solids

Measurements of the proportion of VSS are graphed on Figure 6.9. The VSS proportion dropped earlier 1998 from March through to June and it is considered that at least two factors have contributed to this reduction. One was that the sludge age was increased during this period to ensure nitrification and greater endogenous respiration would have reduced the sludge yield and the proportion of VSS. The other factor is that alum dosing of the belt filter press return only became a consistent operation from the end of March The precipitates resulting from the alum dosing contribute to the fixed suspended solids in the mixed liquor. The alum dosing was set at a rate which was generating in the order of 100kgs of precipitates per day (the actual mass depending on the compounds being formed), thus it would have been expected that the FSS of the mixed liquor would have increased by about 5% once steady state was reached.

6.8 Secondary Clarifiers

6.8.1 Sludge Settlmg Propertes

6.8.1.1 Selection of SVI Test

Following the startup of the BNR process the sludge settling properties were tested using the settled sludge volume and a diluted settled sludge volume test, from which the SVI and DSVI were calculated using the suspended solids concentration. As suggested by the literature at the MLSS used at Selfs Point of 3500mg/L or greater the SVI was found to be of little use. The SSV_{30} was typically greater than 950mL.

Both the stirred and diluted tests have provided reasonable results which appear to reflect the clarifier performance. The stirred test has been made the standard test as on some occasions the diluted test gave no result due to denitrification causing sludge to float. The stirring appears to release any gas bubbles and the stirred test does not have this problem. The stirred test has simpler sample preparation in that no dilution is required, though does require the specialist stirring apparatus.

The unstirred test is still performed and in one test during late summer an SSV of 240mL/1L was measured, little more than the stirred test. Once the stirred SVI reaches about 80mL/g the test appears to become relatively insensitive to sludge settling property changes and the unstirred test provides a better relative measure.

6.8.1.2 Variations of Sludge Settling Characteristics

As is demonstrated by Figure 6.10, the sludge settling properties at Selfs Point have been very seasonal with the stirred SVI dropping to about 80mL/g in late November after being up to 140mL/g the previous winter. The clarifiers had been designed to an SVI (diluted) of 150mL/g (ANI-Krüger 1996). The settling properties continued to improve until March 1998 when the stirred SVI reached 60mL/g. The unstirred SVI was about 75mL/g. The effluent still had very low turbidity during this period despite the very low SVI. There were still sufficient numbers of filamentous bacteria to ensure the condition of pin floc was not occurring. As noted before it is thought that the trickling filters may have contributed to these very good settling characteristics.

At the beginning of 1998 as the mixed liquor temperature dropped below 15°C the sludge settling properties rapidly worsened. This also coincided with the commissioning of the improved prefermentation operations and the additional VFA may have assisted the increase in numbers of *M. parvicella* which was observed. In July 1998 the SVI levelled off at about 120mL/g which is slightly less than the previous year. The minimum mixed liquor temperatures of 14°C were about one degree higher than the previous year and this may have been a factor in the reduction in peak SVI values. The previous year there were problems associated with the thickening systems and the total quantity of biomass in the system and the level of sludge blanket in the clarifiers was higher and these may have contributed to the poorer settling properties.

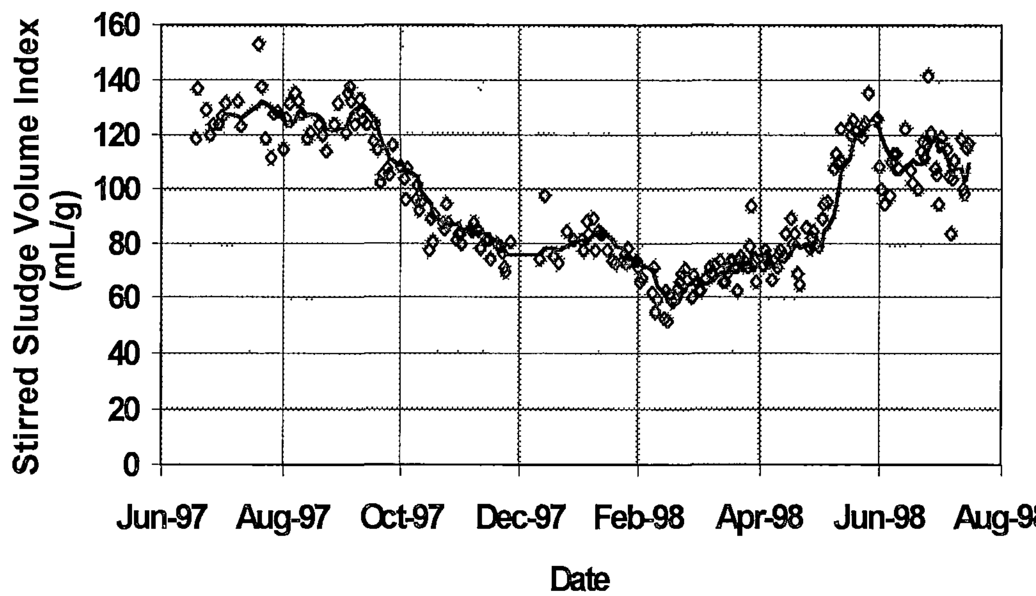

Figure 6.10 Mixed Liquor SVI from July 1997 to July 1998

The settling properties to date have shown a direct relationship to the numbers of the filamentous bacteria *M. parvicella* While in very late autumn type 0803 was briefly in high numbers, *M. parvicella* quickly became dominant. As noted in Chapter 4 most of the factors which appear to favour *M. parvicella* are present at Selfs Point in winter, including primary sedimentation, the use of oxidation ditch reactors, alternating anoxic and aerobic conditions and temperatures below 15°C. There are some scum traps in the BNR system which could act as seeding sources and scum is returned to the aeration tanks. These could both be exacerbating the proliferation of *M. parvicella* at noted by Pitman (1996)

While further work is needed preliminary information indicates that settling properties do worsen significantly if there is a deep sludge blanket in the clarifiers as noted by Hagland, Westlund and Rothman (1998). The greater unaerated mass fraction which is prevalent when the sludge blanket is deep may act as a selective pressure. The presence of a milk processor in the catchment may be a factor as the preferred food source of *M. parvicella* appears to be longer chain fatty acids such as oleic acid. The VFA profile of the prefermenter described in Section 6.9 indicates that there could be significant concentrations of such fatty acids in the filtrate from the prefermenter. *M. parvicella* is noted as being able to take up these fatty acids under anaerobic conditions (Andreasen and Nielsen 1997), and thus could be favoured by the prefermentation products of milk waste.

6.8.2 Denitrification in Sludge Blanket

As the Selfs Point Plant has no pre-anoxic zone for the RAS prior to the anaerobic zone, the process relies on either excess RBCOD to denitrify the RAS in the two $25m^3$ mix tanks prior to the two main $250m^3$ anaerobic tanks or on endogenous denitrification in the clarifier sludge blanket.

There is not considered to be an excess concentration of RBCOD, with most, if not all, required for EBPR, thus the control of the sludge blanket depth is seen as an important control parameter. During summer the sludge settled so well there was difficulty in maintaining a sludge blanket. The RAS pump control was made more flexible through a SCADA alterable sequence, but this system may need further refinement to ensure no nitrate return during summer months.

In winter however, with the poor settling sludge, the morning peak flow pushes a large proportion of the system's sludge into the clarifiers, and due to poor compactibility the RAS concentration remains low and the sludge blanket can build up to 1.5 to 2 metres deep. At this depth it is considered that denitrification would be complete and that phosphorus release may occur. A laboratory test taking a mixed liquor sample from the process found that soluble phosphorus remained constant for about 2 hours before rising at a rate of about 2mg/L per hour. Thus it would appear that if the residence time of the sludge in the clarifier exceeds 2 hours phosphorus release may occur. Based on an assumed average sludge blanket suspended solids concentration of 7000mg/L, the maximum blanket depth at average dry weather flow before phosphorus release would occur is estimated at about 700-800mm, which is exceeded for portions of the day in winter. This is likely to have been adversely impacting on EBPR performance

6.9 Prefermentation

6.9.1 External Carbon Dosing Trial

The biological phosphorus removal performance of the Selfs Point Plant was erratic and relatively disappointing during 1997, despite occasional low effluent results and evidence of some phosphorus release in the anaerobic zone of the BNR system. For much of December 1997 the prefermenter was offline due to rag buildup on the mixer. This buildup had occurred due to regular failures of the inlet mechanical screen.

The activated primary operation commenced on 10 January 1998 and is considered to have increased RBCOD for EBPR virtually immediately. A sample of RAS was taken and dosed with acetate equivalent to 150mg/L. A release of 14 8 mg/L of ortho-P occurred within 30 minutes and a further degree of release occurred over the following 90 minutes The initial release was equivalent to about 5 mg ortho-P/g VSS/hr of phosphorus release. This value is significantly lower than the 18 8 mg ortho-P/g VSS/hr measured by Danesh and Oleszkiewicz (1997) for primary release of phosphorus using fermented wastewater but higher than the 1.8 mg ortho-P/g VSS/hr the same researchers found for secondary release. Thus the sludge appeared to be exhibiting some luxury uptake but well below that for good phosphorus removal.

The poor performance was thought to be due to inadequate RBCOD on a continuous basis To test this hypothesis 2000 litres of 90% acetic acid was added to the inlet of the anaerobic zone over several days commencing on 30 April 1998. The metering pump flow was initially set to 10 L/h, tested by measuring cylinder. The dosing pump flow was noted as dropping to about 4 L/hr on 5 May, with the drop occurring sometime in the preceding 48 hours. The flow was returned to 10 L/h, until the flow was increased to 20 L/h for 24 hours ending 7pm on 8 May. The trial results are provided in Figure 6.11 comparing effluent total phosphorus to acetate dosing rate.

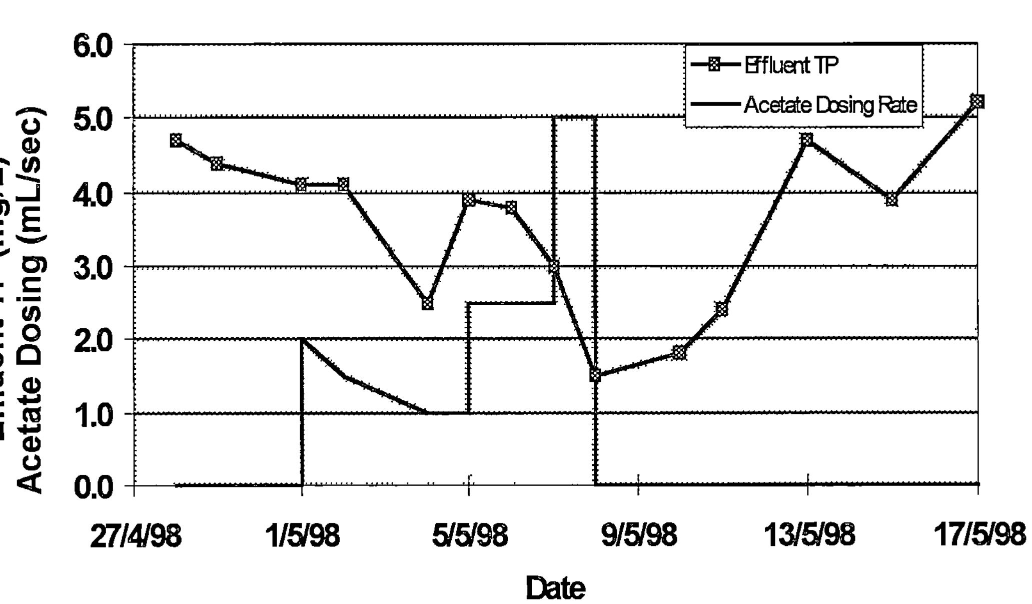

Figure 6.11 Results of Acetate Trial Comparing Effluent TP to Dosing Rate

The online phosphorus analyser readings for the period from 21 April to 18 May are also provided to demonstrate the impact of the acetate addition (see Figure 6.12).

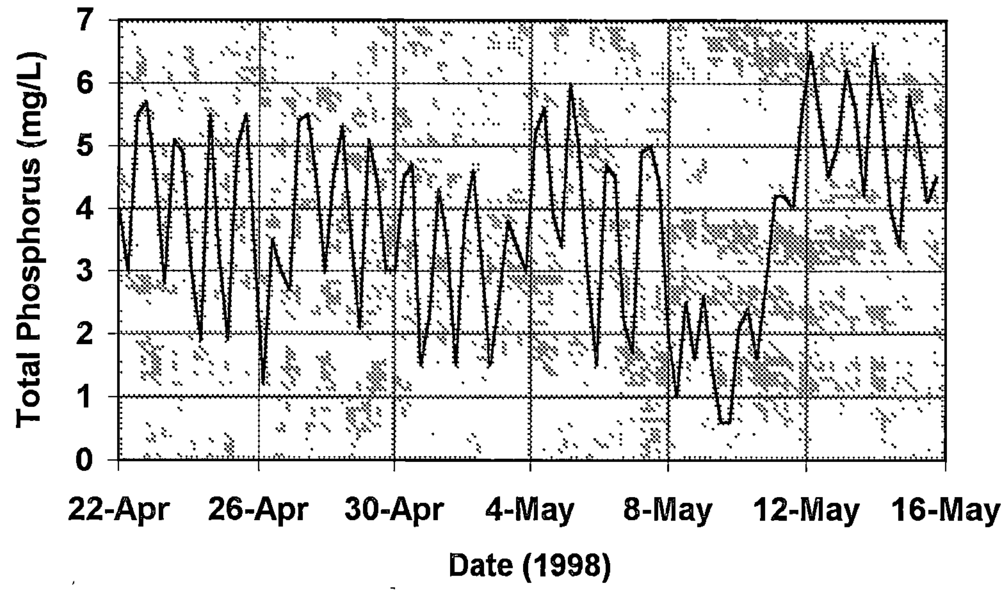

Figure 6.12 Online TP Analyser in Tank C Including Acetate Trial Period

The dip in phosphorus on 4 May is considered as being primarily due to that day being a Sunday, for which the influent total phosphorus content is typically 1mg/L lower than other days of the week (possibly as a result of lower industrial influent).

The drop in effluent phosphorus appeared to lag the dosing by 2-4 days and the low effluent phosphorus remained for over two days following cessation of acetate addition. The return to the previous effluent phosphorus concentrations occurred after the Sunday and the lower load of the Sunday may have been a factor in the apparently rapid increase back to the previous level on the Monday. The results could be an indication that there is a response time lag of about two days by the PAOs to the more favourable conditions.

The test provided strong evidence that the prefermentation system was not providing adequate RBCOD at a consistent level to promote good EBPR and that the prefermentation system had to be improved. A number of actions were concurrently underway to implement this improvement and these included replacement of the inlet mechanical screen, installation of dilution water for both the prefermenter and the thickener and the installation of an elutriation pump for the primary sedimentation tank (discussed previously).

6.9.2 Replacement of Mechanical Screen

The prefermenter was originally designed as a complete-mix unit with a design detention time of about 2 days (ANI-Kruger 1996) The performance of the

the frequent periods of unavailability due to rag buildup on the mixer and other problems resulted in the unit not being particularly effective. With the replacement mechanical screen being commissioned in June 1998 and other works to improve the reliability of the system there is anticipated to be a much reduced downtime for the prefermenter.

6.9.3 Dilution Water to Prefermenter

Following early poor results there were some concerns regarding the effectiveness of the prefermenter and thickener at supplying the generated RBCOD to the BNR process. The prefermenter rotary drum thickener's lowest throughput resulted in a maximum daily run time of 3 hours. With no storage this resulted in a fairly short period of addition, which was considered as not being an efficient use of the filtrate. The relatively high solids content of the primary sludge entering the prefermenter (up to 6%) resulted in a high feed solids to the thickener of about 3.5% total solids. With thickening of up to 8% solids possible 30-50%% of the RBCOD rich liquid phase was typically passing on to the digesters with the thickened sludge.

To overcome these problems it was decided to install dosing water on the prefermenter and also water to elutriate the sludge prior to the thickener. Recycled effluent was used as the water source. The dosing water to the prefermenter permitted a decoupling of the SRT and HRT, by adding a smaller volume of sludge and making up the volume with dilution water. The operation has been converted such that it has a hydraulic residence time of 40-48 hours and an SRT of about 4 days. This mode of operation is performed by pumping down the prefermenter through the thickener to about 50% of full, then adding about 20-25% of dilution water and adding a further 20% of primary sludge during the day. The control of this system has yet to be optimised and there may be gains from operating the prefermenter at the maximum possible volume using appropriate sequence control.

It is noted that the SRT is calculated by dividing the average working volume of 90-100m^3 full by the volume of primary sludge added each day (i.e. 24m^3), giving 3.75 to 4.25 days. The HRT is calculated by dividing the average working volume (90-100m^3) by the sum of the dilution water and primary sludge volumes (i.e. 50-55m^3), giving a time of 42-46 hours.

6.9.4 Prefermenter Efficiency

The majority of the results obtained to date relating to prefermenter performance have been measured in terms of soluble COD. Some recent work at the Plant in early June 1998, briefly summarised in Table 6.7 indicates that the SCVFA content averaged 74% as COD equivalent of the SCOD in mg/L, with the VFA content in mg/L averaging 49 % by weight of the soluble COD in the prefermenter. The work also indicated that at lower soluble COD concentrations the proportion of VFA based COD was a slightly smaller proportion of the total soluble COD (i.e. 55% as COD equivalent for 2000-2500mg/L soluble COD compared to 74% at 5600 mg/L SCOD). It is noted that the average soluble COD during the testwork of 5600mgCOD/L was greater than the values of 3000-3600mgCOD/L typically experienced in the prefermenter and thus caution must be used in interpreting and using the results.

During 1997, the prefermenter was fed about 50m^3 per day from the primary sedimentation tank at an average feed of about 3-3.5% solids. When the primary tank was converted to activated operation in January 1998 the feed solids content increased to 5 5% and volume reduced (20-25m^3/day), to retain solids in the primary tank. With the addition of dilution water the SRT dropped slightly and the HRT significantly.

Table 6.7 Prefermenter VFA Concentrations

Volatile Acid Component	Concentration		Component as % of Total VFA COD
	mg/L	mgCOD/L	
Acetic acid	784	836	20.1%
Propionic acid	977	1479	35 6%
Butyric acid	555	1008	24 3%
Isobutyric acid	86	157	3.8%
Valeric acid	257	525	12.6%
Isovaleric acid	75	153	3.7%
Total VFA	2730	4160	100%
Total Soluble COD	5600	5600	-
Percentage of SCOD	49%	74%	-

The estimated volumetric efficiency for each of the modes of operation are summarised in Table 6.8 based on a working volume of 120m^3 a little less than the nominal volume of 130m^3 which is not fully used due to potential for overflow during the automatic filling process.

Table 6.8 Estimated Typical Data on Prefermenter Operating Modes

Mode	HRT (days)	SRT (days)	TS (%)	SCOD (mg/L)	VFA (mgCOD/L)	r_{VFA}* (mg/L/h)
High Feed Flow	2	2	2.7%	5000**	3700	75
Low Feed Flow	4.5	4.5	3.8%	4200	3000	28
Dilution	1.8	3 7	2.8%	3000	2000	45

* - r_{VFA} denotes the prefermenter efficiency as used by v. Münch and Koch (1997)
** - This value is based on very limited data

The complete-mix mode at low HRT and SRT gave a very high volumetric efficiency. While based on limited data which would need to be confirmed, this is thought to be due to the short SRT. In batch studies the relative rate of VFA production is higher in the first 2-3 days when 50% or more of the maximum VFA production can occur as discussed in Section 4.6. If the Selfs Point prefermenter had a six day SRT the expected value for r_{VFA} would be perhaps 50% or less of the values given in Table 6.8 due to the extended residence time, bringing the values to within the range of 15-30mg/L/h considered as typical for sidestream prefermenters (v. Münch 1998). Thus a higher relative performance based on volumetric efficiency can be obtained at short SRTs. The total SCOD generation in this mode was estimated at about 250kg/day of which 130 to 180kg/day reached the BNR system over a period of three hours due to reasons discussed in Section 6.10.

The efficiencies in the latter two modes were reduced by the use of the activated primary prior to the prefermenter which hydrolysed the more readily fermentable COD, thus reducing the maximum VFA yield obtainable in the sidestream prefermenter. The efficiency values are high when compared with those described by v. Münch and Koch (1997). The increase in volumetric efficiency for the dilution water operation was achieved despite lower ambient temperatures.

While there are several possibilities as to why this occurred it is thought the dilution water by creating a more dilute liquid concentration of VFA, provide a more favourable environment for the fermentative bacteria. The high r_{VFA} values are again considered as being due to the short SRT. The performance of the two prefermentation processes is provided in Table 6.9.

From the values in Table 6.9 it can be seen that the Selfs Point plant has an atypical prefermentation system with the combined activated primary and sidestream prefermenter both having relatively small volumes as evidenced by the short HRT of the activated primary and the short SRT of the prefermenter.

Table 6.9 Prefermenter Operating Data and Parameters

Parameter	Units	Activated Primary		Prefermenter	
		Selfs Pt	Typical	Selfs Pt	Typical
Plant flow	ML/day	9.0	N/A	9.0	N/A
Prefermenter flow	% of inlet	100	100	0.2	2-20
Volume	m^3	340	N/A	120	N/A
Total solids content	%	N/A	N/A	2.3%	1-2%
SRT	days	1.5-2.5	8	3.5-4	6-8
pH		7.0	7.5	5.2	5-6
Effluent C_{VFA}	mgCOD/L	15	13	2000	200-900
HRT	hours	0.85	6-16	48	10-40
r_{VFA}	mg/L/hr	18	1-10	43	15-30
Change in Plant C_{VFA}	mgCOD/L	14	13	23	10-50

Note: The "Typical" values have been taken from v. Münch and Koch (1997) and N/A indicates that the value is not applicable.

Both of the latter two modes noted in Table 6.8 "Low feed flow" and "Dilution" were operated in tandem with the activated primary. The activated primary is assessed as producing about 220kg/day of SCOD, while the two prefermenter modes of undiluted and diluted generated an estimated 100kg/day and 150kg/day respectively. About 60% of the SCOD from the activated primary passes to the BNR process, while the rest passes to the trickling filters. About 30% the SCOD from the undiluted prefermenter operation was lost to the anaerobic digesters as discussed in Section 6.10, while very little of the SCOD from the diluted mode is lost. Thus the SCOD to the BNR process from the two modes was about 200kg/day from the undiluted prefermenter operation and 270kg/day from the diluted mode. The SCOD is also added to the process continuously from the activated primary and over a much greater portion of the day from the thickener as discussed in Section 6.10. The equivalent VFA increase to the influent of the BNR anaerobic zone from both the activated primary and the diluted prefermenter operation is estimated at 25mg/L.

6.9.5 VFA Profile of Prefermenter Contents

From the limited work performed to date it appears that the VFA profile generated in the prefermenter at Selfs Point is significantly different to that typically obtained elsewhere. The comparison with other reported results is provided in Table 6.10 on a weight percentage basis. The VFA profile of the DAF waste from a milk and chocolate processing factory wastewater treatment system used in the trial to increase VFA production is also included for comparison.

The proportion of acetic acid in the prefermenter is well below the values noted in the literature while the concentrations of the longer chained butyric and valeric acids are significantly greater than the values elsewhere. Only limited measurements have been taken to date and further work is to be carried out to confirm that this is a long term trend.

The unusual profile may provide some indication of the influence of the milk processor waste. The use of an activated primary tank prior to the prefermenter may also be a factor in that the more easily fermentable COD has been hydrolysed in that tank to form acetic acid and the more difficult to degrade COD is passing on the prefermenter. The processes generating the slightly longer chain acids (butyric and valeric) may be more suited by the substrate and conditions that the acetogenesis reactions.

Table 6.10 VFA Profile in Prefermenter with Typical Values from Other Sources

Volatile Fatty Acid (wt. %)	Acetic Acid	Propionic Acid	Butyric Acid	Valeric Acid	COD-equivalent (in gCOD/g)
v. Münch (1998)	50-70	24-45	0-13	0-6	1.25-1.36
Barnard (1992)	43-46	41-46	4-12	3-4	1.34-1.38
Selfs Point	30	36	23	12	1.52
DAF waste	68	17	14	1	1.26

The VFA content of the DAF waste appears to be a more typical of a fermented sludge with a profile and COD-equivalent value generally in line with the values in the literature.

6.9.6 Trial of DAF Waste in Prefermenter

A chocolate manufacturing factory in Hobart produces a significant quantity of wastewater treatment sludges from its pre-treatment system. Most of this sludge is generated in a dissolved air flotation (DAF) unit prior to an upflow anaerobic sludge blanket plant. This DAF sludge is treated at Hobart City Council's other treatment plant at Macquarie Point, where it is added to the anaerobic digesters. The DAF waste is high in milk proteins and fats, but was thought to be a good potential source of VFAs with fermentation. A trial involving the addition of two 5000 litre loads of DAF waste to the prefermenter was performed in early June 1998. The trial was performed at this time because the production rate of the DAF waste drops to very low levels from mid-June to mid-August each year during the annual shutdown of the milk crumbing plant at the chocolate factory. The timing of the trial however was very soon after the introduction of changes to the operation of the prefermenter with the addition of dilution water and thus the changes attributable to each of these events was difficult to assess.

The DAF waste had a pH of 4.1, solids content of about 7%, a soluble COD concentration of 10,000mg/L and a VFA content of 2400mg/L, prior to fermentation. Phosphorus removal did improve rapidly in the days after the trial but the removal performance change was mostly attributed to the changes to the prefermenter operation as they persisted after the DAF waste addition ceased. The VFA profile in the prefermenter remained virtually constant during the course of the trial. As the solids in the DAF waste are primarily milk fat and proteins these may well breakdown into similar proportions of VFAs as the primary sludge in the plant which has a high proportion of milk waste. Further trials are to be performed after the production of the DAF waste recommences.

6.10 Thickening

6.10.1 General Performance

The rotary drum thickeners (RDTs) used at Selfs Point have provided reliable and trouble free service, with most of the down time attributable to control issues or the recycled effluent system being down. The capital and operating costs are both low and the units can be operated unattended. There has however been difficulties in thickening the prefermenter and waste activated sludges sufficiently to achieve the 5.5% solids concentration required in the anaerobic digesters.

6.10.2 Prefermenter Sludge Thickening

The prefermenter thickener has provided a high solids thickened sludge from startup due to the better dewatering characteristic of this sludge. The thickened sludge has typically had a solids content of 6-8%. The prefermenter sludge entering the thickener has typically been 3-3.5% solids. Thus 30-50% of the VFA rich liquor was passing to the digesters. Dilution water in the form of recycled effluent is now added both to the prefermenter to reduce the HRT and improve performance as discussed above and to the sludge being fed to the RDT. With these two streams of dilution water over 95% of the VFAs generated in the prefermenter now pass to the anaerobic zone. A further outcome is that the thickened sludge from the RDT has a measurably higher solids content, which is at times greater than 10% solids. This high solids concentration can be a problem if the system is shutdown and not started for several hours as the sludge line can block. This problem is presently overcome by operating the RDT with a lower solids thickened stream, but options are being considered so as to be able to operate at the higher solids content.

The dilution water addition has also resulted in being able to operate the RDT for 10-12 hours per day rather than the previous maximum of 3 hours. As there is no filtrate storage this extension of hours of filtrate addition is considered to have increased the potential for good EBPR significantly.

6.10.3 Waste Activated Sludge Thickening

Thickening of the WAS to high solids has been difficult possibly due to the prevalence of filamentous bacteria reducing compactibility of the sludge. While thickened solids concentrations of 3.5 to 5% have regularly been achieved, this has only been possible at a reduced feed flowrate of 2 L/sec (7m^3/hr) well below the rated capacity range of 20-30m^3/hr. Higher flows can be thickened but only at lower solids content in the thickened sludge.

The low flow has resulted in a long run time per day to achieve the required wasting rate, with the RDT having to operate for 20 or more hours per day. This left no flexibility in the event of a need to increase wasting rates. In the short term this was overcome by installing pipework to allow "direct dewatering". The additional piping permitted the pumping of thickened WAS direct to the belt filter press for dewatering. Wasting rates of 4-6L/sec were possible with direct dewatering as the thickener underflow solids content could be lower. The resulting sludge was however not adequately stabilised for further processing.

A liquid dosing polymer unit has now been installed and on its first full scale trial on 6 July 1998, it was possible to increase the feed flow to the RDT from 2L/sec to 6L/sec with a 4.5 to 5.5% solids content thickened sludge. The polymer flow was about 10L per tonne dry solids (DS) of sludge or about 5kg/tonne DS of active polymer, similar to that previously used with dry powder polymer. Optimisation of the thickener operation with liquid polymer is continuing and higher solids and lower polymer consumption than the first trial are expected over time.

6.11 Anaerobic Digestion and Dewatering

6.11.1 Phosphorus Release in Anaerobic Digesters

The anaerobic digestion of EBPR sludge is expected to result in a high release of phosphorus as the PAOs encounter anaerobic conditions. Such a release of phosphorus was demonstrated by Westlund, Hagland and Rothman (1998) in sludge from a Stockholm treatment plant.

Prior to commencement of BNR operation the filterable reactive phosphorus content of the primary anaerobic digester at Selfs Point was 80-100mg/L (about 20-25% of the total phosphorus). At times when EBPR appears to have been functioning well the filterable reactive phosphorus content of the digesters has typically been in the range 300-350mg/L, with a total phosphorus of about 1000mg/L.

From calculations in Section 6.7 it is estimated that about 70 kg of phosphorus enters the digesters. A daily volume of 60-70m^3 is withdrawn from the digesters for dewatering. With a soluble phosphorus concentration of 300-350mg/L, the mass of phosphorus leaving in soluble form is just over 20kg/day. Thus it appears that about 30% of the phosphorus entering the digester is solubilised. This is high compared to the Danish experience, where the water is harder and contains more calcium and magnesium, but appears to be less than that experienced at the Ballarat South Plant (Rose 1990).

There has been some difficulty in raising the digester solids concentration to the design value as discussed in Section 6.10 and the solids content has typically been a little less than 3%. The impact of raising the digester solids content to the design 5 5% on phosphorus solubilisation is difficult to determine. There may be some concentration effects associated with chemical equilibria which may result in a greater proportion of phosphorus precipitating. However an increased digestion time is likely to reduce the final dry weight of sludge releasing more phosphorus. As the solids content is increased this will be monitored closely.

Aeration trials of the digested sludge were carried out to determine if the concentration of soluble phosphorus could be reduced through CO_2 stripping. The tests carried out in the laboratory were quite variable with some samples achieving a 25% reduction in soluble phosphorus, with a pH rise to about 8, while in other samples very little reduction occurred. As discussed below these trials were suspended once dosing of the feed sludge to the belt filter press was established, as the pH rise from aeration would be negated by alum addition.

6.11.2 Chemical Dosing for Released Phosphorus

As part of the specification for Selfs Point it was acknowledged that phosphorus may be released in high quantities from the digesters and sidestream dosing to precipitate this phosphorus was permitted. Lime and alum were the two most thoroughly investigated. The iron salts were more expensive than alum and had other potential problems as discussed in Section 4.11 and were not considered.

Bench tests with lime dosing of digested sludge to remove sufficient phosphorus resulted in a product with a pH of 8.5-9. As noted in Chapter 5 this sludge underwent simulated stockpiling and the pH did not drop significantly within 6 weeks. Lime amended sludge was added to one worm farm, while normal sludge was added to a control farm. The worms in the farm to which the lime amended sludge was added were noticeably avoiding the sludge and were smaller and less active than in the control. As vermiculture is Council's preferred post treatment process for biosolids, lime was thus rejected as an option for treating digested sludge.

Two dosing points for alum were initially installed, one in the belt filter press filtrate line and the other at the inlet to the clarifiers. The clarifier inlet was considered as possibly being the more effective location due to potentially fewer other reactions and the alum was expected to assist in the settling properties of the sludge. Settling did not however appear to be markedly affected by alum dosing as the turbidity of the effluent was already typically less than 2NTU without filtration. Alum dosing of the effluent was a cause for concern due to the already low alkalinity.

Dosing of the belt press filtrate was successful at a molar ratio of about 1:1 with the soluble phosphorus dropping to about 5mg/L. The resulting precipitate does however return to the process, reducing the ratio of MLVSS to MLSS and generating a less active sludge mass. While it was not thought likely there was also the possibility of phosphorus release occurring at some point in the process.

Alum dosing of digested sludge and filtrate was compared in the laboratory and was found to have phosphorus removal efficiencies within 10% of one another. Thus it was decided to trial alum dosing of the sludge prior to the belt press. This does not appear to have been performed at many other plants Full scale trials found the method to be effective though the sludge dewaterability was found to be slightly poorer and there have been some crystalline deposits build up on parts of the belt press. The deposits have not yet been analysed, but can be fairly readily cleaned off. Alum consumption is estimated at being marginally (<10%) higher using this method, but the precipitate does not return to the process. It is considered that this process provides more certainty that the phosphorus will not re-enter the liquid stream.

One interesting facet of the sludge dosing is that sludge entering the belt filter press still has a soluble phosphorus of about 150-200mg/L after mixing with alum, but that the filtrate has less than 10mg/L soluble phosphorus. The reason for what appears to be an anomaly is being investigated.

6.11.3 Digested Sludge Filtrate Ammonia Load on BNR Process

As discussed in Section 6.3, the effluent results for ammonia nitrogen deteriorated in May and June 1998. Initially this was considered as possibly being due to nitrification limitations A mass balance was performed and it was noted that the belt filter press was being operated during normal working hours, resulting in the ammonia rich filtrate being passed to the BNR process at peak load.

With about 10m^3/hr of filtrate returning to the process the load on the BNR was an additional 3-4kg/hr of NH_4^+-N. At peak flow the ammonium load on the BNR process from the primary effluent is about 10kg/hr, thus the additional flow was a 30-40% increase. At peak flow the BNR process is only just able to nitrify the influent load at colder temperatures and thus any additional load remains untreated and can breakthrough to the clarifiers.

The dewatering process, which had originally been operated during the 4pm to 12am period, had been moved to the day during summer for ease operation. The additional nitrification capacity at warmer mixed liquor temperatures made the timing of this return stream less critical. The belt press operation was returned to afternoon/evening period in June 1998 and the effluent ammonia nitrogen concentrations quickly returned to specified limits.

6.11.4 Impact of Digester Foaming

During the latter stages of June 1998, the BNR system suffered significant foaming, which was related to the increase in population of *M. parvicella*. The possible reasons for this abundance are discussed in Section 6.8 A similar abundance in *M. parvicella* had occurred the previous winter but had not been accompanied by foaming to any significant degree. The foaming gradually decreased in stability and tank coverage during July.

In early July it was noticed that the digesters were overflowing and this appeared to have resulted from the transfer of the waste activated sludge. The foam from the digesters was checked under the microscope and was found to be very similar to that on the aeration tanks, with a high population of *M. parvicella*. As noted in Section 4.11 the bacteria responsible for foaming may not be able to survive the anaerobic conditions in the digesters but the bacteria can still generate foam. Foaming is noted as being more pronounced when gas mixing is used as at Selfs Point, compared to mechanical mixing.

The foaming scum resulted in overflows from the digesters to the inlet wet well. The volume of overflow has been exacerbated by the delays in installing two cogeneration units, which are to use much of the gas being generated as a result of the increasing solids content.

While the volume of overflow was initially thought to be small an hourly profile of the inlet and primary effluent phosphorus over 24 hours, indicated that about 2-3mg/L of phosphorus was being added from the overflowing foam to the influent concentration, adding a load of about 30% on to the normal influent phosphorus load. A number of digester operational changes were made in an attempt to reduce the volume of overflow, such as operating the digesters at a lower liquid level. In late August the overflow was diverted to the offline primary sedimentation tank for either transfer back to the digesters or to dewatering. The daily volume estimated as being collected in this tank was 20,000 to 30,000 litres.

During the period when the overflow was occurring the effluent phosphorus was typically 6mg/L despite the plant otherwise working well. The day following the diversion of the overflow the effluent total phosphorus dropped to 2.4mg/L and dropped further on subsequent days. The online phosphorus and ammonia analyser results for the period prior to and after the diversion are shown in Figure 6.13, indicating the rapid decline in the effluent phosphorus after the 23 August when the overflow was diverted.

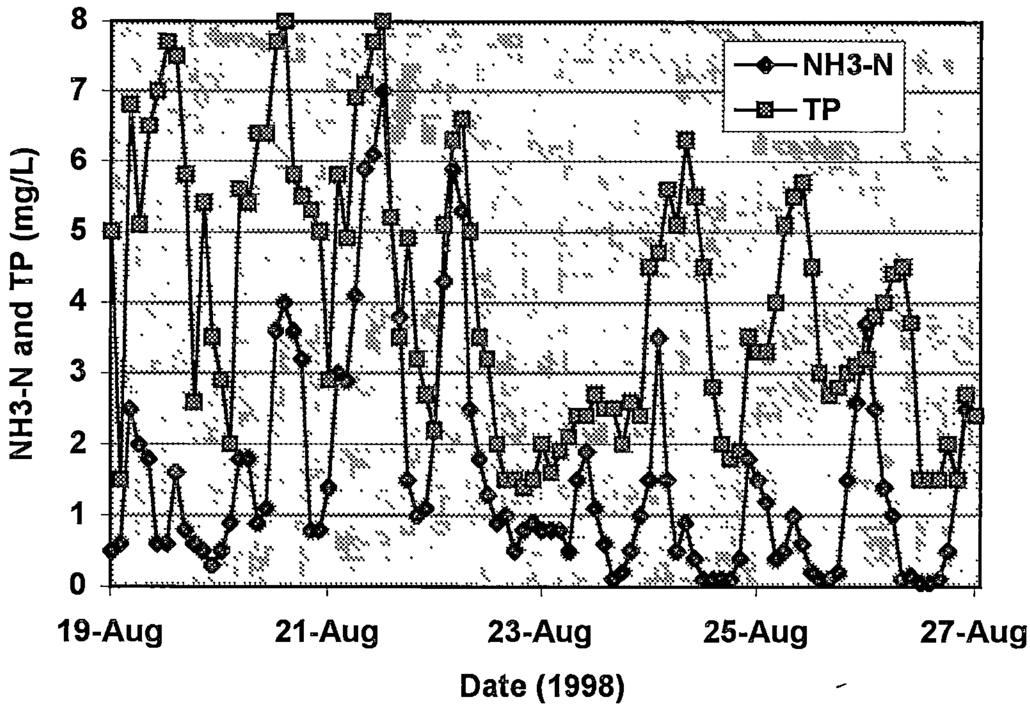

Figure 6.13 Change in Effluent Nutrients After Diverting Digester Overflow

6.12 Chapter Summary

This chapter has summarised much of the investigative work that has been carried out during commissioning and process optimisation at the Selfs Point Plant, with particular regard to the biological nutrient removal processes The major findings in analysing each of the major processes is presented and the results discussed. Many of the findings can be described as preliminary and will require further work.

The performance of the primary sedimentation tanks, the trickling filters, the BNR system, the thickeners, prefermenter and the digesters are described and related to factors which may have impacted on their performance.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

This study has reviewed the history of wastewater from early times and its development through increasing levels of treatment. The development of the wastewater treatment in Hobart has been traced to the present day and the first full scale biological nutrient removal wastewater treatment plant in Tasmania.

The theory of wastewater treatment processes, particularly how they relate to nutrient removal was discussed in some depth and this discussion gave some idea of how each of the processes has impacts on the function of other processes

The upgraded Selfs Point Plant has demonstrated a high degree of flexibility in being able to handle shock loads and flows, which may not have been treated to the same level if the biological section of the treatment plant had been solely activated sludge. The performance of the plant has yet to reach the specification requirement for phosphorus removal in a consistent manner, but optimisation work on the prefermentation system in particular has significantly increased the capacity of the system to achieve good EBPR.

The following conclusions have been drawn from the results obtained to date from the plant operations and the review of theory and practice and innovation at other treatment plants.

1. Wastewater characteristics, especially those which include an industrial component, have a major impact on the performance of a treatment plant. For biological phosphorus removal in particular a high organic load in terms of BOD or COD may not necessarily reflect a high level of very degradable material.

2. While not confirmed by this study, a review of the literature indicates that the structure of the sewer system including steep grades, residence times and force mains can have major impacts on the quantity of VFA or other RBCOD with both generation and destruction possible. The steep grades and short residence time of Hobart sewers are considered as being unfavourable for VFA generation and could well be removing VFA.

3. The installation and commissioning of a storage tank at the milk processor, with the facility for offpeak discharge has reduced the peak organic load and significantly increased the overnight organic load received by the plant, with the transfer of about 4% of the plant's daily organic load.

4. The primary sedimentation tanks at Selfs Point have greater than expected removal efficiencies. The particular reason is not known, though several potential factors were identified in the study. The greater than expected performance resulted in one tank being taken offline due to the reduced quantity of organic material passing to the BNR system.

5. The conversion of a primary sedimentation tank to activated mode can be very simple and can provide a degree of prefermentation. At Selfs Point however activated primary operation did not increase the RBCOD enough to support good EBPR. The soluble COD was increased by an average of 20mg/L (12%) for a sludge retention time of about 2 days.

6. The use of trickling filters in a BNR plant can provide an increased degree of flexibility for unusual influent conditions and can reduce reactor volumes and aeration costs, but would require appropriate influent characteristics to ensure sufficient BOD for the BNR process. The DEPHANOX process is an alternative which could be used for low organic strength influents as the trickling filters are used in nitrifying duty only.

7. The rock media trickling filters at Selfs Point provide good nitrification and carbonaceous removal. The high nitrification rates may be assisted by the low organic load received by the trickling filters for much of the day, thus reducing competition from heterotrophs. The filters nitrification rate appears to be limited by oxygen mass transfer at periods of high organic load and by insufficient alkalinity at lower loads. The filters also perform a significant amount of denitrification, which may be due the relatively low hydraulic rates for much of the day allowing a thicker biofilm to build up that would provide anoxic conditions, especially at higher organic loads.

8. The trickling filters may be assisting in providing a very good settling sludge in the BNR system during summer, when filamentous populations are low, but this assistance can not overcome high populations of filamentous bacteria.

9. Prefermentation has been essential to achieving good EBPR at Selfs Point. The influent appears to be very low in suitable RBCOD. The combination of activated primary tank and complete-mix prefermenter has provided additional sludge retention time compared to the original design at very little cost.

10. The performance of the complete-mix prefermenter was improved by the use of dilution water, which could be used to reduce the reactor concentrations and thus improve the yield of RBCOD. The dilution water assisted in permitting the extended hours of operation of the thickener, which is also considered to have had a major impact on EBPR performance.

11. The VFA profile in the complete-mix prefermenter was unusual when compared to that reported in the literature. Greater proportions of the slightly longer chained fatty acids (butyric and valeric) may be related to the composition of the milk processing facility's effluent.

12. The early biological phosphorus removal performance of the upgraded Selfs Point Plant was poor and this was almost certainly due to inadequate RBCOD as evidenced by the improvement in EBPR during the acetate dosing trial.

13. The impact of changes to the RBCOD concentration to the anaerobic zone (particularly increases in RBCOD) appears to have about a 2-4 day lag following the changed concentration.

14. The low alkalinity of the influent to Selfs Point may occasionally be resulting in impaired nitrification rates due to pH depression in the BNR system.

15. Some results appear to indicate that phosphorus uptake is poorer at a dissolved oxygen level of 1mg/L when compared to that at a dissolved oxygen of 2mg/L, but that the uptake does not improve significantly at a DO of 4mg/L compared to 2mg/L..

16. Both the DSVI and stirred SVI appear to be suited to the MLSS concentrations typically used at Selfs Point, particularly during winter. When the sludge characteristics are very good however the stirred test becomes less sensitive to sludge property changes and the unstirred SVI was found to have some utility under these conditions.

17. Variations of the activated sludge SVI at Selfs Point has a strong relationship to the amount of *Microthrix parvicella* in the sludge. Both vary significantly with the seasons, with high SVI and high numbers of *M. parvicella* during winter, when the mixed liquor temperatures are less than 15°C.

18. *M. parvicella* abundance has also been related to deeper sludge blankets in secondary clarifiers, and as the MLSS is increased in winter to provide adequate

nitrification the sludge blanket also increases in depth and may be contributing to the proliferation of the filamentous bacteria at this time.

19. The addition of dilution water appeared to significantly improve the performance of the prefermenter rotary drum thickener.

20. A liquid polymer was successful at tripling the throughput of a rotary drum thickener, compared to the best dry polymer previously used.

21. Approximately 30% of the phosphorus entering the anaerobic digesters was solubilised and it is considered that the low dissolved solids content of Hobart potable water and in particular low levels of magnesium and calcium contribute to this level of release, which is higher than that occurring in anaerobic digesters in Denmark operating at similar duties.

22. Addition of alum to digested sludge prior to a belt filter press can be effective in reducing ortho-P concentration in the press filtrate from over 300mg/L to less than 10mg/L at about 1:1 molar ratio.

This study has been very wide ranging and there are numerous further areas for research in such areas as better defining the influent characteristics, including the major sources of phosphorus and whether any of these sources can be reduced, the performance of the prefermentation systems, the operation of the digester and factors affecting the quantum of the release of phosphorus from the digesters.

There are however several areas which would appear to require research to improve the operability of the plant.

1. Research to better determine the major factors causing of the high populations of *Microthrix parvicella* should be undertaken so that the poor sludge settling characteristics of winter can be avoided.

2. Review and assess methods to improve the control of denitrification prior to the anaerobic zone, as the present variability of sludge settling properties makes accurate control of denitrification in the secondary clarifier difficult. This review should include giving consideration to modifying the filtrate return line such that the filtrate is added after the two small (25m^3 each) anaerobic tanks to provide some anoxic volume prior to the major portion of the anaerobic zone.

3. The timing of the return of strong sidestreams is an important consideration to ensure that these loads are applied when capacity is available and consideration should be given to storage of sidestreams so that thickening and dewatering operations can occur at times appropriate for those activities. Treatment of the digester filtrate by passing it over the trickling filters prior to it entering the BNR process should be considered to reduce the ammonia load on the activated sludge system.

4. Once the automatic actuated valve is installed for splitting the flow between the trickling filters and the BNR system experimental trials should be set up to investigate the optimum use of this system.

5. The prefermentation system should be reviewed to ensure there is sufficient redundancy in the system to cover breakdowns and other unplanned outages due to the critical nature of the process in achieving phosphorus removal.

6. The option of returning thickened prefermented sludge back to the prefermenter for part of the day and increasing the use of dilution water could be investigated to further reduce the HRT, while maintaining the SRT, with the aim of increasing VFA yield.

7. Further work should be performed to trial the chocolate factory DAF waste to improve the VFA yield from the prefermenter.

8. The activated sludge system should be reviewed for scum traps and the design and operation should be modified to minimise scum buildup and a system installed to collect and remove any scum from the process. The objective of these actions is to reduce the amount of foam generating bacteria which may be seeding the process.

9. Undertake investigative work in the catchment to determine whether the sources of recent increases in influent phosphorus and BOD concentrations can be identified and assess options for reduction of phosphorus discharges into the catchment.

10. To reduce the likelihood of digester foaming consideration should be given to the use of thermophilic temperatures in one of the digesters, which could be used in primary mode, rather than in parallel operation as at present. This operation would also increase the stability of the resulting sludge.

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