

The Impact of Innovative Effluent Permitting Policy on Urban Wastewater System Performance

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Abstract

This thesis investigates innovative effluent point-source permitting approaches from an integrated urban wastewater system (UWWS) perspective, and demonstrates that three proposed permitting approaches based on optimal operational or control strategies of the wastewater system are effective in delivering multiple and balanced environmental benefits (water quality, GHG emissions) in a cost-efficient manner.

Traditional permitting policy and current flexible permitting practices are first reviewed, and opportunities for permitting from an integrated UWWS perspective are identified. An operational strategy-based permitting approach is first developed by a four-step permitting framework. Based on integrated UWWS modelling, operational strategies are optimised with objectives including minimisation of operational cost, variability of treatment efficiency and environmental risk, subject to compliance of environmental water quality standards. As trade-offs exist between the three objectives, the optimal solutions are screened according to the decision-makers' preference and permits are derived based on the selected solutions. The advantages of this permitting approach over the traditional regulatory method are: a) cost-effectiveness is considered in decision-making, and b) permitting based on operational strategies is more reliable in delivering desirable environmental outcomes. In the studied case, the selected operational strategies achieve over 78% lower environmental risk with at least 7% lower operational cost than the baseline scenario; in comparison, the traditional end-of-pipe limits can lead to expensive solutions with no better environmental water quality. The developed permitting framework facilitates the derivation of sustainable solutions as: a) stakeholders are involved at all points of the decision-making process, so that various impacts of the operation of the UWWS can be considered, and b) multi-objective optimisation algorithm and visual analytics tool are employed to efficiently optimise and select high performance operational solutions.

The second proposed permitting approach is based on optimal integrated real time control (RTC) strategies. Permits are developed by a three-step decision-making analysis framework similar to the first approach. An off-line model-

based predictive aeration control strategy is investigated for the case study, and further benefits (9% lower environmental risk and 0.6% less cost) are achieved by an optimal RTC strategy exploiting the dynamic assimilation capacity of the environment.

A similar permitting approach, but simpler than the first two methods, is developed to derive operational/control strategy-based permits by an integrated cost-risk analysis framework. Less comprehensive modelling and optimisation skills are needed as it couples a dynamic wastewater system model and a stochastic permitting model and uses sensitivity analysis and scenario analysis to optimise operational/control strategies, hence this approach can be a good option to develop risk-based cost-effective permits without intensive resources.

Finally, roadmaps for the implementation of the three innovative permitting approaches are discussed. Current performance-based regulations and self-monitoring schemes are used as examples to visualise the new way of permitting. The viability of the proposed methods as alternative regulation approaches are evaluated against the core competencies of modern policy-making.

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List of Abbreviations

AML	Average monthly limitation
ASM	Activated sludge model
AWL	Average weekly limitation
BMP	Best management practice
BOD	Biochemical oxygen demand
CCC	Criterion continuous concentration
CMC	Criteria maximum concentration
CRC	Carbon reduction commitment
CSO	Combined sewer overflow
CSS	Combined sewer system
CSTR	Continuous stirred-tank reactor
CV	Coefficient of variance
DO	Dissolved oxygen
DWF	Dry weather flow
EA	Evolutionary algorithm
FFT	Flow to full treatment
FIS	Fundamental intermittent standard
GA	Genetic algorithm
GHG	Greenhouse gas
GMC	Generic model control
HRT	Hydraulic retention time
IWA	International Water Association
IPPC	Integrated Pollution Prevention Control Directive
LA	Load allocation
LHS	Latin hypercube sampling
LTA	Long-term average
LTCP	Long-term control program
MDL	Maximum daily limitation
MLSS	Mixed liquor suspended solids
MOEA	Multi-objective evolutionary algorithm
MOS	Margin of safety
NSGA-II	Non-dominated sorting genetic algorithm-II
NPDES	National pollutant discharge elimination system
NPGA	Niched-Pareto genetic algorithm

OAT	One-at-a-time
PAES	Pareto archived evolution strategy
PDF	Probability density function
p.e.	Population equivalent
PFF	Pass forward flow
PID	Proportional-integral-derivative
RBMP	River basin management plan
RIA	Regulatory impact assessment
RQP	River quality planning
RTC	Real-time control
RWQM1	River water quality model No.1
SPEA	Strength Pareto evolutionary algorithm
SuDS	Sustainable urban drainage system
TBEL	Technology-based effluent limitation
TMDL	Total maximum daily load
TN	Total nitrogen
TP	Total phosphorus
TSS	Total suspended solids
UWS	Urban water system
UWWS	Urban wastewater system
UWWTD	Urban Waste Water Treatment Directive
WET	Whole effluent toxicity
WFD	Water Framework Directive
WLA	Waste load allocation
WQBEL	Water quality-based effluent limitation
WWSP	Wastewater service provider
WWTP	Wastewater treatment plant

Notation

C	Downstream river water quality (mg/L)
$C_{aeration}$	Cost for aeration (£)
C_j	Total ammonia concentration in river at time j (NH ₃ -N mg/L)
C_{limit}	The 90%ile river total ammonia standard (NH ₃ -N mg/L)
C_{pump}	Cost for pumping (£)
C_{sludge}	Cost for sludge treatment (£)
C_T	Threshold limit for the calculation of environmental risk (mg/L)
C_{ts}	Concentration of thickened waste sludge (mg/L)
E	Trade effluent flow rate (m ³ /d)
$E_{aeration}$	Total electricity consumption from aeration (KWh)
E_C	Consequence value corresponding to river water quality C (mg/L)
E_{pump}	Total electricity consumption from pumping (KWh)
G	Per capita domestic sewage flow rate (m ³ /(d·capita))
I_{DWF}	Dry weather infiltration rate (m ³ /d)
I_{MAX}	Maximum infiltration rate over a complete year (m ³ /d)
P	Population in a catchment
P_j	Probability of occurrence of C_j exceeding C_{limit}
P_r	Probability of downstream river water quality being the value C
STD_{AMM}	Standard deviation of total ammonia concentration in effluent discharge (NH ₃ -N mg/L)
V_{ts}	Total volume of thickened waste sludge (m ³)

1 Introduction

1.1 Background

A key requirement of any environmental protection policy is to establish a well-designed, operated and policed system of controlling (water, gas or solid) waste emissions to protect the environment. This typically consists of a permitting policy (also known as “consents”, “licences” and/or “authorisation”). Under an environmental permitting regulation, activities which may cause pollution by using, treating, disposing or storing waste should meet certain requirements to be environmentally safe. The operation of urban wastewater systems (i.e. sewer systems and wastewater treatment plants) is routinely regulated during the collection, treatment and disposal of urban wastewater. Strict quality and/or quantity limits are often set on the effluent from treatment processes based on treatment technology and estimation of the impact to the environment (Environment Agency, 2011a; U.S. Environmental Protection Agency, 2010). Despite the progress achieved so far by the policies in maintaining and improving environmental quality, effectiveness of the traditional regulation paradigm is being challenged by increasingly complex environmental issues, ever growing public expectations and the need for cost-effective approaches as illustrated below.

Challenge 1: Increasingly stringent environmental water quality standards

As protection of the aquatic environment has become highly valued and understood, environmental water quality standards have become more comprehensive and stringent. For example, the EU Water Framework Directive (WFD) (European Parliament and Council of the European Union, 2000) was introduced in 2000 to establish a holistic legislative framework consolidating relevant environmental water quality standards and set an overarching aim of “good status” required for all water bodies within member states by 2015. By “good status”, as specified in the WFD, it means both “good ecological status” and “good chemical status”. Each component status needs to be graded according to the performance of relevant quality elements, and the overall status is determined by the “one out, all out” principle (i.e. the final status is determined by the poorer of the ecological or chemical status). The classification system is illustrated in Figure 1.1.

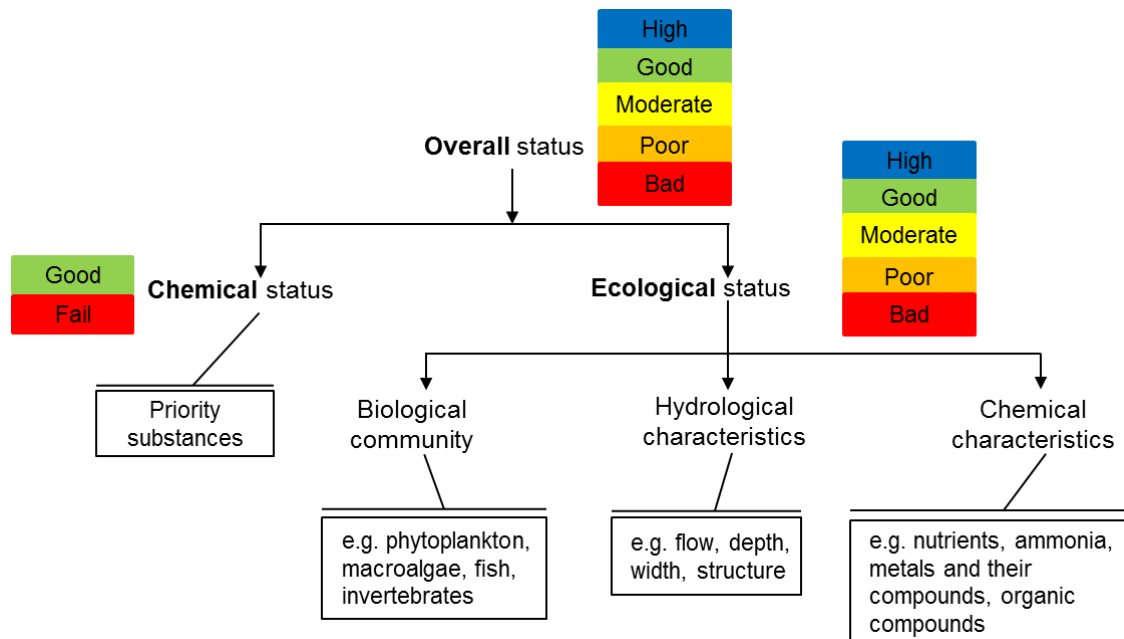


Figure 1.1 Classification system of surface water quality by the WFD

To achieve the goals set by the WFD, member states are obliged to transpose and implement the daughter and consolidated Directives of the WFD (Figure 1.2), such as the Groundwater Directive (European Parliament and Council of the European Union, 2006a), Environmental Quality Standards (priority substances) (European Parliament and Council of the European Union, 2008), Drinking Water Abstraction Directive (Council of the European Communities, 1975), Freshwater Fish Directive (European Parliament and Council of the European Union, 2006b), Shellfish Directive (European Parliament and Council of the European Union, 2006c) and Dangerous Substances Directive (European Parliament and Council of the European Union, 2006d). Beside, other environmental quality-based Directives need also to be complied with, such as the Bathing Water Directive (European Parliament and Council of the European Union, 2006e), Birds Directive (European Parliament and Council of the European Union, 2009) and Habitats Directive (Council of the European Communities, 1992). To deliver the environmental water quality standards, emission-based Directives are set to regulate and control wastewater emissions, such as the Urban Waste Water Treatment Directive (UWWTD) (Council of the European Communities, 1991a) for urban wastewater discharges, Integrated Pollution Prevention Control (IPPC) Directive (Council of the European Union, 1996) for industrial discharges and Nitrates Directive (Council of the European Communities, 1991b) for runoffs from agricultural lands.

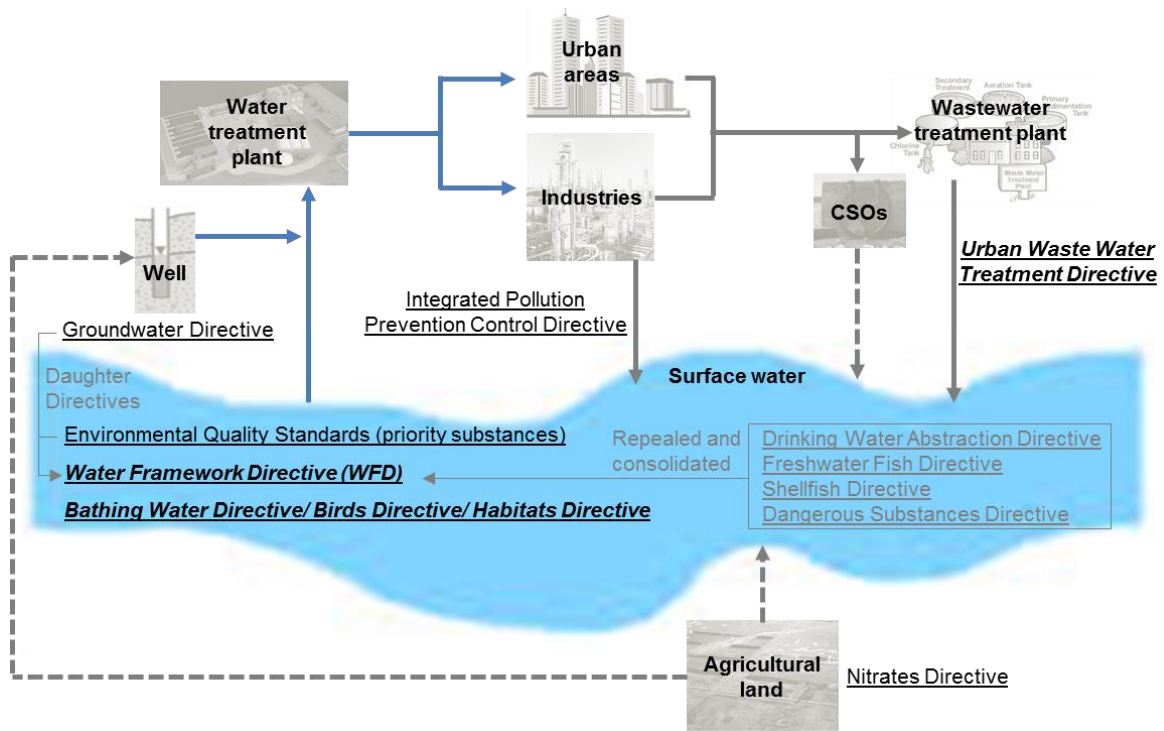


Figure 1.2 Key environmental quality-based and emission-based European Directives

According to a recent report (European Environment Agency, 2012), of the overall 127,000 surface water bodies investigated across Europe by 2012, more than half of them had not reached the good ecological status or potential (a term used for highly modified or artificial waters) required by the WFD and results are poorer for rivers and transitional waters than lakes and coastal waters as shown in Figure 1.3. In contrast to the ecological classification system, the monitoring network and assessment methods for chemical status remained to be fully developed, as more than 40% of the surface water bodies were reported as having “unknown chemical status” (Figure 1.4). Point source pollution from UWWs was identified as a major pressure affecting surface water body status, among others such as industrial wastewater discharges, runoffs from agricultural lands and hydro-morphological pressures (European Environment Agency, 2012). Though urban wastewater treatment has been greatly improved over past decades (European Parliament and Council of the European Communities, 2007), the UWWTD remains to be fully implemented (Figure 1.5).

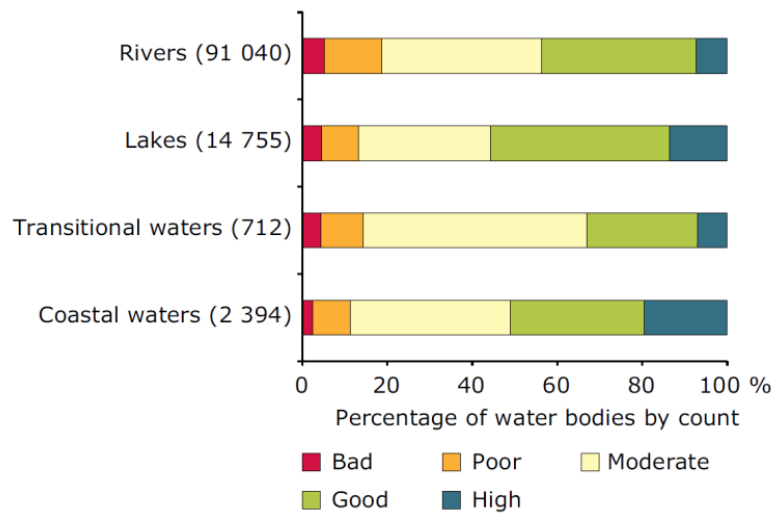


Figure 1.3 Distribution of ecological status or potential of classified surface waters in Europe (European Environment Agency, 2012)

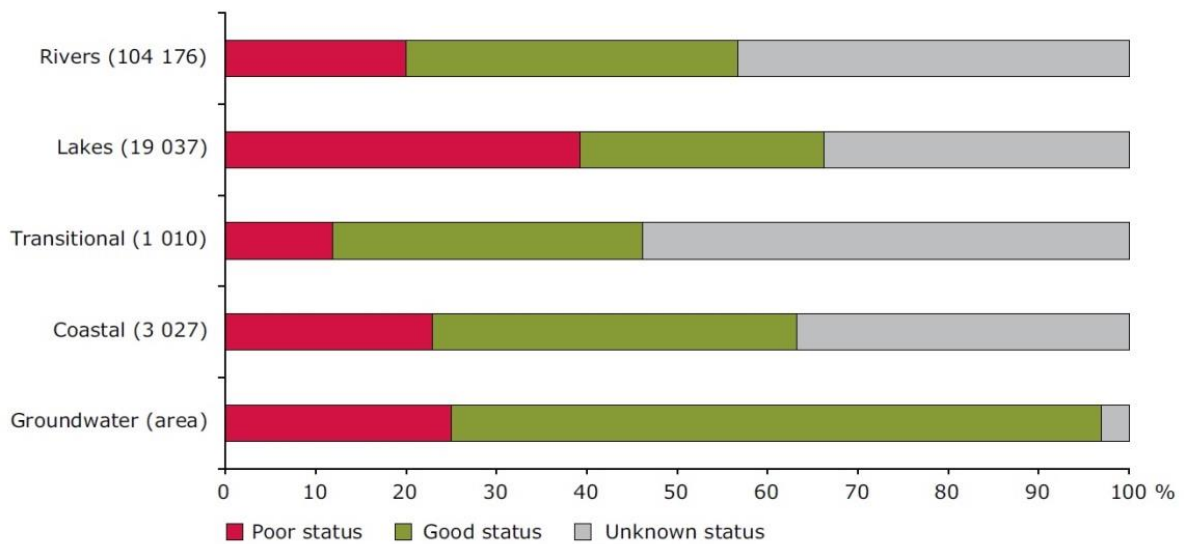
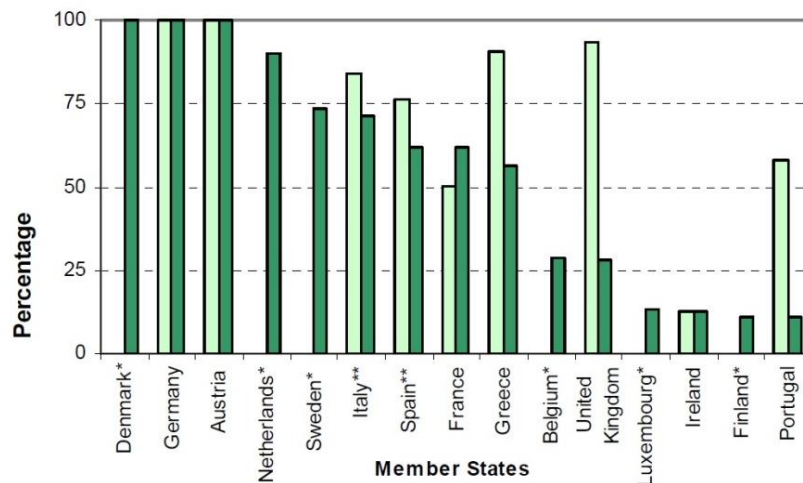


Figure 1.4 Distribution of chemical status or potential of classified surface waters in Europe (European Environment Agency, 2012)

**Legend:**

* Member State has designated its all territory as sensitive area or applies more stringent treatment in all agglomerations

- % of load [p.e.] in conformity in normal areas
- % of load [p.e.] in conformity in sensitive areas

** Information for Italy and Spain presented here does not reflect the situation on 329 agglomerations in Italy and 343 agglomerations in Spain having the total load of about 56 million p.e., which was not reported by the deadline according to the article 15(4) for this report.

status as at 01/01/2003

Figure 1.5 Compliance status with the UWWTD by member states of the EU (European Parliament and Council of the European Communities, 2007)

To deliver the environmental water quality-based and emission-based legislation, permits for point source wastewater discharges, especially effluent from wastewater treatment plants (WWTPs), have become more onerous and more costly. This is particularly challenging for the wastewater industry, as compared to most industrial sectors, inflow to the treatment process is huge in volume, complex in composition, highly dynamic in water quality and flow rate, and moreover – there is no returning of the wastewater flow to its suppliers (Olsson and Newell, 1999)! The UK water industry expects to invest £27 billion (\$46 billion) to install additional treatment capacity between 2010 and 2030 (Severn Trent Water Limited, 2013).

Challenge 2: Carbon reduction commitment

Besides the issue of environmental water quality deterioration, UWWs can also contribute to climate change by Greenhouse Gas (GHG) emissions. Urban wastewater treatment results in direct emission of GHGs carbon dioxide (CO₂), methane (CH₄) and nitrous oxide (N₂O), and indirect emission from energy consumption, chemical manufacture and sludge disposal, etc. (Bani Shahabadi et al., 2009; Sweetapple et al., 2014a). The wastewater industry is identified as one of the major contributors of GHG emissions (Harfoot et al., 2009; Sturchio

et al., 2010). According to the figures for the US in 2005, the wastewater sector is responsible for about 1% indirect GHG emissions resulted from energy use and 0.37% (the figure is 1.4% globally according to U.S. Environmental Protection Agency (2012)) direct non-CO₂ GHG emissions (U.S. Environmental Protection Agency, 2015, 2006)

With global warming being widely understood, many countries are committed to reduction of GHG emissions (United Nations, 1998). For example, a target was set in the UK (also the EU) to cut GHG emissions by 80% by 2050 with respect to a 1900 baseline (European Commission, 2011a; Parliament of the UK, 2008). To achieve the carbon reduction target, the Carbon Reduction Commitment (CRC) Scheme (Parliament of the UK, 2010) was established in the UK targeting carbon emissions from large non-energy intensive businesses and public sectors (defined as organisations whose mandatory half-hourly metered electricity use exceeds 6,000 MWh per year). As wastewater service providers (WWSPs) fall into the category of large non-energy intensive businesses, they are required to contribute to the reduction in GHG emissions (Harfoot et al., 2009). This, however, places the wastewater industry in somewhat of an environmental dilemma as enhanced wastewater treatment often increases GHG emissions (Flores-Alsina et al., 2011; Sweetapple et al., 2014a). It is estimated that the increased wastewater treatment under WFD is likely to increase CO₂ emissions by over 110,000 tonnes per year from operational energy use and emissions associated with the additional processes required (Georges et al., 2009).

Challenge 3: Limited control on combined sewer overflows

Besides effluent discharges from WWTPs, UWWs may also cause water pollution through intermittent wastewater discharges (e.g. tank or sewer overflows) under wet weather conditions (Butler and Davies, 2011; Hvitved-Jacobsen, 1982). In particular, overflows from combined sewer systems (i.e. CSOs) are a major concern and have been a focus of investigation and research. Combined sewer systems (CSSs) are most commonly found in old systems, e.g. some European cities and older east coast cities in the US (Butler and Davies, 2011), which collect and transport rainwater runoff, domestic sewage and certain industrial wastewater in the same pipes to WWTPs. An

advantage of CSSs over separate sewer systems is the treatment of stormwater (which may be polluted) in light rain without overflowing to the receiving water body. However, during periods of heavy rainfall when the volume of sewage exceeds the capacity of the UWWs, untreated wastewater is allowed to spill with stormwater via CSOs to nearby watercourses (Environment Agency, 2011a). Structures such as screenings and storage tanks can be built to provide preliminary physical treatment (Environment Agency, 2011a), yet the efficiency of the treatment is limited and pollutants (in particular soluble substances) could still be of high concentration and pose detrimental impacts to the environment. For instance, CSOs can affect human health by high loads of pathogens, and endanger aquatic life by high concentration of toxic unionised ammonia or Biochemical Oxygen Demand (BOD) leading to dissolved oxygen depletion (Blanksby, 2002; Hvitved-Jacobsen, 1982; Ruffier et al., 1981). It was estimated that some 8,000 of approximately 25,000 CSOs in England and Wales were causing water problems at the beginning of the 1990s (Clifforde et al., 2006) and many remain underperforming even today (Nardell, 2012).

Despite the recognition of potential environmental risks, CSOs are regulated by simplistic measures such as spill frequency, duration or volume (Blanksby, 2002; Environment Agency, 2011a; U.S. Environmental Protection Agency, 2010). These surrogate indicators are incapable of representing the impact of the overflows as research has revealed the poor correlation between reducing CSO spill frequency or volume and improving receiving water quality (Lau et al., 2002). Indeed, it is difficult to assess the performance of CSOs due to the technical and financial viability required to measure flows in sewers and rivers and collect representative samples (Blanksby, 2002). Hence, permitting on CSOs in the UK is complemented by prescribing risk averse design (e.g. screenings, storage tanks) and operational strategies (real-time control schemes). However, determination of the prescriptive measures is usually made by empirical rules or models with limited representation of the interactions between CSOs and WWTP effluent. This may lead to under-optimal solutions as the overall impact to the downstream river are not fully appraised (Lau et al., 2002). In view of the cost implication of improving CSOs, e.g. £2.9 billion (\$4.9 billion) estimated for the UK (Clifforde et al., 2006) and £26.5 billion (\$45 billion)

for the US (U.S. Environmental Protection Agency, 1999), there is a need for more cost-effective CSOs control measures.

Challenge 4: Adaptation to population growth, urbanisation and climate change

The world population has been constantly growing. It reached 3 billion in 1960 and took about 13-14 more years for each additional billion people thereafter (National Research Council, 2012). In 2010, over half of the 7 billion world population lived in urban settlements. Projections showed that by 2050, the urban population would be 70% of the 9 billion people estimated due to economic development and urbanisation (OECD and CDRF, 2010). As a result of the population growth and urbanisation, WWSPs need to cope with a rising amount of wastewater produced and discharged to the UWWSSs. Moreover, the pattern of the wastewater flow rate and pollutant loading is becoming more uncertain due to changing land uses and water consumption patterns (Astarai-Imani et al., 2012). Climate change, by disrupting usual weather patterns and increasing the chance of extreme weather events, adds more pressures by raising the uncertainty in the quantity and quality of wastewater transported to the WWTP and overflowed to the environment (Butler et al., 2007; Fortier and Mailhot, 2015; Semadeni-Davies et al., 2008). In the meanwhile, the environmental capacity may be reduced due to the combined effects of urbanisation and climate change (Whitehead et al., 2009), thus pose stricter requirements on the performance of WWSPs. The cost implications to accommodate the wastewater services to the changing environment are significant. For example, £72.4 billion to £148.2 billion (\$123 billion to \$252 billion) investment was estimated for wastewater services (e.g. infrastructure, operations and maintenance) in the US to adapt to climate change (NACWA and AMWA, 2009).

Following the traditional regulatory approach, end-of-pipe limits on WWTP effluent discharges and CSO spill frequency are likely to be tightened to meet the increasingly higher environmental water quality demand. However, it is difficult to comply with a stricter wastewater discharge permit without raising GHG emissions (or cost) by the intuitive strategy of enlarging the capacity of the existing treatment processes. Hence, innovative wastewater management strategies based on technological innovation should be explored to tackle the

multiple (or even conflicting) environmental challenges in a sustainable way (Kemp, 1994). Six examples of innovative strategies are presented as follows.

a) Sustainable urban design: This is a holistic and strategic solution which integrates environmental management into urban planning and development from the earliest stages to maximise the opportunities for sustainable development (Wong, 2006). An example is incorporating sustainable urban drainage systems (SuDS) (Casal-Campos et al., 2015; Wang et al., 2013) into urban design to minimise stormwater discharged to UWWs and reduce flood risk and water pollution via CSO discharges (Communities and Local Government, 2009; Van Berkel et al., 2009). Support from the local/federal governments is needed for the implementation of this strategy.

b) Pollution prevention: By reducing waste generated at source and avoiding the cost and efforts for wastewater treatment, this is one of the most desirable environmental management strategies (U.S. Environmental Protection Agency, 1993). This approach, however, needs cooperation from all sectors, such as energy (e.g. increasing energy efficiency) (European Commission, 2011b), transport (e.g. using renewable energy sources) (European Biogas Association, 2011), agriculture (e.g. cultivating crop strains with natural resistance to pests rather than using pesticide) (U.S. Environmental Protection Agency, 1993), industry (e.g. leak detection and repair) (Jones, 1996) and domestic activities (e.g. buying non-hazardous products and reusing them).

c) Resource recovery and recycling: After waste has been generated, the impact can be greatly reduced by resource recovery and recycling. For example, grey water (i.e. urban wastewater from baths, showers, hand basins, washing machines, dishwashers and kitchen sinks), which constitutes 50-80% of the total household wastewater, has low levels of contaminating pathogens and nitrogen and thus can be recycled and reused on-site rather than discharging to UWWs (Li et al., 2009; Nolde, 2005). Even after being conveyed to WWTPs, urban wastewater can still be treated as a potential resource (water, energy, plant fertilizing nutrients) rather than waste by water, biogas and nutrients recovery/reuse technologies in the WWTPs (Guest et al., 2009; Jin et al., 2015; Mccarty et al., 2011).

d) Innovative wastewater treatment technologies: To adapt to the changing technological, economic and regulatory climates, the end-of-pipe wastewater treatment technologies are evolving to meet the demands of the environment. A range of innovative technologies (e.g. ANAMMOX) are emerging that could produce satisfactory effluent quality with less energy requirement (Castro-Barros et al., 2015; Strous et al., 1997). U.S. Environmental Protection Agency (2013) provides a comprehensive review on the emerging technologies for wastewater treatment and in-plant wet weather management.

e) Efficient operation and control of UWWs: This strategy makes best use of what's there already by adjusting the operation or control in a wastewater system to the environmental needs. Based on modelling of an integrated UWWs (i.e. sewer system, WWTP and the receiving water), operation in the sewer and WWTP can be optimised in a coordinated manner to maximise environmental benefits without entailing excessive cost. For example, research showed that significant improvement in river water quality can be achieved with no more energy cost by optimising an integrated operational strategy of the UWWs (Fu et al., 2008; Schütze et al., 2002). Further savings are achievable by implementing real-time control (RTC) strategies to exploit the dynamic capacity of the environment (e.g. high dilution capacity of the river) without detrimental environmental impacts (Schütze et al., 2002).

f) Safe wastewater disposal: As the last and least desirable resort, wastewater from UWWs can be discharged to a location or at a time that causes least environmental impacts (U.S. Environmental Protection Agency, 1993). For instance, CSOs discharged to protected water bodies, such as for bathing or fishing purposes, could be diverted to less sensitive coastal waters (James, 1992).

The six strategies cover the whole life cycle of wastewater from its generation, reuse/recycling, treatment and disposal (Figure 1.6). An increasing level of changes to existing systems may be needed in the order of strategies from f) to a), yet the potential environmental benefits may also increase by moving from end-of-pipe strategies to source control solutions. However, the more sustainable strategies may not necessarily be appealing under the traditional permitting paradigm due to the separate regulation of CSO discharges and

WWTP effluent, and the fragmented control of water pollution and GHG emissions. Indeed, uncoordinated institutional frameworks and other socio-institutional factors are identified as major barriers for sustainable urban water management rather than technological reasons (Brown and Farrelly, 2009). Thus to encourage innovation and technology adoption, a holistic and flexible permitting approach is in need. As strategy e) can be built on existing systems with few changes, it is investigated in this work for the exploration of innovative permitting policy.

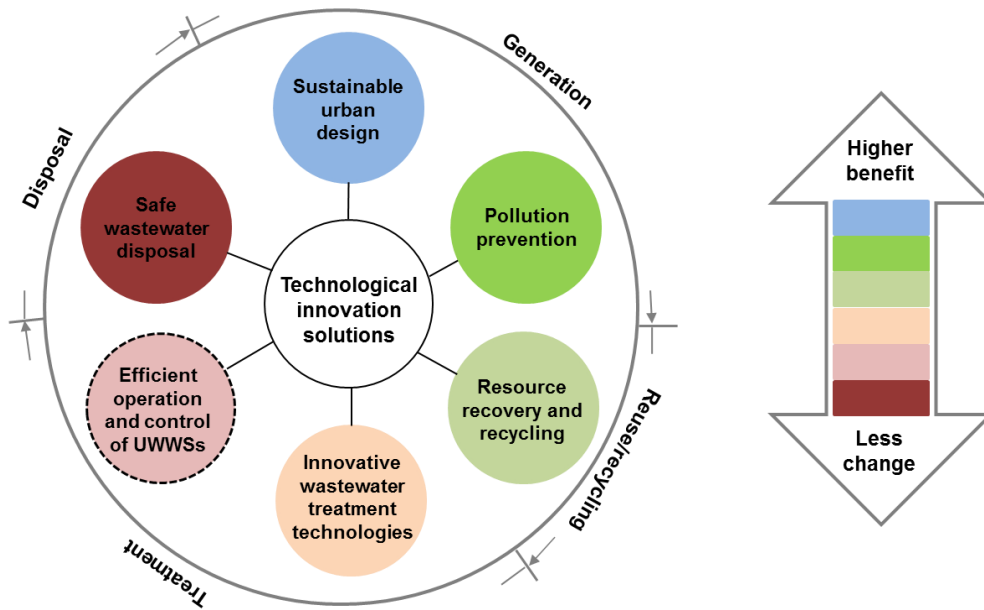


Figure 1.6 Hierarchy of innovative technological strategies for cost-effective urban wastewater management

1.2 Project Context

This PhD work is funded by the EU SANITAS project (EU FP7 Marie Curie Initial Training Network), an objective of which is to provide scientific inputs related to urban water systems (UWSs) to ensure that policy is framed within the context of what is technically possible and to ensure the future policy frameworks enable the uptake and application of European innovation. There are 14 other individual projects covering a range of topics, such as innovative treatment technologies for water/biogas reuse and nitrogen/micropollutant removal, integrated modelling and control of UWWs to reduce GHG emissions, nitrate production and micropollutant discharges, and multi-criteria decision-making analysis for sustainable design and management of UWSs. This work complements the other individual projects by exploring unconventional

permitting policy options to promote the uptake and application of the innovative wastewater treatment and management technologies.

1.3 Aim and Objectives

The aim of the work is to explore the advantages and disadvantages of innovative effluent point-source permitting policy and practice from an integrated UWWS perspective.

To achieve this aim, seven objectives are identified:

- 1) Review policy on permitting regulations, catchment management and environmental water quality standards;
- 2) Review literature on integrated modelling and real-time control of UWWSs and multi-objective optimisation;
- 3) Build and modify a model of an integrated UWWS for long-term evaluation of integrated operation/control strategies;
- 4) Develop an operational strategy-based permitting approach by integrated modelling and multi-objective optimisation;
- 5) Establish a real time control-based permitting approach to maximise urban wastewater system performance in a reliable, energy and environmentally efficient manner;
- 6) Develop a risk-based cost-effective permitting approach based on the current permitting model River Quality Planning (RQP) as practised in England and Wales; and
- 7) Investigate the advantages and disadvantages of the three proposed forms of innovative permitting and seek out the pathways for the implementation.

1.4 Thesis Structure

The thesis contains eight chapters corresponding with the achievement of the objectives. They are:

Chapter 1 - Introduction

The rationale of the research is presented by delineating four challenges faced by the traditional effluent discharge permitting policy and identification of the opportunities of addressing the challenges by efficient operation and control of integrated UWWs. The SANITAS project is briefly introduced and the role of the research in achieving the aim of SANITAS is explained. The aims and objectives of the research are identified. The originality and contribution to knowledge provided by this work are also highlighted.

Chapter 2 - Policy Review: Permitting Approaches and Policy Landscape

This chapter provides the policy background of the research. The wide policy landscape is outlined to illustrate the role of wastewater discharge permitting in a big policy picture of water pollution control at the catchment level. A comprehensive review on the traditional permitting policy of WWTP effluent discharges and CSOs are provided by using regulation examples in England and Wales and the US. Current practices of flexible permitting policy are reviewed and remaining gaps identified.

This chapter is based on and extended from the following project deliverable (Meng, 2013):

Meng, F., 2013. Literature Review on Catchment-Based Consenting (CBC), Real Time-Based Consenting (RTBC) and Its Application. SANITAS Project Report.

http://www.sanitas-itn.eu/wp-content/uploads/2013/10/Deliverable_2.1_Lit-Review-on-CBC-and-RTBC-and-its-application.pdf.

Chapter 3 - Literature Review: Integrated Modelling and Control of Urban Wastewater Systems and Multi-Objective Optimisation

The state-of-the-art in optimisation of operation and real-time control of integrated UWWs is reviewed in this chapter. As background knowledge, the tools and techniques (integrated UWWs modelling, RTC technology and multi-objective optimisation tools) essential for developing optimal operation and control of integrated UWWs in accordance to multiple objectives are also introduced.

This chapter is based on and extended from the following project deliverable (Meng, 2013):

Meng, F., 2013. Literature Review on Catchment-Based Consenting (CBC), Real Time-Based Consenting (RTBC) and Its Application. SANITAS Project Report.

http://www.sanitas-itn.eu/wp-content/uploads/2013/10/Deliverable_2.1_Lit-Review-on-CBC-and-RTBC-and-its-application.pdf.

Chapter 4 - Operational Strategy-Based Permitting

An innovative permitting approach based on operational strategies, rather than traditional end-of-pipe limits or CSO spills, is introduced. The permitted operational strategies are optimised and derived by a proposed four-step permitting framework (facilitated by integrated UWWs modelling and multi-objective optimisation), with stakeholder involved at all points of the decision-making process. The advantages of the proposed permitting approach over the conventional regulatory method in achieving multiple and balanced benefits are discussed.

This chapter is based on and extended from the research presented at the 13th International Conference on Urban Drainage (Meng et al., 2014):

Meng, F., Fu, G., Butler, D., 2014. Incorporating Cost-Effectiveness Analysis into Effluent Permitting through Integrated Urban Wastewater System Modelling and Multi-objective Optimisation, in: 13th International Conference on Urban Drainage. 7-12 September, 2014, Sarawak, Malaysia.

Chapter 5 - Real Time Control-Based Permitting

A similar but more advanced permitting approach than that introduced in Chapter 4 by an application of integrated RTC strategies is introduced. The further benefits achievable than the operational strategy-based permitting resulted from the exploitation of the dynamic capacity of the environment, is analysed.

Chapter 6 - Risk-Based Cost-Effective Permitting

A simpler method for operational or RTC strategy-based permitting is introduced in this chapter. No integrated UWWWS modelling or advanced optimisation technique is required, yet satisfactory results can be produced by a proposed integrated cost-risk analysis framework. Details on the permitting model are given, and uncertainty analysis of the model results is also made. The potential linkage to catchment-based permitting is also discussed.

This chapter is based on and extended from the research presented at the 9th IWA Symposium on Systems Analysis and Integrated Assessment (WATERMATEX 2015) (Meng et al., 2015):

Meng, F., Fu, G., Butler, D., 2015. A Risk-Based Approach to Wastewater Treatment Plant Cost-Effective Permitting, in: 9th IWA Symposium on Systems Analysis and Integrated Assessment (WATERMATEX 2015). 14-17 June, 2015, Gold Coast, Queensland, Australia.

Chapter 7 - Roadmaps to Proposed Innovative Permitting Approaches

The three innovative permitting approaches proposed in Chapters 4-6 are appraised and compared in terms of cost, benefit, risk and viability as modern policy. The roadmaps for the implementation are also discussed.

Chapter 8 - Conclusions and Recommendations

This chapter summarises the key research findings in previous chapters, and discusses the opportunities for future work.

1.5 Originality and Contribution to Knowledge

This thesis has:

- Demonstrated that optimising an integrated operational strategy of an UWWWS can achieve significant reduction in operational cost (potentially GHG emissions), variability of treatment efficiency and environmental risk whilst maintaining compliance of environmental standards, thus is a win-win solution to both the environment and the WWSP.

- Shown that further improvement is achievable by applying an optimal RTC strategy than an optimal fixed operation strategy in the three objectives, in particular the reduction of environmental risk. Cost savings by the investigated form of real-time aeration control are found to be insignificant if pollutant discharge load is not to be increased.
- Illustrated that whilst conventional end-of-pipe permitting method works well in controlling effluent water quality, permitting on operational or control strategies is a more effective approach in achieving multiple environmental benefits in an economic way.
- Shown that pollutant concentration limits regulated in environmental standards (e.g. 90%iles and 99%iles) are only partial representations of environmental impacts of wastewater discharges. Other indicators, such as pollutant discharge load and environmental risk proposed in this study could be employed as a complement.
- Developed innovative decision-making analysis frameworks for the three proposed permitting approaches which engage stakeholders at all points of the decision-making process, facilitate identification and selection of high performing operational/control strategies efficiently and derive permits based on the optimal solutions selected.
- Highlighted the importance of sampling frequency and timing on the permitting results. Thus more detailed sampling and representative sampling both in and out of working hours need to be taken.

2 Policy Review: Permitting Approaches and Policy Landscape

To explore innovative effluent point-source permitting approaches from an integrated UWWS perspective, it is necessary to first understand the traditional permitting policy and study the current progress towards flexible permitting. A brief review on catchment management policy facilitates better understanding by providing a wider policy context and showing how the regulation of urban wastewater discharges is coordinated with other water pollution control measures in a catchment.

In this chapter, the catchment management policy is first introduced, followed by a review on the traditional permitting approaches through two examples of comprehensive sophisticated permitting methods practised in England and Wales and the US. Finally, some current practices of flexible permitting policies are reviewed, and opportunities for innovative approaches from an urban wastewater system perspective are briefly discussed.

2.1 Catchment Management Policy

Catchment management is a process bringing the various parties and interests in a catchment together through regional land and water management plans to achieve whole catchment improvements (EU LIFE Environment Programme, 2009). As a systematic environmental planning framework, it requires consideration of complex relationships between natural and physical resources and social, cultural, economic and political matters (Feeney et al., 2010). This integrated, adaptive, coordinated and participatory approach is a product of technological, legislative and institutional progress in water pollution control, as can be indicated from a brief history of water pollution management in the UK presented in section 2.1.1. It has now been applied in many countries, such as the EU member states, the US, Australia and South Africa (Ashton, 1999; Bellamy et al., 2002; Defra, 2013a; National Research Council, 2001). Though similarities exist, the implementation of the policy differs in details. The description of the policy in section 2.1.2 is based on the catchment management practices in the UK and the US.

2.1.1 A Brief History of Water Pollution Management in the UK

Figure 2.1 shows a timeline of regulatory and technological milestones for surface water protection in the UK. Although the report of the Health of Towns Commission raised the first bugle call in 1844 of the great campaign for public health, sewage could not be efficiently treated until the invention of the activated sludge treatment process in 1914. For instance, sewage irrigation (or farming), which was a common practice from 1840s to 1870s, had low treatment intensity of about $0.03 - 0.1 \text{ m}^3/(\text{m}^2 \cdot \text{day})$ (Kinnicutt et al., 1919). In comparison, the activated sludge process is more efficient, and a typical value of $2.8 \text{ m}^3/(\text{m}^2 \cdot \text{day})$ was reported for a treatment plant in Norwich/England serving a population of about 150,000 (Schütze et al., 2002). Over the last century, technologies for urban wastewater treatment have flourished and matured. In addition to a number of variations of the activated sludge treatment process, treatment technologies also include biofilm treatment (e.g. trickling filters and rotating biological contactors), chemical treatment (e.g. chemical precipitation, coagulation, oxidation, ion exchange and ozone disinfection), membrane filtration and adsorption technologies (Tchobanoglous et al., 2004).

With the technological development and deeper understanding of the environmental problems, the focus of water environment protection has been shifted from public health, such as safe drinking water, food (through ingestion of seafood) and recreation (e.g. bathing, rowing), to ecological integrity for sustainable development. As a result, the scope of pollution control has expanded from domestic sewage discharges, to industrial wastewater discharges, and further to other pollution sources in the catchments, such as pollution from agricultural lands, urban areas and navigation. The evolvement of water pollution governance is reflected in the legislation, regulation and guidance set to enforce the pollution control. For example, the fifth and eighth reports by the UK Royal Commission on Sewage Disposal in 1908 was an early attempt to set up water quality standards on urban wastewater effluent discharges, which published the well-known 20/30 standard (i.e. 20 mg/L BOD and 30 mg/L TSS). The Public Health (Drainage of Trade Premises) Act 1937 set constraints (e.g. composition, volume and flow rate) on industrial wastewater discharges to sewer systems and watercourses. After the UK joined the European Commission in 1973, the country was subject to a variety of EU

policies, such as the IPPC Directive (Council of the European Union, 1996) and the Nitrates Directive (Council of the European Communities, 1991b).

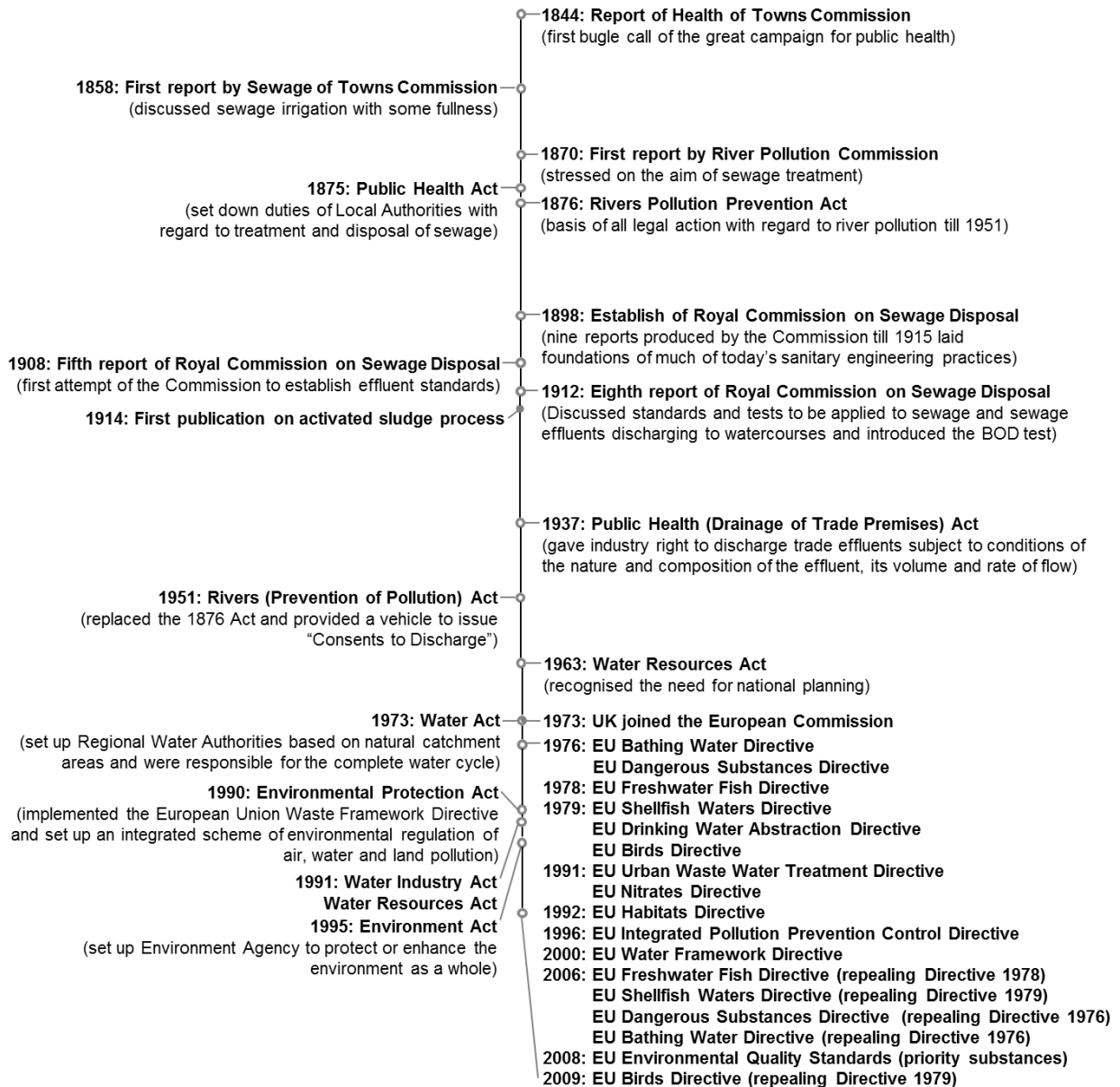


Figure 2.1 Timeline of milestones for surface water protection in the UK

Despite the considerable progress achieved in tackling individual issues, an integrated and coherent regulation approach was recognised to be necessary for more cost-effective management as water is interconnected within the same catchment. Indeed, the catchment-based management approach not only promotes the delivery of a better quality water environment, but also encourages collaborative effort to support transparent decision-making and long-term self-sustaining funding arrangement (Defra, 2013a). The catchment management scheme, named 'River Basin Management Plan' (RBMP), is now

an enforced practice across EU under the legislative framework WFD. In line with the legislative and regulatory requirements, coordinated and coherent institutional organisations should also be set up to facilitate successful implementation of catchment management. The Regional Water Authorities founded in 1973 in the UK is a good early example (Lynk, 1993) of water governance institutions based on natural geographical and hydrological units rather than by administrative or political boundaries. Similar institutional establishments are becoming a common practice now across Europe driven by the WFD.

2.1.2 Catchment Management Practices

Though the catchment management strategies in the UK and the US vary in detail and use different terminologies (e.g. 'catchment' used in the UK while 'watershed' in the US), they follow a similar form and structure, which is presented in Figure 2.2 and summarised in seven steps as presented below.

Step I: Define different surface water uses (e.g. for drinking, bathing, shellfish life) and formulate environmental water quality standards to attain the water uses. Examples of environmental standards in the UK and the US are mentioned in section 2.2.

Step II: Designate water uses for all waterbodies in a catchment. The designation is based on a set of criteria such as the current and predisturbance conditions of a waterbody, advantages derived from a certain designated use and costs of achieving the designated use (National Research Council, 2001).

Step III: Classify waterbodies by evaluating the current water quality condition against the environmental standards. Waterbodies are classified as 'satisfactory' or 'impaired' in the US (National Research Council, 2001), and in five grades of 'high', 'good', 'moderate', 'poor' and 'bad' in the UK (European Parliament and Council of the European Union, 2000).

- If the water quality is 'satisfactory' or 'high/good', the following steps need not to be analysed, and the water quality should be maintained without deterioration;

- If the waterbody is classified as 'impaired' or 'moderate/poor/bad', continue to step IV;

Step IV: Identify pressures affecting the achievement of the environmental standards. A water body can be impaired by a single or multiple pressure(s), such as urban/agricultural/transport pollution, abstraction and other artificial flow regulation, commercial fisheries, mines and minewaters, and physical modification (Environment Agency, 2009).

Step V: Propose actions to address the pressures. Point source pollution discharges from UWWs and industries are typically controlled by a provision of appropriate treatment process regulated by permitting policy; non-point source pollution from urban runoffs can be mitigated by construction of green infrastructures such as SuDS (Defra, 2011); and pollution from agricultural runoffs is usually managed through good agricultural practices, e.g. application of fertilisers at appropriate time and in adequate doses, and soil erosion reduction measures such as hedging and ditching (Defra, 2009).

In the UK, planned actions within the same catchment are coordinated under a RBMP to achieve incremental environmental water quality improvement (e.g. from 'poor' to 'moderate', and from 'moderate' to 'good') in a cost-effective manner. A similar but more quantitative policy is implemented in the US through Total Maximum Daily Load (TMDL) programme (National Research Council, 2001). In this programme, a maximum amount of pollutant load is determined for a catchment without violating the environmental standards. The TMDL is then allocated to individual discharges in the catchment from point sources (Waste Load Allocations, WLAs), nonpoint sources (Load Allocations, LAs), background/natural sources, a reserve capacity to accommodate increased or new discharges in the future, and a margin of safety (MOS) to account for uncertainty, as expressed in Equation 2.1.

$$\text{TMDL} = \Sigma\text{WLA} + \Sigma\text{LA} + \text{MOS} + \text{reserve capacity} \quad (2.1)$$

Step VI: Review the effectiveness of proposed action programmes. Models are usually applied for the estimation of the impact and effectiveness of the planned actions. SIMCAT (Warn, 2010) is a stochastic model widely used in the UK for catchment water quality evaluations. It extends from RQP (Murdoch, 2012), a

permitting model for single point source wastewater discharges, by taking into account pollutant emissions from agricultural livestock and arable lands, highway runoffs, urban runoffs, atmosphere deposition and septic tanks in a catchment. In the US, a variety of modelling techniques can be applied depending on data accuracy of input variables. Dynamic models (e.g. continuous deterministic simulation, Monte-Carlo simulation, and lognormal probabilistic duration models) rather than steady-state ones can be employed if detailed historical monitoring data are available and less conservative solutions are sought.

Step VII: Implement catchment management strategies. Successful implementation of catchment management strategies needs careful planning, stakeholder commitment and well in-placed monitoring systems. If monitoring data shows a measure is working well, the success should be highlighted to promote good practice; otherwise, alternative actions should be identified and implemented.

Due to likely imperfect understanding of the problem at the initial stage of the programme and new issues may emerge afterwards, the catchment management procedure, in particular steps III-VII, needs to be timely reviewed which usually takes places every few years.

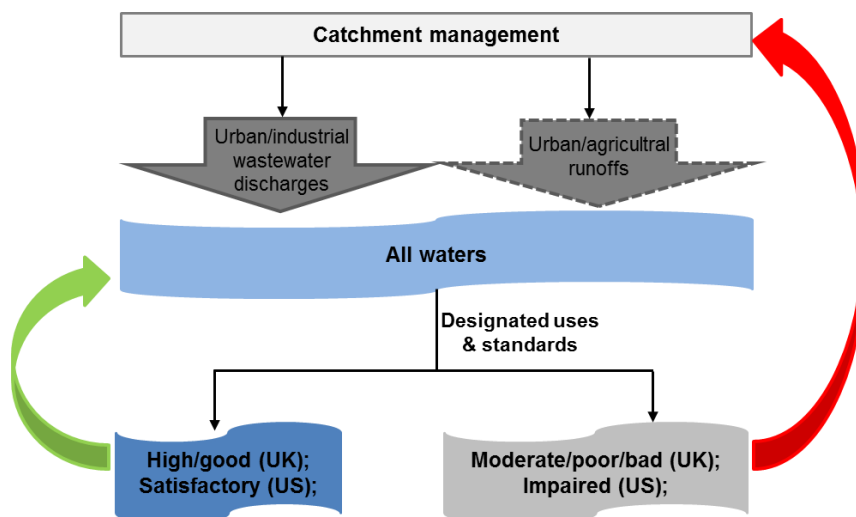


Figure 2.2 General forms of catchment management strategies

Despite the success achieved in the catchment management practices so far, some key questions or issues remain to be investigated and addressed further for more cost-effective implementation, which include:

- The definition of aquatic health and representative indicators to measure it (Logan, 2001; Norris and Thoms, 1999);
- The cause-and-effect relationships between pollutants and aquatic health (Allan, 2004; Monaghan et al., 2007; Walsh, 2000; Young et al., 1999);
- Key sources and transportation pathways of pollutants to surface waters (Heathwaite and Johnes, 1996; Hughes et al., 2005; Jordan et al., 2005; Kronvang et al., 1997; Wood et al., 2005);
- Mitigation strategies to prevent and reduce pollutant discharges to watercourses (Kampas et al., 2002; Withers and Jarvis, 1998);
- Effective stakeholder engagement (Löwgren, 2005; Rogers, 2006);
- Decision-making tools, such as catchment models, multi-objective assessment models and uncertainty analysis (Arheimer et al., 2005; Brodie et al., 2009; Holmes et al., 2005);
- Sound monitoring of wastewater discharges and the aquatic environment to track changes and facilitate cause-and-effect studies (Irvine, 2004; Parr et al., 2003); and
- Strategies to cope with challenges from climate change and urbanisation (Palmer et al., 2008).

2.2 Permitting Regulations in England and Wales

Permitting is a key catchment management strategy in the UK for the control of urban wastewater discharges. As the permitting regulations are slightly different in England and Wales, Scotland and Northern Ireland, only the policy in England and Wales is reviewed in this section.

Wastewater discharge permitting is practised under the environmental permitting regime in England and Wales, which aims to (Defra, 2013b):

- protect the environment so that environmental targets and outcomes are achieved;
- deliver certain environmental targets effectively and efficiently in a way that provides increased clarity and minimises the administrative burden on both the regulators and operators;

- encourage regulators to promote best practices in the operation of facilities; and
- fully implement European legislations.

The complete regulatory cycle of wastewater discharge permitting, similar to other forms of environmental permitting, includes preparation, determination, enforcement, compliance assessment and review (Defra, 2013b). Appendix A shows an example permit determined for effluent discharge and storm tank overflow of a WWTP in England and Wales, which include site-specific emission limits and detailed requirements on monitoring, reporting and compliance analysis. The characteristics of continuous wastewater effluent discharges from WWTPs and intermittent spills from CSOs/storm tanks are different in many aspects, thus they are permitted by different methods as summarised in Figure 2.3 and are described in detail in sections 2.2.1 and 2.2.2 respectively.

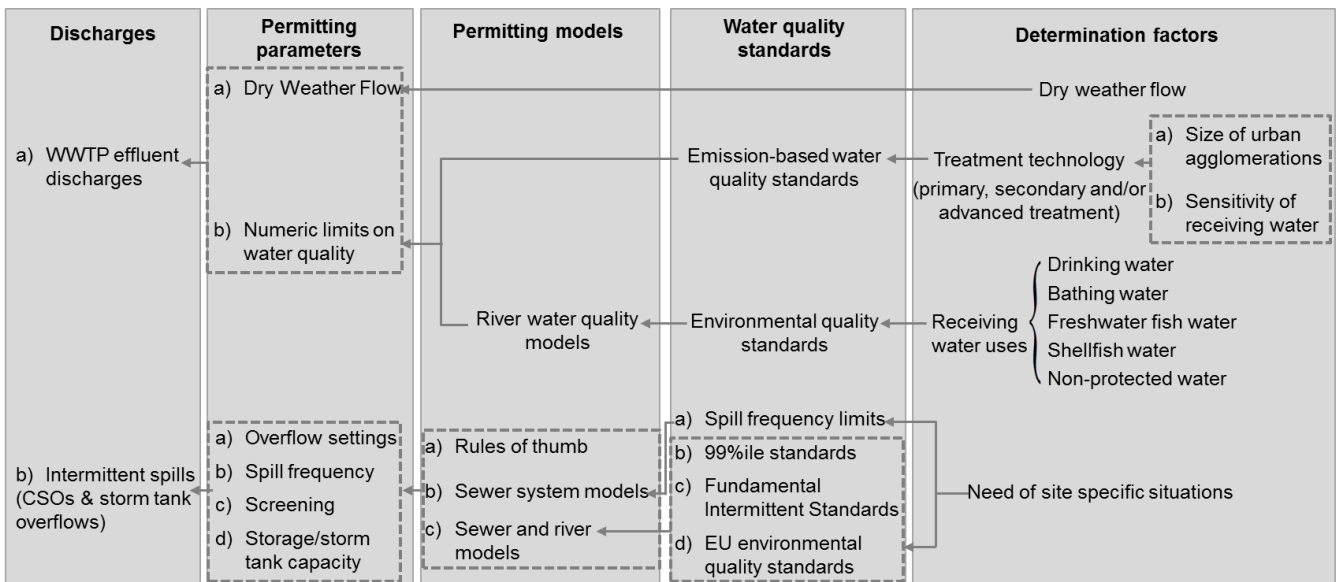


Figure 2.3 Permit derivation process for urban wastewater discharges in England and Wales

2.2.1 Permitting for WWTP Effluent Discharges in England and Wales

The quantity and quality of WWTP effluent discharges are both limited through permitting to restrict total waste loadings to the environment. Effluent flow rate is controlled by the parameter Dry Weather Flow (DWF), which is a measure of average wastewater flow received and treated by the WWTP. Higher DWF values result in more stringent water quality limits so that the downstream water

quality objectives are maintained. The DWF values need to be reported by WWSPs derived by either method as below.

a) DWF calculated from historical monitoring data:

DWF (in m³/d) is found to be well represented by the non-parametric 80%-exceeded total daily flow (known as Q₈₀ or 80%ile). Q₈₀ is calculated by ranking historical data to determine the 80% exceeded value (Environment Agency, 2011a). Thus with 365 measured records of daily flow, the 329th ranked value is taken as the Q₈₀ or DWF.

b) DWF calculated by empirical formula:

An alternative method is by a 'rule of thumb' shown in Equation 2.2. This approach is particularly applicable to new discharges where no historical flow data is available.

$$\text{DWF (m}^3\text{/d)} = PG + I_{DWF} + E \quad (2.2)$$

Where P is population in the catchment, G (m³/(d·capita)) is per capita domestic sewage flow rate, I_{DWF} (m³/d) is dry weather infiltration rate, and E (m³/d) is trade effluent flow rate. P , G and E should be based on predictions for the design horizon of the discharge. Where possible, the measured dry weather infiltration data from nearby discharges should be used to estimate the likely infiltration (Environment Agency, 2011a).

According to the UWWTD, the amount of wastewater flow and sensitivity of the receiving water jointly determine the level of treatment required (e.g. primary, secondary or advanced treatment) before wastewater can be discharged to the watercourse. A simpler and pollutant load-based indicator population equivalent (p.e., assuming 60 g BOD₅/(person·day), Council of the European Communities 1991a) is used to represent the wastewater flow scale or the size of urban agglomerations; sensitivity of the receiving water is decided by whether the waterbody is under a risk of eutrophication or is a protected area (e.g. source of drinking water abstraction). More effective treatment technologies are needed for large urban agglomerations and/or if the wastewater is discharged to sensitive receiving waters. The UWWTD (Council of the European Communities, 1991a) provides criteria for the selection of an appropriate level of treatment technology and sets effluent water quality limits for the different levels of treatment processes. Table B.1 shows the effluent quality standards for

secondary treatment processes, and Table B.2 presents the additional requirements on nutrient concentration limits if more advanced treatment processes are applied to minimise the potential for eutrophication. The numeric limits shown in Table B.2 are described in annual averages, while those in Table B.1 are 95%ile values meaning there is no compliance failure if they are met for more than 95% of the samples collected.

In addition to the emission (or technology)-based control limits, permitting for effluent discharges needs also to consider the impact of the discharges to the local environment. Assuming no control on the wastewater flow quantity and upstream watercourse (quantity and quality) conditions, effluent water quality is set at a level to ensure the waterbody can maintain or improve its current water quality status after receiving the wastewater discharges. Environmental water quality standards are formulated at EU and national levels for waterbodies with different water uses, such as protected areas for drinking water (Council of the European Communities, 1975), fish life (European Parliament and Council of the European Union, 2006b, 2006c) and bathing (European Parliament and Council of the European Union, 2006e). Waterbodies that fall out of the scope of protected areas are controlled under the WFD requirements (European Parliament and Council of the European Union, 2000). Table 2.1 summarises the requirements from some key EU Directives.

RQP (Murdoch, 2012) is the most widely used model in England and Wales for single urban wastewater discharge permitting. It is a stochastic model where flow rate and water quality of WWTP effluent discharge and upstream river flow are represented as random variables, described by probability distributions (typically lognormal, as illustrated in black curves in Figure 2.4) yielded from historical monitoring data. Monte-Carlo simulation (Fishman, 1995) is employed to draw values from the distributions and yield the downstream river water quality value by solving the mass balance equation. After simulating a sufficient amount of events, the percentile (e.g. 90%ile, 99%ile) values for downstream water quality can be estimated (assuming also lognormal distribution) from the results obtained. If the calculated downstream river water quality violates the environmental standard, the water quality probability distribution of the WWTP

Table 2.1 Summary of requirements in some key EU Directives related to urban wastewater discharges

Directive	Category	Parameter	Compliance method
Drinking Water Abstraction Directive (75/440/EEC, repealed by WFD)	<ul style="list-style-type: none"> a) A1 (simple physical treatment and disinfection); b) A2 (normal physical and chemical treatment and disinfection); and c) A3 (intensive physical and chemical treatment, extended treatment and disinfection) 	46 physical, chemical and microbiological parameters, including 7 parameters without standard values	<ul style="list-style-type: none"> a) 95%ile (parameters with mandatory requirements); and b) 90%ile (other cases)
Bathing Water Directive (2006/7/EC)	<ul style="list-style-type: none"> a) Excellent; b) Good; c) Sufficient; and d) Poor 	Intestinal enterococci and Escherichia coli	<ul style="list-style-type: none"> a) 95%ile (excellent and good quality water bodies); and b) 90%ile (sufficient water bodies)
Freshwater Fish Directive (2006/44/EC, consolidated to WFD)	<ul style="list-style-type: none"> a) Salmonid waters; and b) Cyprinid waters; 	14 physical and chemical parameters, including 3 parameters without standard values	<ul style="list-style-type: none"> a) 95%ile (pH, BOD₅, nitrites, non-ionised ammonia, total ammonium, total residual chlorine, total zinc and dissolved copper); b) Average (TSS); c) 98%ile (temperature); and d) 50%ile and maximum (DO)
Shellfish Water Directive (2006/113/EC)	--	12 (groups of) physical, chemical and microbiological parameters, including 5 (groups of) parameters without numeric limits	<ul style="list-style-type: none"> a) 95%ile (salinity, DO); b) Maximum (organohalogenated substances and metals); and c) 75%ile (other controlled parameters)

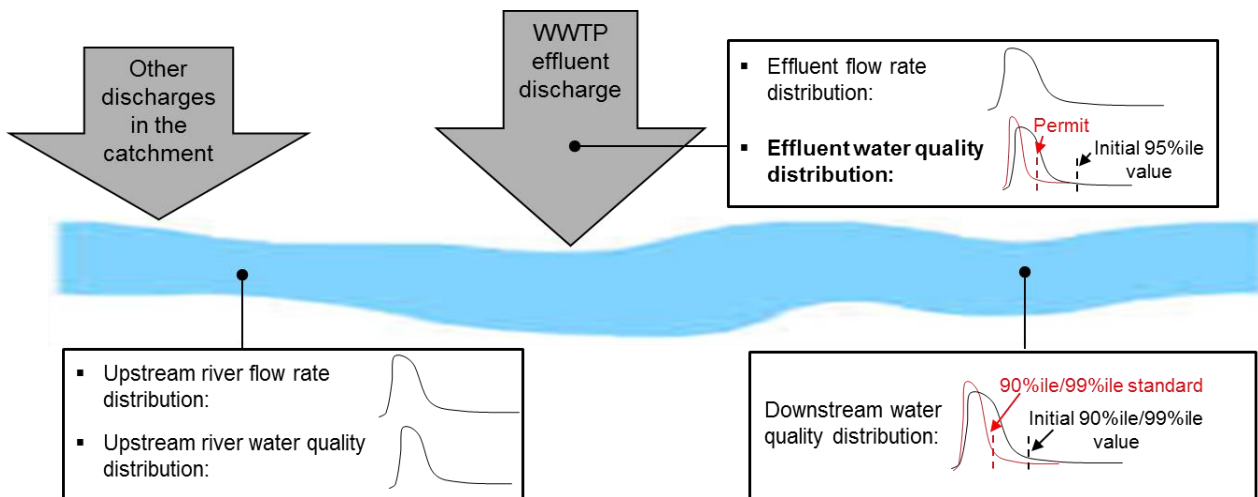


Figure 2.4 Illustration of the permitting concept by RQP (black curves are probability distributions before permitting, and red curves are modified distributions for permitting)

effluent discharge is modified by reducing mean and standard deviation values by the same scale (i.e. coefficient of variance is assumed to be constant) so that the environmental standard limit is met. The permit for WWTP effluent discharge is set according to the modified distribution (illustrated in red line in Figure 2.4) and can be described in the same statistical forms as the emission-based standard limits (e.g. 95%ile and average used in the UWWTD).

Both the emission-based and environmental quality-based limits are prescribed in the permit and the WWSPs need to meet both. Permit compliance is assessed annually based on monitoring data of the discharge in the preceding 12 months, and the analysing method varies for different parameters.

- For the DWF limit, although it is set based on the 80%ile exceedance value (Q_{80}) of historical monitoring data, the compliance analysis takes the 90%ile value to allow for natural variability (the 90%ile exceedance value is lower than the 80%ile exceedance value) (Environment Agency, 2011a). Caution needs to be taken when interpreting the percentile form of the DWF, as it is different from what is used in Table B.1, B.2 and 2.1 (Q_{80} equals to 20%ile if presented in a consistent manner).
- For pollutants in Table B.2, average values of the monitoring data are taken for the assessment.

- A look-up table (Table A.6) is provided in the UWWTD for the compliance assessment of 95%ile limits. It specifies the number of samples (24-hour composite) (Foundation for Water Research, 1994) allowed to 'fail' for a given total number of samples. The table is produced from statistical procedures with assumptions of binomial distribution and 95% confidence level (not to be confused with 95%iles) (Barnett and O'Hagan, 1997).
- For pollutants not regulated under the UWWTD but permitted in 95%iles (e.g. ammonia), the look-up table is also employed for compliance assessment. However, instantaneous spot samples rather than 24-hour composite ones are used (Foundation for Water Research, 1994).

2.2.2 Permitting for Overflow Discharges in England and Wales

The EU Directives listed in Table 2.1 also apply to CSOs/storm tank spills. Moreover, 99%ile standards and Fundamental Intermittent Standards (FIS, in concentration-duration-frequency forms) (Foundation for Water Research, 2012) are developed in the UK to protect aquatic life under wet weather conditions as shown in Table B.3 to B.5 in Appendix B. The return period of a particular set of conditions (e.g. 0.065 NH₃-N mg/L unionised ammonia for 1 hour) in the FIS is the average period of time over a sequence of years which elapses between two events when the river conditions are equal to or worse than the stated conditions. Thus the 0.065 NH₃-N mg/L - one hour - one month standard means that unionised ammonia concentration at any given point in the river can occasionally fall below 0.065 mg/L for periods equal to or longer than one hour provided that the average interval between such events is not less than one month (Foundation for Water Research, 2012).

Despite rigorous environmental standards, intermittent wastewater overflows have been controlled in a simplistic manner compared to the effluent discharge, due to the poor predictability and highly dynamic nature of the stormwater. Control measures include: a) setting a minimum pass forward flow (PFF, i.e. overflow threshold for CSOs) to the WWTP; b) building storage/storm tanks to allow for sedimentation before overflows, and c) installing overflow structures and screens to provide for elementary treatment (see details in Table 2.2). Quantitative values (e.g. tank capacity, screen size) are often set for these requirements (examples in Table A.4) which can be derived by different

approaches. More comprehensive methods generally produce more cost-effective results due to less conservative assumptions, however, more intensive resource and advanced techniques are often needed as well (Environment Agency, 2011a).

The simplest permitting approach is by 'rule of thumb'. For example, minimum capacity of storm tanks in the WWTP is often set to be 68L/capita served or storage equivalent to 2 hours at the maximum flow rate to the storm tanks (Environment Agency, 2011a). Empirical formulas can be used to determine pass forward flow and flow to full treatment (FFT, i.e. overflow threshold for storm tank overflows) as shown in Equations 2.3 and 2.4 (Environment Agency, 2011a). Equation 2.3 is also known as 'Formula A' (Foundation for Water Research, 2012).

$$\text{Pass forward flow (m}^3\text{/d)} = (PG + I_{MAX} + E) + 1360P + 2E \quad (2.3)$$

$$\text{Flow to full treatment (m}^3\text{/d)} = 3PG + I_{MAX} + 3E \quad (2.4)$$

Where P , G and E are as defined above, and I_{MAX} is the maximum infiltration rate over a complete year.

Spill frequency is regulated (in particular for protected waters, as shown in the second column of Table 2.2) in England and Wales as a surrogate indicator for surface water protection. Sewer hydraulic models (Environment Agency, 2011a; Foundation for Water Research, 2012) can be employed to evaluate the expected overflow frequency/volume. The capacity of storage/storm tanks or PFF/FFT settings is adjusted to meet the emission-based limits. To apply this method, efforts and investment are needed to collect data and build and calibrate the sewer model so that the performance of the sewer system is well predicted.

A more comprehensive approach is integrated modelling of the sewer system and the receiving water, which enables detailed analysis of the environmental impacts (such as the chemical and biological environmental indicators in Table 2.1) of potential compliance strategies (Environment Agency, 2011a; Foundation for Water Research, 2012). Though more cost-effective solutions can be produced by more comprehensive models (Environment Agency, 2011a), extra efforts and resources are needed for model development and calibration. Hence, the application should be justified by demonstrating the expected

benefits (e.g. avoidance of investment in enlarging treatment capacity) would exceed the cost (e.g. data collection and time and technical skills to establish the model).

Table 2.2 Design standards for wastewater overflows permitting (summarised from Environment Agency, 2011)

	Spill frequency	Screening	Overflow settings	Storm tank capacity
Bathing water	<ul style="list-style-type: none"> ▪ 3 spills/bathing season (good and sufficient status); and ▪ 2 spills/bathing season(excellent status) 	6 mm screening or equivalent aesthetics control	<ul style="list-style-type: none"> ▪ Empirical formulas to determine PFF or FFT; or ▪ Simulation models to achieve: <ol style="list-style-type: none"> a) Spill frequency requirements; b) FIS; c) 99%ile standards; or d) relevant EU Directives 	Minimum capacity of storm tank in the WWTP is 68L/capita served or storage equivalent to 2 hours at the maximum flow rate to the storm tanks
Freshwater fish water	<ul style="list-style-type: none"> ▪ 10 spills per annum; or ▪ Spill for 3% of the time 			
Shellfish water	10 spills per annum			
Waters under CRow Act ¹ or Habitats (BOD or ammonia)	--	Appropriate aesthetics control		
Waters under WFD (BOD or ammonia)	--			

Note: ¹Countryside and Rights of Way Act 2000 (Parliament of Great Britain, 2000).

Permit compliance is usually assessed by site inspection to make sure the required provision is in place, and by flow monitoring to record the spill events. As flow rate or total overflow volume is relatively difficult and expensive to monitor, spill event time and duration are more commonly monitored (e.g. by use of level sensors) for compliance assessment (Environment Agency, 2011a).

2.3 Permitting Regulations in the United States

Similar to the permitting regulations in the EU, a combined approach is practised in the US to control urban wastewater discharges on the basis of

emission-based standards as well as environmental water quality-based standards. The National Pollutant Discharge Elimination System (NPDES) is the regulatory framework for the permitting of urban wastewater discharges as well as other point source discharges such as industrial wastewater discharges and concentrated animal feeding operations (U.S. Environmental Protection Agency, 2010). It is coordinated and integrated with the catchment management schemes if a TMDL is developed for the catchment. The regulations under NPDES on WWTP effluent discharges and CSO spills are detailed in sections 2.3.1 and 2.3.2, respectively.

2.3.1 Permitting for WWTP Effluent Discharges in the United States

To limit waste discharge loadings to surface waters, mass-based limitations are required on WWTP effluent discharges in the permit. The limits are calculated by multiplying design flow rates and pollutant concentration limits determined by both the emission-based and environmental water quality-based limits.

The emission-based limits (named “technology-based effluent limitations - TBELs”) for WWTP effluent discharges are developed by considering performance and cost associated with the treatment technologies (U.S. Environmental Protection Agency, 2010). Secondary treatment technologies are the basic requirement in the US and a minimum level of effluent quality needs to be met as shown in Table C.1. Equivalent secondary standards (Table C.2) are set for existing processes, which employ technologies such as trickling filters and waste stabilization ponds that cannot consistently achieve the secondary treatment standards (Table C.1) but are capable of significant pollutant reductions (U.S. Environmental Protection Agency, 2010). In other words, no upgrade of these existing treatment processes is needed if the requirements in Table C.2 are met. Though secondary treatment technology is required both in the EU (Council of the European Communities, 1991a) and the US (U.S. Environmental Protection Agency, 2010), the effluent performance standards are different in many aspects, such as:

a) COD is regulated in the EU but not in the US, while pH is controlled in the US but not in the EU;

b) Effluent concentration limits and percentage reduction of BOD₅ and TSS are described in 95 percentiles (based on 24-hour composite samples) in the EU while 30-day/7-day averages in the US;

c) Compliance of percentage removal of BOD₅ and TSS can be replaced by satisfying effluent concentration limits in the EU, but the percentage reduction requirements are mandatory in the US to encourage reduction of high quantities of infiltration and inflow from the sanitary sewer systems and to prevent intentional dilution of influent wastewater; and

d) Lower effluent quality than that in Table C.1 and C.2 is allowed in the US if the flow/loading of BOD₅ or TSS introduced by industries exceeds 10% of the design flow/loading to the WWTP, whereas no such allowance is given in the EU.

In addition to the provision of appropriate treatment technologies, wastewater effluent discharges should not affect the designated uses of the receiving waters. Different from the EU policy, environmental standards are not set at national level for each water use (drinking, fish life, shellfish, recreation, wildlife, agriculture, industry and navigation, etc.). Rather, four general sets of federal environmental standards, i.e. aquatic life, human health, biological, and sediment criteria are developed, and it is the responsibility of each State to promulgate water quality standards to support designated uses of local waterbodies by referring to the national recommended values (U.S. Environmental Protection Agency, 2010, 1991). The environmental standards for aquatic life and human health are most commonly used for effluent discharge permitting thus are briefly introduced here.

The standards for aquatic life are defined in Criteria Maximum Concentration (CMC) and Criterion Continuous Concentration (CCC) for toxic pollutants with both acute and chronic effects. The CMCs and CCCs are used with conditions of duration and frequency, and could be hardness-dependent in particular for metal pollutants. An example of the chronic and acute criterion of cadmium is provided in the NPDES Permit Writers' Manual (U.S. Environmental Protection Agency, 2010) and is shown in Box 2.1. For non-toxic pollutants, only numeric values and associated duration requirements are regulated. The impact of

mixtures of pollutants can be controlled by setting whole effluent toxicity (WET) criteria to protect the aquatic life from the aggregate and synergistic toxic effects of a mixture of pollutants.

The criteria for the protection of human health are in general set to restrict chronic and bio-accumulative effects from consumption of water and/or aquatic organisms. An example for dichlorobromomethane is presented in Box 2.2. Besides, indicators like bacteria criteria are also set for the control of short-term exposure impact from activities such as contact recreation.

Box 2.1:

Chronic criterion:

The 4-day average concentration (in µg/L) does not exceed the numerical value given by $e^{(0.7409[\ln(\text{hardness})] - 4.719)}(1.101672 - [(\ln \text{hardness})(0.041838)])$ more than once every 3 years on average.

Acute criterion:

The 24-hour average concentration (in µg/L) does not exceed the numerical value given by $e^{(1.0166[\ln(\text{hardness})] - 3.924)}(1.136672 - [(\ln \text{hardness})(0.041838)])$ more than once every 3 years on average.

Box 2.2:

For the protection of human health from the potential carcinogenic effects of dichlorobromomethane through ingestion of water and contaminated aquatic organisms, the ambient water criterion is determined to be 0.55 µg/L.

For the protection of human health from the potential carcinogenic effects of dichlorobromomethane through ingestion of contaminated aquatic organisms alone, the ambient water criterion is determined to be 17 µg/L.

These values were calculated based on a national default freshwater/estuarine fish consumption rate of 17.5 grams per day.

The procedure to derive environmental water quality-based limits (named “water quality-based effluent limitations – WQBELs”) can be summarised into four steps as shown in Figure 2.5 and explained in detail as follows.

- Step I: Determine dilution allowance or mixing zone. Depending on the toxicity and impact of a pollutant to the environment, a mixing zone or

dilution allowance can be allowed when setting WQBELs. According to the definition by the USEPA, “a mixing zone is an area where effluent discharges undergoes initial dilution and is extended to cover the secondary mixing in the ambient waterbody. Mixing zone is an allocated impact zone where water quality criteria can be exceeded as long as acutely toxic conditions are prevented” (U.S. Environmental Protection Agency, 1991). Dilution allowance, described in river flow rate, is an aggregated way to designate the dilution capacity of the receiving water that can be used if the mixing process is not to be considered or complete mixing can be assumed.

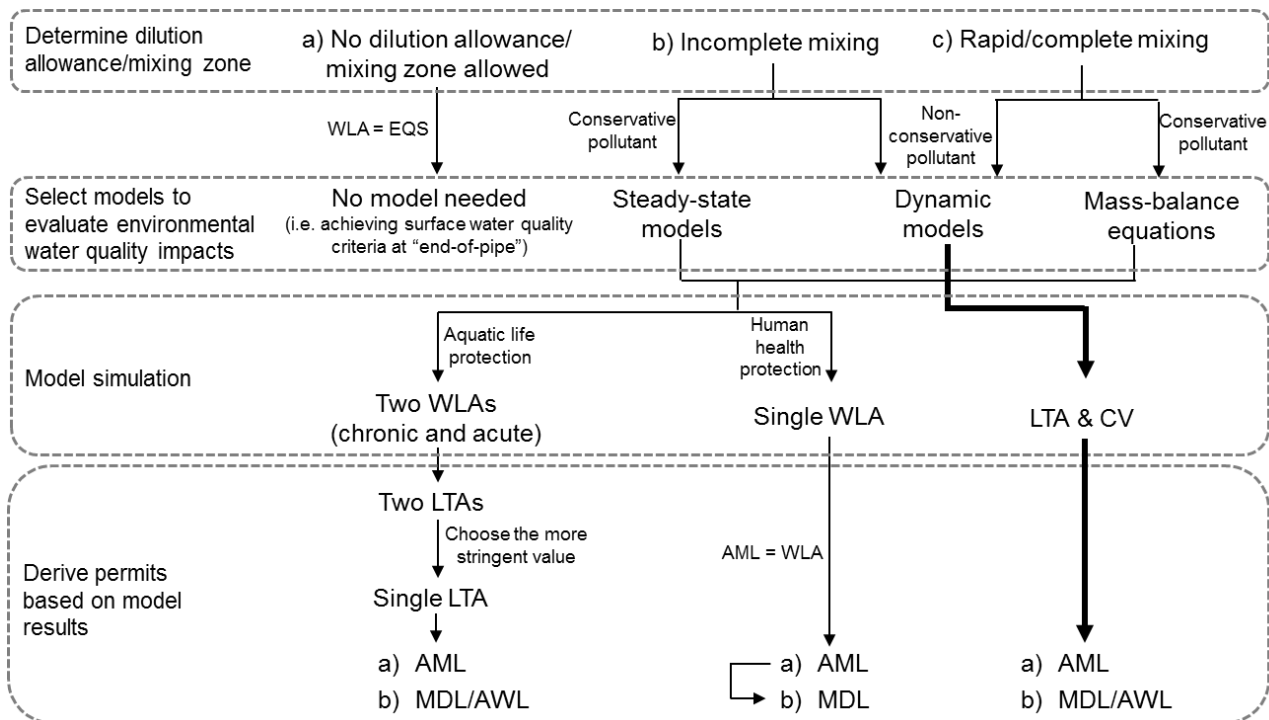


Figure 2.5 Illustration of the methods to develop WQBELs

- **Step II: Select models to evaluate environmental water quality impact.** If flow dilution is not allowed in the first step, no evaluation of the interaction between an effluent and the receiving water is needed, and environmental water quality standards need to be met at 'end-of-pipe'. Otherwise, assessment of the environmental impact of the discharge is necessary. Different approaches and models can be used depending on the nature of the pollutant and the dilution condition of the receiving water. For conservative pollutants (e.g. metals) discharging to a rapidly flowing waterbody where complete mixing can be assumed, mass-balance equations are sufficient to calculate the pollutant concentration in the

downstream watercourse; for incomplete mixing situations, steady-state hydrodynamic models such as CORMIX (Jirka et al., 1996) can be used to predict the mixing zone behaviour. For non-conservative pollutants subject to reactions and decay, dynamic rather than steady-state models are necessary to study the pattern of pollutant concentration change over time. The mixing zone model CORMIX allows simulation of first-order decay or growth processes. More complex chemical or biological processes are represented by dynamic models such as QUAL2E (Brown and Barnwell, 1987) and WASP (Wool et al., 2003) with no simulation of mixing behaviours.

- Step III: Establish and calibrate the model. If a water quality model is to be employed, flow and water quality data of the effluent and the receiving water should be collected. As mass-balance equations and steady-state models use only one set of parameters, data of critical conditions are used to make conservative estimates. The critical conditions are usually combinations of worst-case assumptions of the river flow, effluent and environmental effects. Examples are 7Q10 (7-day average, once in 10 years) river flow, highest effluent discharge flow and lowest upstream river water quality. Dynamic models produce less conservative results as variability in the flow and quality of effluent discharge and the assimilation capacity of the receiving water are considered in developing effluent requirements (U.S. Environmental Protection Agency, 1991). However, development of dynamic models is usually more resource intensive, and model calibration is often necessary if a comprehensive model is selected.

The final result from a mass-balance calculation and a steady-state model is a WLA (i.e. effluent water quality value) back calculated to achieve the environmental standard, or two WLAs if the receiving water is protected for aquatic life so two sets of environmental standards should be complied with. If a WLA has already been assigned to a discharge through a TMDL, it can be used directly for permit derivation in the next step. With regards to a dynamic model, the average pollutant concentration value (named “long-term average - LTA”) and coefficient of variance (CV) produced to meet the environmental standards, would be the basis for permit derivation.

- Step IV: Derive permits based on model results. Manipulation is needed to convert model results yielded in the previous step to appropriate forms required for permits. A statistical procedure, as briefly introduced below, is provided by the USEPA for permit derivation (U.S. Environmental Protection Agency, 1991) assuming that the effluent water quality values follow lognormal distributions. In cases where this assumption is not valid, different statistical procedures need to be followed.

In cases where two WLAs are produced, the WLAs (usually assumed to be 99%ile or 95%ile concentration values) are first transformed to LTAs following the lognormal distribution assumption. The more stringent value of the two LTAs is often chosen for permitting to be environmentally protective. The selected LTA is transformed to average monthly limitation (AML, which is the highest allowable value for the average of daily discharges obtained over a calendar month), maximum daily limitation (MDL, which is the highest allowable discharge measured during a calendar day or 24-hour period representing a calendar day) or average weekly limitation (AWL, which is the highest allowable value for the average of daily discharges obtained over a calendar week). MDLs are often required for toxic pollutants, as AMLs and AWLs designed for TBELs of conventional pollutants could average out peak toxic concentrations thus are inappropriate for the control of acute toxic effect. The derivation of MDL, AML and AWL (often expressed in 99%ile or 95%ile) from LTA is similar to that for WLA to LTA, only that the number of samples taken to determine the average value is factored in producing the AML or AWL. Permitting based on dynamic models are simpler, as the LTA and CV from model outputs can be directly used to calculate MDL, AML or AWL.

Permitting for human health protection is somewhat different from the procedure described above because the exposure period is rather long-term which can be up to 70 years. Hence, a more defensible method is recommended, which makes the WLA equals to AML, and the MDL is then calculated by multiplying AML by a ratio factor determined jointly by CV of the effluent discharge and number of samples taken to yield the AML.

The AML and MDL/AWL constitute the WQBELs for the effluent discharge. WWSPs should comply with both the TBELs and WQBELs. Self-monitoring programmes, overseen by quality assurance schemes, are set up to reduce regulatory burdens without sacrificing the quality of the monitoring and data collection practices. (U.S. Environmental Protection Agency, 1991).

2.3.2 Permitting for Overflow Discharges in the United States

Similar to the situation in the EU, intermittent wastewater overflows have not been as effectively controlled as WWTP effluent discharges. Short-term and long-Term Control Programs (LTCP) (U.S. Environmental Protection Agency, 1995) are set to reduce waste loadings from CSOs. The short term program consists of nine minimum controls as listed below.

- 1) Proper operation and regular maintenance programs for the sewer system;
- 2) Maximum use of the collection system for storage;
- 3) Review and modification of pretreatment requirements to assure CSO impacts are minimised;
- 4) Maximisation of flow to the publicly owned treatment works for treatment;
- 5) Prohibition of CSOs during dry weather;
- 6) Control of solid and floatable materials in CSOs;
- 7) Pollution prevention;
- 8) Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts; and
- 9) Monitoring to effectively characterise CSO impacts and the efficacy of CSO controls.

Besides the minimum requirements set by the nine controls, an LTCP should also be adopted eventually to enhance surface water quality. The LTCP can be implemented by a demonstration approach or a presumption approach. The demonstration approach is applicable to cases where sufficient data are available or can be collected. Under this approach, the adequacy of the CSO control program to meet water quality standards need to be demonstrated by the monitoring data. If the environmental standards and designated water uses are not met in part because of natural background conditions or pollution

sources other than CSOs, a TMDL which allocates waste loading to different pollution sources including CSOs should be developed.

The second approach is based on the presumption that water quality standards will be attained with implementation of an LTCP that meets certain performance criteria below.

- No more than an average of four overflows events per year; and
- The elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected during precipitation events on a system-wide annual-average basis.

Models of the sewer systems (and receiving waterbodies) are usually needed for this approach.

2.4 Towards Flexible Permitting Policy

The two permitting systems reviewed in sections 2.2 and 2.3 represent the most comprehensive practices of conventional wastewater discharge permitting. Despite the differences in many aspects, such as the permitting model and statistical form of permitted pollutant limits, some similarities are identified as summarised below.

- a) The main (if not the only) goal of traditional wastewater discharge permitting is environmental water quality protection, and the impact of the regulation on GHG emission control and cost is given limited consideration.
- b) WWTP effluent discharges are controlled by prescription of the minimum level of wastewater treatment technology (most commonly secondary treatment) and setting up end-of-pipe pollutant concentration limits.
- c) End-of-pipe concentration limits are determined based on the capability of the treatment technology as well as the environmental needs of the receiving water.
- d) CSO discharges are less effectively regulated, monitored and appraised than WWTP effluent discharges. The pollution control is mainly by provision of sufficient storage capacity, screening of floatable materials, sedimentation of particulate pollutants, and/or operation to minimise the volume of wastewater spilled and retain for treatment in the WWTP.

As such, the limitation of the traditional permitting policy is obvious: the fragmented regulation of WWTP discharges and CSO spills, and the uncoordinated treatment efforts with other environmental protection measures. Under the increasingly stringent environmental water quality requirements, in particular the focus on ecological integrity of surface waters, an innovative permitting policy is in demand to improve the environment in a sustainable way. This has been explored by exploiting the spatial and temporal changes in wastewater generation patterns and environmental conditions, and by taking a coordinated and integrated management approach. Some examples of flexible permitting practices are given below in this section.

2.4.1 Catchment-Based Permitting Practices

Catchment management policy offers an opportunity to coordinate and optimise treatment efforts for all polluting sources in a catchment, instead of putting unnecessary rigorous limits on discharges from the UWWs. The most commonly practised catchment-based permitting approach, which is also the closest form to the traditional regulatory method, is to issue permits for individual discharges based on a holistic analysis of the catchment conditions. By coordinating the individual permits, catchment water quality can be more effectively and efficiently improved than single-source oriented regulations. Nevertheless, the individual permits need to be strictly complied with at each discharge point with little regard to the compliance cost. To encourage delivery of environmental goals in a cost-effective manner, more flexible forms of catchment-based permitting are practised in some areas, such as the two examples given below.

a) Multi-source catchment-based permitting

In the US, pollution sources in the same catchment can apply for and obtain permit coverage under the same permit. This is especially suitable for cases where a catchment management plan identifies the need to address a specific pollutant from multiple sources (U.S. Environmental Protection Agency, 2007). A permit obtained by this approach is developed according to the agreed-upon actions for achieving environmental goals in a catchment management plan and identification of pollution sources that are logical to group under a single permit. For instance, a permit can be issued on phosphorus reduction to all WWTPs in

the catchment. This type of permit can work as an addition to existing ones, meaning other pollutants would continue to be addressed through each facility's individual permit.

b) Water quality trading

Water quality trading allows one pollution source to meet its regulatory obligations by purchasing pollutant reductions created by another source that has cheaper, environmentally equivalent or superior pollutant reductions (U.S. Environmental Protection Agency, 2007). This market-based approach is designed to achieve water quality improvement at a reduced cost and can be practised among point pollution sources, or between point and non-point pollution sources. For example, trading is applicable when the implementation of non-point source Best Management Practices (BMPs) is less costly per unit of pollution reduction compared to upgrading point source treatment technology (Woodward and Kaiser, 1990; Woodward, 1996; Ng and Eheart, 2005).

Despite the growing interests in water quality trading and its application in the US, Australia, New Zealand and Canada (Selman et al., 2009), the implementation has not been as effective as in air pollution markets for a number of reasons (Woodward, 1996). Firstly, the physical property of water determines that water pollution could not be uniformly dispersed over a wide area, but instead confined to a catchment. Thus the number of potential participants for effluent trading is limited, and chances for suitable trade could be slim. Secondly, the environmental impacts of water pollution can be quite variable depending on the point of discharges. Localised pollution problems might be yielded by trading which contradicts the principle of water environment protection. Thirdly, monitoring and enforcement are very expensive, predictions of nutrient loads need to be more precise and legal conflicts might arise between estimated pollutant reductions achieved by the trading and the reductions required by the regulation.

2.4.2 Integrated Permitting Practices for Urban Wastewater Discharges

A similar principle of catchment-based permitting has been applied to urban wastewater systems to manage wet weather overflows and/or WWTP effluent discharges in a holistic way. Flexible permitting practices include integrated

permitting of overflow discharges in the same sewer system, and integrated permitting of overflow spills and WWTP effluent discharges in the same UWWS.

a) Integrated permitting of wet weather discharges

Municipal wet weather discharges share a number of common characteristics, such as driven by rainfall events or snow melts, containing similar types of pollutants and may be hydraulically connected (U.S. Environmental Protection Agency, 2007). By integrated permitting of wet weather discharges, it offers an opportunity of comprehensive planning so that the sewer system can be operated and managed in a better way to achieve improved water quality outcomes, greater efficiency and less cost. Moreover, the integrated permitting could promote source control measures (e.g. green roofs, SuDS) if they are more cost-effective than the traditional end-of-pipe approaches.

b) Integrated municipal permit

This approach bundles a number of point source discharge (e.g. CSOs, WWTP effluent discharges) permits for a municipality into a single permit. It reduces administrative burden for both the regulated party and permitting authority, and promotes delivery of better environmental outcomes. By strategic assessment, the most critical problems can be targeted and addressed with greater resources and protection.

A successful application is the integrated permit issued to the Clean Water Services (a public utility) for the control of thermal loads to the Tualatin River Watershed (Oregon, US) from four WWTP effluent discharges, two industrial stormwater discharges and a separate sewer system discharge. To meet the permit requirement, 10 miles of riparian shading was planted which prevented 101 million Kcal/day of thermal energy from impacting the Tualatin River. The integrated permit facilitates the achievement of environmental objective in a cost-effective manner as the adopted control measure is cheaper than alternative compliance strategies such as installation of refrigeration equipment at the WWTPs or piping treatment facility effluent to another river basin, and it provides economies of scale for the Clean Water Services in terms of resource use (Clean Water Services, 2007).

2.4.3 Dynamic Permitting Practices

The operation of the UWWs has commonly been conservative, with a large number of WWTPs operating at full capacity continuously (Tchobanoglous et al., 2004) instead of taking advantage of the fluctuating load to the treatment plant and the changing assimilation capacity of the receiving waterbody at different times of a year. A reason for this is that effluent discharge permits are traditionally set to be complied with throughout the year. To address this, a few forms of dynamic permitting have been introduced which allow for flexible operation in accordance with season, month, or almost in real-time with the changing wastewater flow or environmental conditions.

a) Seasonal-based permitting

Rivers usually exhibit seasonal patterns of flow rate and temperature as a result of local climate, thus different treatment processes or treatment levels could be employed to adapt to the varying assimilation ability of the receiving water among seasons. Advanced treatment processes (e.g. biological nitrification process) rather than normal secondary level could be applied during summer periods, when the river flow rate is low on average and the temperatures are high. In winter, on the contrary, only normal secondary treatment process, or even lower treatment removal levels are needed (Boner and Furland, 1982; Lence et al., 1990). In the research by Ferrara & Dimino (1985), a modified aeration tank unit was bypassed in winter, so that the treatment plant operated as a conventional activated sludge plant instead of a two-stage nitrification facility. This approach is appropriate for the nitrification system because of the lower nitrification rate, lower NH_3 toxicity and higher flow rate in the receiving water during winter times. Impact assessment shows that river water quality could still be preserved if seasonal varying effluent limits are complied with. Furthermore, a total annual saving of £48,235 (\$82,000) was estimated for the first year operation of the system, due to the reduced electricity costs, monitoring and sampling analysis needs and sludge treatment efforts.

b) Monthly-based permitting

If the environmental changes (e.g. river flow rate and temperature) display regular monthly patterns and do not deviate much inter-annually, monthly

variable permitting might be applicable for cost savings without deteriorating surface water quality. Compared to seasonal permitting, it is more accurate to represent the dynamic environmental conditions in months, while still keeps reasonable time intervals for generalisation and for operation changes in WWTP. For example, the aeration could be reduced if less stringent effluent limits are required; or some filtration units could be bypassed with reduced BOD₅ limits in certain months. An investigation in Georgia, US suggested capital cost savings of up to 16% and annual operational cost savings of up to 19% if monthly variable effluent limits were to be adopted (Reheis et al., 1982). Though considerable cost savings could be foreseen, frequent changes in major operations are not recommended, especially when biological treatment processes are involved, which usually require days or weeks to reach the required steady states.

c) Real time-based permitting

As the environmental condition is constantly changing, requirements on effluent performance should in theory be varying accordingly. However, it is impossible to impose and comply with real-time end-of-pipe permits, hence prescriptive permitting which specifies real-time operational and/or control strategies has been introduced. Operational/control handles with long reaction time for change to take effect, like sludge pumping rates for biological treatment processes, are often not considered for this permitting approach. A representative example of practice is permitting on real-time control of sewer systems, with aims of retaining wet weather flows in the system and diverting to WWTPs for treatment rather than overflowing to the environment (Environment Agency, 2011a; U.S. Environmental Protection Agency, 1995). Disinfection of effluent discharges in England and Wales employs a similar approach (Environment Agency, 2011a). The operation of UV disinfection needs to vary in accordance with effluent quality and performance of the disinfection equipment. If the effluent transmissivity is poor, extra UV lamps need to be turned on to achieve the required level of disinfection.

2.5 Conclusions

The conventional permitting approaches have been reviewed in this chapter, which discloses the lack of coordination in the regulation of WWTP effluent

discharges and CSOs as well as the control on water quality and GHG emissions. The progress towards flexible permitting, as shown by some current practises in section 2.4, suggests that integrated management strategy and technological advances are being increasingly valued and embraced for better wastewater discharge permitting. The impact of urban wastewater discharges can now be appraised at a catchment scale with other pollution sources, and flexibility in the operation of UWWS can be utilised to control wastewater transportation and treatment in accordance to environmental changes. Yet the existing flexible permitting practices are still fragmented in the control of continuous and intermittent wastewater discharges and improvement of water quality remains to be the focus of regulation. Hence, there is a need for more integrated permitting policy such as through integrated operation and control of the UWWS based on multi-criteria analysis. Three innovative permitting approaches are proposed in this work, as will be presented in Chapters 4 to 6, based on a holistic understanding of the UWWS performance to achieve enhanced and balanced environmental benefits in a cost-effective manner. The newly developed methods differ from the existing traditional/flexible permitting approaches in that more sophisticated wastewater system modelling and optimisation tools are employed to support decision-making, thus a review on integrated UWWS modelling and multi-objective optimisation algorithms is provided in Chapter 3; furthermore, they are proposed as performance-based regulatory tools rather than the widely used outcome-based approach, hence their viability as regulation alternatives as well as the roadmaps for the implementation are discussed in Chapter 7.

3 Literature Review: Integrated Modelling and Control of Urban Wastewater Systems and Multi-Objective Optimisation

In light of the opportunity of incorporating operational and control strategies of UWWs into flexible permitting as identified in Chapter 2, this chapter presents how improvement in environmental quality can be achieved by optimising an integrated control strategy of UWWs. In the following sections, the progress in integrated modelling of UWWs is reviewed first, followed by a presentation of the studies on optimisation of operational and control strategies. As multiple criteria (such as surface water quality and cost) can be simultaneously considered in the optimisation, tools which enable multi-objective optimisation are introduced in section 3.3. The modelling, control and optimisation techniques selected for the work in Chapters 4 to 6 are described in section 3.4. As an extensive review can be found in previous studies on integrated modelling, operation and control of UWWs (Meirlaen, 2002; Olsson and Newell, 1999; Schütze et al., 2002) and multi-objective optimisation (Naeini, 2013; Wang, 2014), only a brief overview is provided in this chapter for the background knowledge of the following chapters.

3.1 Integrated Urban Wastewater System Modelling

For better manipulation and control of the urban wastewater transportation and treatment processes, it is useful to establish models for individual or combined components of the integrated UWW. Whilst steady-state models could be fit-for-purpose for system design or regulatory management, dynamic models are often needed in developing optimal operation and control strategies against a changing environment. Hence the review in this section only focuses on dynamic models.

To enable water quality prediction, it is often necessary to simulate the hydraulics, pollutant transport and physicochemical and biochemical reactions in the wastewater system. Based on improved understanding of the system processes, components of the integrated UWW can now be represented in great detail (Henze et al., 2000; Hvitved-Jacobsen et al., 2013; Jolánkai, 1992). Schütze et al. (2002) provided a comprehensive overview on some widely used

modelling concepts and software for each component so details are not repeated here. Some commonly used modelling techniques for the sewer system and river (other surface waterbody types not reviewed), and WWTP (broken down to different treatment units) are summarised in Tables 3.1 and 3.2, respectively.

Traditionally, models for the individual components of the integrated system were developed in a separate way with limited consideration of the impact from/to other components. However, the interactions are non-negligible to the system performance. For example, the surface runoff directly determines the wastewater load transported in the combined sewer systems, which in turn affects the amount of CSOs to the receiving water and inflow to the WWTP; sewage septicity and sulphide generated in the sewer system are associated with sludge bulking in the WWTP; and the operation in the primary clarifier affects the treatment performance in the activated sludge reactor, which in turn influences the solid settling property in the secondary clarifier (Schütze et al., 2002). To maximise the performance of the entire system, efforts have been made in modelling the sewer, WWTP and receiving water in an integrated manner and several simulation software platforms, such as SIMBA (IFAK, 2009), WEST (Meirlaen, 2002) and SYNOPSIS (Schütze et al., 2002) are available now. In the context of integrated UWSWS modelling, however, it is resource and time inhibitive to represent all components in the system in a comprehensive manner, as a significant amount of monitoring data would be required to identify the large number of parameters in a complex model (Beck, 2002). Therefore, simplified strategies are commonly adopted in the simulation of certain processes or components using current integrated modelling platforms.

Biochemical reactions in aeration tanks of activated sludge treatment processes (other types of wastewater treatment technologies not reviewed) are almost always modelled in detail due to their key role in biological wastewater treatment. The International Water Association (IWA)'s Activated Sludge Model (ASM) series (Henze et al., 2000), including ASM1 (carbon oxidation, nitrification and denitrification), ASM2 (ASM1 plus biological phosphorus removal), ASM2d (a minor extension to ASM2 by including denitrifying phosphorus accumulating organisms and two chemical processes for chemical

precipitation of phosphorus) and ASM3 (correcting ten defects in ASM1) are the state-of-the-art and most widely used models for the description of the biochemical and (limited) physicochemical reactions. Water quality processes in the receiving water are also simulated in integrated modelling to evaluate the environmental impacts of wastewater discharges. River Water Quality Model No.1 (RWQM1) (Reichert et al., 2001; Shanahan et al., 2001), developed also by the IWA, is applied in several integrated modelling platforms (e.g. SIMBA, WEST) due to its compatibility with the IWA ASMs. However, the full RWQM1 is rather comprehensive (30 processes and 24 components) thus simplified versions are often used (Vanrolleghem et al., 2001). In addition to the RWQM1 family models, a few (relatively) simple river water quality models, such as Lijklema (Lijklema et al., 1996) and SWQM (Schütze et al., 2011) simulating key in-stream water quality processes (e.g. degradation of organic matters, nitrification, reaeration, photosynthesis and sedimentation), have also been applied. Biological activities in other components of the integrated UWWS system are often neglected in modelling for simplicity, though the impact may not be non-negligible in certain situations (e.g. sewer systems or tanks with long retention time) (Gernaey et al., 2001; Wang et al., 1995) thus should be considered in modelling.

As the hydraulic retention time of the process units in the WWTP is designed to be long (in orders of hours) to facilitate sedimentation of solid pollutants and biochemical reactions in the aerator, the WWTP treatment units can be assumed as ideal reactors with complete mixing (Henze, 2008). The sedimentation process is often described by empirical equations as a function of residence time and/or inflow rate, though more comprehensive layer models can be used in particular for secondary (or primary) clarifiers for more accurate prediction. The flow regime in the sewer system and the river is similar and is simulated as open channel flow: the flow transport process can be modelled by detailed but time-consuming hydrodynamic methods or simpler hydrologic flow routing methods; and the pollutant transport can be represented as mechanistic Advection-Dispersion equations or simpler continuous stirred-tank reactor (CSTR) approaches.

Table 3.1 A summary of modelling methods for stormwater and wastewater transport and reactions in catchment, sewer and river

UWWS component	Flow transport	Pollutant (soluble) transport	Physicochemical or biochemical reactions
Surface runoff	<ul style="list-style-type: none"> a) Hydrologic flow routing methods (e.g. single/cascade linear/nonlinear reservoirs); and b) Unit hydrograph 		First-order production/decay equations
Sewer	<ul style="list-style-type: none"> a) Hydrodynamic modelling methods (e.g. full/approximations of Saint Venant equations); b) Hydrologic flow routing methods (e.g. Nash cascade); c) Unit hydrograph; and d) Pure translation 	<ul style="list-style-type: none"> a) Advection-Dispersion equation; and b) CSTR 	<ul style="list-style-type: none"> a) Sedimentation and resuspension processes; and b) Multi-phase (wastewater, biofilms and sewer sediments) transformation processes (e.g. WATS model)
River	<ul style="list-style-type: none"> a) Hydrodynamic modelling methods (e.g. full/approximations of Saint Venant equations); and b) Hydrologic flow routing methods (e.g. reservoir cascades, the Muskingum, Muskingum-Cunge and Kalinin-Miljukov methods) 		<ul style="list-style-type: none"> a) First-order production/decay equations; b) Streeter-Phelps equation (DO & BOD); and c) Detailed multi-pollutants models similar to ASMs

Table 3.2 A summary of modelling methods for process units in activated sludge WWTPs

WWTP component	Solid pollutant removal (i.e. sedimentation)	Soluble pollutant removal
Storage/storm tank	a) Empirical models as a function of: <ul style="list-style-type: none"> ▪ settling velocity and flow velocity; or ▪ residence time; and b) Four operational mode models (i.e. fill, dynamic sedimentation, quiescent settling and draw modes)	No removal is usually assumed
Primary clarifier	a) Empirical models as a function of: <ul style="list-style-type: none"> ▪ suspended solids concentration and/or inflow rate; or ▪ Residence time; and b) Distributed-parameter models (i.e. predicting both temporal and spatial behaviour of the system)	No removal is usually assumed, though first-order reactions can be defined (such as for hydrolysis process)
Aerator	--	a) Time-series models; b) Reduced order methods; c) Quasi-dynamic models; and d) Detailed mechanistic dynamic models (e.g. ASM No.1, No. 2, No. 2d and No. 3)
Secondary clarifier	a) Empirical models (e.g. effluent SS is related to inflow rate and return sludge rate); b) Sophisticated layer models; and c) 2D/3D models considering hydrodynamic effects	No removal is usually assumed

In the absence of integrated modelling, surrogate indicators, such as CSO spill frequency, overflow volume and pollutant discharge load, had to be used in the evaluation of the impact of wastewater discharges on the receiving water. These emission-based surrogate indicators are highly aggregated and can be misleading in representing environmental impacts (Lau et al., 2002). Integrated modelling is a valuable tool in providing a holistic view of the system performance. Indicators of river water quality can be used directly in the evaluation of system design, operation and control. Results from previous studies have shown the potential for significant improvement in river water quality by optimising an integrated operational strategy of an UWWS without the need for upgrade or redesign of the treatment system (Butler and Schütze, 2005; Schütze et al., 2002). Further enhancement in the system performance is achievable by a) implementing integrated real-time control (RTC) strategies in responsive to the dynamic environment and b) incorporating multiple objectives in the optimisation as reviewed in sections 3.2 and 3.3.

3.2 Real-Time Control of Integrated Urban Wastewater Systems

3.2.1 An Overview of Real-Time Control Technology

A system is considered to be real-time controlled, if process variables are monitored in the system and, (almost) at the same time used to operate the actuators in the flow process (Schütze et al., 2004). The control is implemented in the unit of control loop, which consists of hardware components such as sensors, controllers and actuators. Sensors monitor process evolution and transmit the state of the system to controllers, which would adjust the actuators according to the deviations of the controlled process variables from the desired values (set-point values). There are three widely used forms of control: open-loop control, feedback control and feed-forward control, as illustrated in Figure 3.1.

Open-loop control is the simplest form of automatic control. There is no automatic feedback from measurement and the control is based on a timer or predetermined programme of action (Olsson and Newell, 1999). It can be applied to turn on a pump or air compressor at certain times of the day (Qasim,

1998), or to add chemical solutions in proportional to wastewater flow (Henze, 2008), etc.

Feedback control monitors the status of the process variables, and feed the information to the inputs to steer the process to where it is wished to go. There are different types of control algorithms for feedback control, of which on-off and Proportional-Integral-Derivative (PID) algorithms are the two dominating ones in wastewater system application. On-off control is widely used in the overflow structures of CSOs and storm tanks, where tank filling is initiated when the flow rate is above a threshold and water in the tank is emptied and pumped back to the treatment process when the flow rate reduces below a level. The PID algorithm is a more complex control type and is commonly used in process industries. It is a combination of three control actions, proportional (P), integral (I) and derivative (D), the control of which are based on present errors, accumulated past errors and predicted future errors based on the current rate of change. An example of the application of the PID algorithm (or its simplifications P, PI and PD) in wastewater industry is the control of air supply according to the DO concentration in the aerator to maintain a relatively stable DO level (i.e. set-point). By cascading two or more feedback control loops, a more advanced control can be achieved by varying the set-point value with time according to the need of the process. For example, the air supply intensity can be controlled according to the ammonia concentration at the end of the aerator (Olsson and Newell, 1999) to save energy without compromising the treatment efficiency.

In feed-forward control, sensors are installed in the process inputs to detect disturbances so as to adjust the system operation before the process is affected by the disturbances. It is a more desirable control type compared to the open-loop and feedback control in that the process output could be free from disturbances if the feed-forward operation can compensate and cancel out the effects of the disturbances. For example, the DO set-point in the aerator can be determined according to the flow rate and ammonia concentration of WWTP influent so that a right amount of air could be supplied for the removal of ammonia. Feed-forward control is considered as a simple form of model-based control (Olsson and Newell, 1999), as it uses a simplified (often linear) representation of the process to predict the system response and determine the control action accordingly. For more advanced model-based predictive control,

non-linear models of the controlled system may be used directly without linearization.

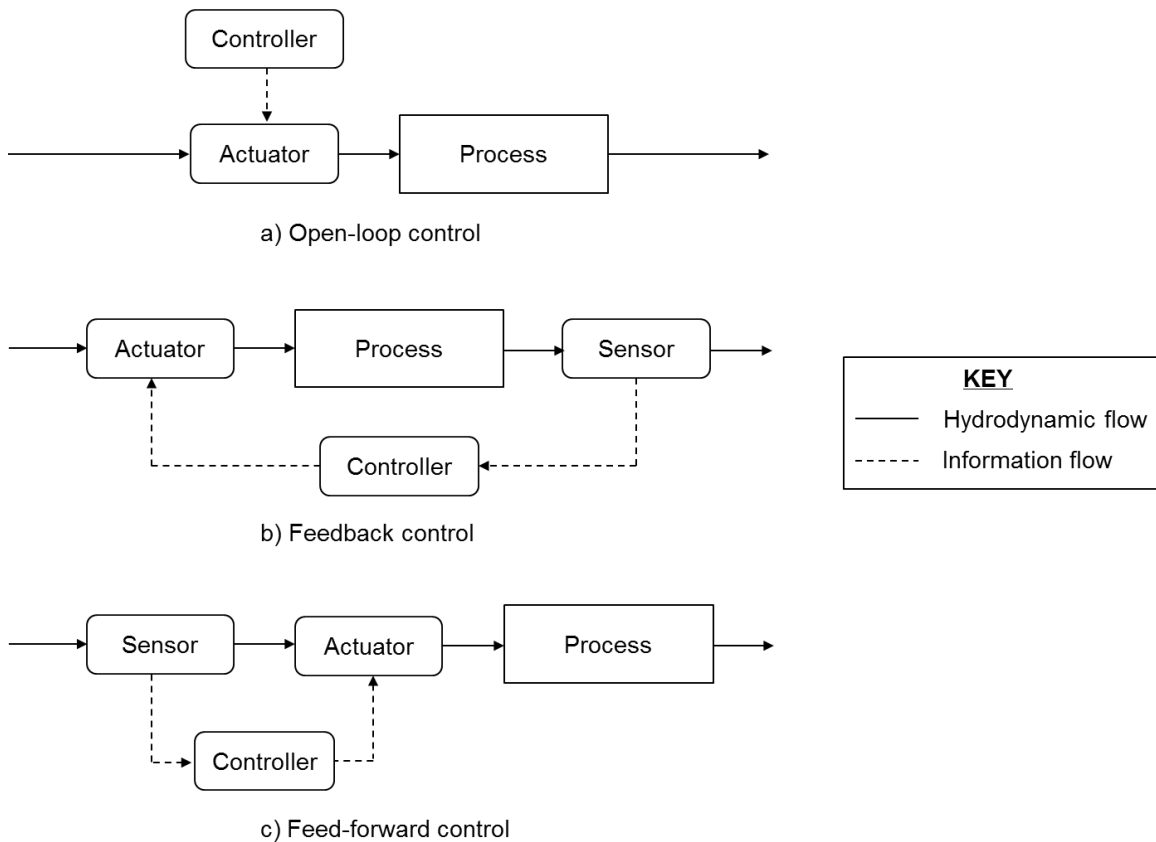


Figure 3.1 Illustration of control loops for: a) open-loop control; b) feedback control; and c) feed-forward control (adapted from Pleau et al., 2005)

3.2.2 Development of Real-Time Control Technology in Integrated Urban Wastewater System Control

The application of RTC technology in the control of wastewater treatment process has gradually matured in line with the development of automation control technology and the understanding of the treatment processes. Started from primary control of water levels, flow rates, pressures and temperatures, the application then developed further into concentration control, which requires at least a basic knowledge of the reactions and processes. Mathematical models are usually formulated based on the acquired knowledge and combined into the controller design. With the development of on-line nutrient sensors, the fixed set-point DO control in the reactor, which is a surrogate parameter for biological process control, evolved into variable set-point DO control with direct and more reasonable objectives such as ammonia removal rate (Olsson, 2012).

Besides the control of single treatment units, RTC can be applied to the whole WWTP to coordinate and optimise the control in the plant systematically (Duyy and Laboratorium, 1975; Serra et al., 1993). This is termed “global control” as defined in Schütze et al. (2002), referring to control where sensor information from within the same subsystem (i.e. sewer system, WWTP, or receiving water) is used to determine the setting of a control device. Examples are ratio controlled return sludge pumping rate according to inflow rate to the WWTP (Bauwens et al., 1996), and overflow threshold setting of CSOs based on volume and quality measurement in the sewer system (Petruck et al., 1998).

The “integrated control” of the UWWS, as opposed to “global control”, is characterised by two aspects (Schütze et al., 1999):

- *Integration of objectives:* Objectives of control within one part of the UWWS may be based on criteria measured in other subsystems; and
- *Integration of information:* When taking a control decision within one part of the system, information about the present or predicted future state of another subsystem may be used, hence state information is transferred across subsystem boundaries.

Examples of integrated control are Aeration Tank Settling in the WWTP based on rainfall prediction or flow information in the sewer (Nielsen et al., 1996); overflow threshold setting of detention basins by downstream river DO condition (Rauch and Harremoës, 1999a); control of the inflow to the WWTP according to ammonia concentration in the downstream river (Meirlaen et al., 2002); and a hierarchical control that overrides local controllers on CSO overflow threshold and inflow to the WWTP according to the loading condition in the sewer system, WWTP and the receiving water (Schütze et al., 1999). Despite the advance in the research on RTC of the integrated UWWS, real-life implementation of the RTC technology is mostly (if not all) limited to local (i.e. sewer system or WWTP) control (Alsuis et al., 2004; Fuchs and Beeneken, 2006; Maeda et al., 2004; Pleau et al., 2005; Thornton et al., 2010).

For predictive global or integrated UWWS control, a complex non-linear system model is usually used to determine time-varying set-points or control inputs

according to process evolution. The RTC system is typically structured in three hierarchical levels, i.e. field level, system level and supervisory level (Olsson, 2012; Schütze et al., 2004) as represented in Figure 3.2. The sensor information from all units of the system is gathered and structured in the field level and transmitted to the system level. On this second level, reasoning modules, containing a heuristic knowledge of the process, would use the experience from previous similar and particular operating situations to provide suggested strategies. The strategies yielded on the system level would be sent upwards to the supervisory level, where simulation model of the system would be employed to evaluate the strategies. The optimised strategy is then conveyed downwards for implementation (Olsson, 2012).

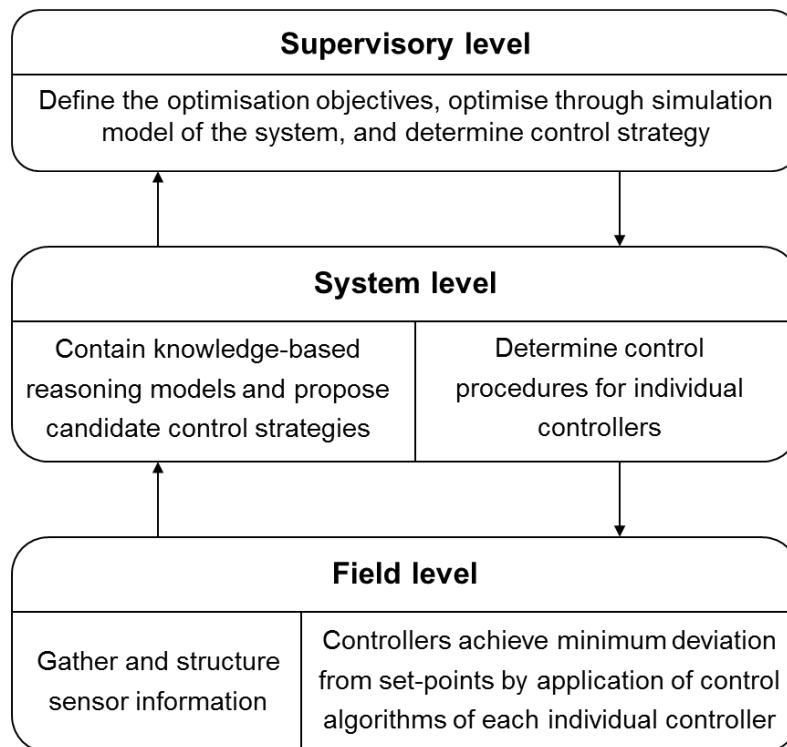


Figure 3.2 Typical hierarchical levels in a global or integrated RTC system (adapted from Olsson and Newell (1999) and Schütze et al. (2004))

Predictive control can be optimised online or offline. In online optimal control, the simulation model is fed by real-time sensor information and provides estimation of the performance of control actions in a specified prediction horizon (e.g. 2 hours). The optimal control action(s), which performs best in achieving pre-defined goals, can be evaluated and implemented at every control time step (e.g. 5 minutes). As the computational time of detailed mechanistic models may

be too great to be practical for online control optimisation, model simplification is often needed (Schütze et al., 2004).

Despite the reasonable logic and successful application in some real-life cases (Pleau et al., 2005; Scheer et al., 2004), the use of online optimal control faces a number of problems related to practical applicability. In particular, when considering the system in its entirety, this can include the potential long-term effects associated with some water flow and quality changes (e.g. loss of nitrification in the treatment plant, sediment oxygen demand in the receiving water body) (Butler and Schütze, 2005). As an alternative, offline optimal control could be employed. As computational time is less of an issue in the offline approach, detailed modelling of the wastewater system can be used to analyse the long-term impacts of the control actions. The control algorithm could be pre-defined in the form of a set of “if-then” rules or a decision matrix. The quantification of the set-point values and parameters in the control algorithm can be optimised by different approaches, ranging from simple trial-and-error method to sophisticated stochastic optimisation tools (e.g. Genetic Algorithms) (Butler and Schütze, 2005).

3.3 Multi-Objective Optimisation Tools

Evolutionary algorithms (EAs) are a class of stochastic optimisation methods that simulate the process of natural evolution (mainly natural selection and variation) (Zitzler, 1999). They are considered to be especially suited to multi-objective optimisation (Zitzler, 1999) and perform better than other blind search strategies (Fonseca and Fleming, 1995; Valenzuela-Rendon and Uresti-charre, 1997). Multi-objective evolutionary algorithms (MOEAs) are chosen for the optimisation of integrated UWWS operation and control in this research because a) the UWWS is a non-linear system with various physical, chemical and biological processes, so the search for ‘best’ control strategy cannot be solved by analytical methods; b) there are many operational/control handles in the system and therefore numerous combinations of operational/control variable settings, which makes it impractical to use enumerative techniques; and c) different (even conflicting) aspects of the system performance can be considered simultaneously in a single optimisation run. Representative MOEAs include Non-dominated Sorting Genetic Algorithm (NSGA) (Srinivas and Deb,

1994), Niche-Pareto Genetic Algorithm (NPGA) (Horn et al., 1994), Strength Pareto Evolutionary Algorithm (SPEA) (Zitzler and Thiele, 1999), and Pareto Archived Evolution Strategy (PAES) (Knowles and Corne, 2000).

3.4 Modelling, Control and Optimisation Strategies for This Work

Integrated UWWS modelling for this work is performed on the widely used software platform SIMBA6. Details of the simulation methods of the case study UWWS are presented in section 4.3.2. NSGA-II (Deb et al., 2002), an improved version of NSGA and popular for its computational efficiency and good performance (Coello, 2006; Khare, 2002), is employed in this study. It is reported to have been applied in other urban wastewater management studies, such as structure optimisation and technology screening of UWWSs (Huang et al., 2015), optimal design of urban drainage systems (Muleta and Boulos) or integrated UWWSs (Quintero, 2012), and optimal operation and control of WWTPs (Sweetapple et al., 2014a) or integrated UWWSs (Fu et al., 2008). Model-based predictive control is used for the investigation of RTC-based permitting, because it could minimise (compared to open-loop and feedback loop control) the adverse impacts of disturbances to the system if designed properly. As long-term and detailed evaluation of wastewater system performance is necessary for the permitting studies in this work, no model simplification is adopted and the control algorithm is optimised offline.

To optimise the integrated UWWS operational/control strategy by NSGA-II, an optimisation problem needs to be formulated first, which consists of:

- Optimisation objectives, i.e. indicators of the integrated UWWS performance (e.g. river water quality, operational cost);
- Decision variables (i.e. the settings of the operational/control handles) and associated value ranges; and
- Constraints, such as design requirements and legal/regulatory obligations to be complied with. As physical/hydraulic laws of water flow in the UWWS are provided by the set of equations that govern the cause-and-effect relationships in the model, they do not need additional specifications for the constraints.

The optimisation of integrated operation and control is carried out by coupling the optimisation algorithm and an integrated modelling platform. NSGA-II first randomly generates a population (i.e. the first generation) of operational/control strategies within the defined ranges, each of which is evaluated by a long-term simulation in SIMBA6. Results of the system performance after the evaluation are fed back to the algorithm and compared with other control strategy solutions in the generation. Those of good performance are selected to 'breed' the next generation, and after a designated number of generations, a Pareto front of optimal solutions is produced. They are non-dominated solutions which cannot be further improved in terms of one objective without worsening another. Although the Pareto optimal solutions are not the best ones in an absolute mathematical sense, they are the best approximate solutions achieved within limited resources.

4 Operational Strategy-Based Permitting

4.1 Introduction

As reviewed in Chapter 3, the advance in integrated modelling enables optimisation of operational strategies in an UWWS based on a holistic view of the system performance, and the potential benefits such as in improving river water quality and reducing cost have been demonstrated by a number of studies. However, the previous research on optimisation of integrated operational strategies has been conducted with limited representation of real-life constraints from environmental policy. This is reflected by the simplified form of standard limits (e.g. maximum ammonia concentration) (Fu et al., 2008; Schütze et al., 2002) in describing river water quality, incomplete application of environmental standards (e.g. wet weather-related standards only) (Lau et al., 2002; Meirlaen, 2002), short evaluation period (e.g. one week) (Fu et al., 2008; Schütze et al., 2002), and no coverage of the impact of applying an optimal integrated operational strategy on the compliance of wastewater discharge permits. As such, the previous research findings provide limited insights on the regulatory motivation to consider and promote integrated operational strategies. To fill this gap, further research is needed to address the following questions:

- How much improvement in environmental water quality can be gained from optimising integrated UWWS operational strategies if restrictions from the environmental policy are fully considered?
- Would it add value to take account of other factors such as GHG emissions in the optimisation?
- What are the advantages and disadvantages of implementing optimal integrated operational strategies under the traditional permitting regime?
- How could the permitting policy be adapted to better deliver the benefits brought by the integrated operational strategy?

These four questions have been investigated in this work, and are presented in reverse order in this chapter. In the following sections, a newly developed approach, based on an integrated operational strategy rather than traditional end-of-pipe limits or CSO overflow frequency, is described first in section 4.2.

Application of the proposed method to a semi-hypothetical UWWs case (description of the case study and modelling strategies provided in section 4.3) is presented in section 4.4. Results are analysed in section 4.5, with a focus on discussing the advantages of the proposed approach over the traditional method and examining the reliability of the method against a dynamic environment.

4.2 Operational Strategy-Based Permitting Framework

A four step decision-making framework is proposed for the development of operational strategy-based permitting as illustrated in Figure 4.1 and explained in detail as follows.

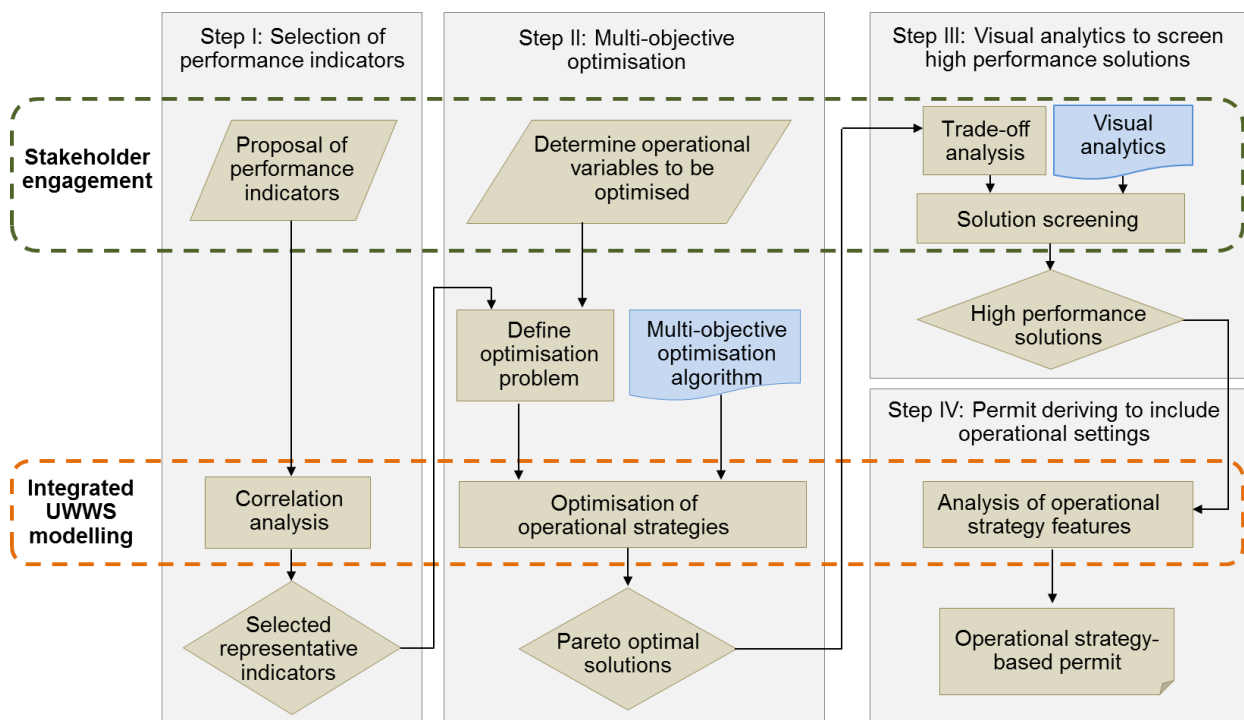


Figure 4.1 Decision-making framework for operational strategy-based permitting

Step I: Selection of system performance indicators to represent different interests.

Due to the wide environmental, economic and social impacts of permitting policy (Johnstone and Horan, 1996), stakeholders are engaged in the first step to identify different interests and formulate them into performance indicators. Representative metrics that appropriately describe the various interests are then used as objectives to optimise system operational strategies in Step II. As UWWs are complex with multiple interactions with the environment, it is

difficult to select representative indicators by intuitive judgment. Thus an UWWS model is used to evaluate the correlations between the performance indicators by analysing results from various operational scenario simulations. If two or more performance indicators are strongly correlated, only one is needed for further steps of the decision-making process (Hurford et al., 2014).

Step II: Multi-objective optimisation of the operational strategies to reveal objective trade-offs.

NSGA-II is employed for the multi-objective optimisation of the operational strategies (each group of settings is one operational strategy), coupled with the integrated UWWS simulation platform. The optimisation is set up according to the procedure provided in section 3.3. The optimisation objectives are the performance indicators selected in the first step. The definition of the decision variables (i.e. settings of the operational handles) and associated value ranges needs support from stakeholders who have detailed knowledge of the UWWS. Constraints of the optimisation could include the environmental water quality standards of the receiving water body.

Step III: Visual analytics to screen high performance solutions.

The results of the optimisation can be difficult to interpret, as a range of optimal solutions are produced which perform differently against various objectives. Visual analytics tools can analyse large data sets in an informative and visually appealing way to facilitate decision-making (Fu et al., 2013; Hurford et al., 2014). Thus it is applied in this study to provide a holistic view of the trade-offs between the objectives, i.e. the benefits achievable in one performance aspect and the level of sacrifice required in other aspects. Based on the trade-off relationships and practical concerns (e.g. financial constraints, water quality planning targets), desirable solutions are selected from the pool of optimal results. An interactive cyclic screening process, assisted by the visual analytics tools, is set up to incorporate the decision-makers' preference in the selection of high performance solutions. Stakeholders are also engaged in this step to input local knowledge so that practically achievable decisions are made.

Step IV: Permit deriving to include operational settings.

Details of the selected solutions are assessed to explore common operational features to achieve the desired performance. Based on this, a set of operational variable values are determined as the permit. In this case, an uncertainty analysis is conducted using Latin Hypercube Sampling (LHS) (Iman et al., 1980; McKay et al., 1979) to assess the sensitivity of system performance to operational setting changes. The confidence ranges of operational settings which produce reliable performance are also included in the permit to allow for flexibility.

4.3 Case Study

4.3.1 Definition of the Case Study Site

As no integrated dataset was available for this work, the proposed permitting approach was applied to a semi-hypothetical integrated UWWS, which consists of a sewer system adapted from a literature standard (ATV, 1992), an activated sludge WWTP (a typical and widely used treatment technology in the UK) based on and calibrated against the Norwich works in the UK (Lessard and Beck, 1993) and a hypothetical river (Schütze et al., 2002). It serves a population of about 150,000 producing an average DWF of 27,500 m³/d. This case study was first built by Schütze et al. (2002) for the research on modelling and control of integrated UWWSs, and has since been used in a number of studies, such as assessment of performance indicators for CSOs (Lau et al., 2002), screening for RTC potential of UWWSs (Zacharof et al., 2004), multi-objective optimisation of integrated UWWS operational strategies (Fu et al., 2008), assessment of combined effects of climate change and urbanisation on the river water quality in an integrated UWWS (Astaraié-Imani et al., 2012) and integrated environmental assessment of green and gray infrastructures (Casal-Campos et al., 2015). The layout of the integrated UWWS is shown in Figure 4.2.

The sewer system consists of a network of seven sub-catchments, with a total impervious area of 725.8 ha (7.258 km²). Four online pass-through storage tanks are set up at the downstream end of the linked sub-catchments. The flow setting limiting the maximum onward flow is defined as a multiple of DWF flowing to each storage tank. Besides the four storage tanks, an off-line pass-through storm tank is located at the inlet of the treatment train, resulting in a

total storage volume of the system of 19,950 m³. Filling of the storm tank starts as soon as the maximum inflow rate to the primary clarifier is reached, and emptying is triggered when the inflow drops below a threshold value. Other process units in the WWTP are a primary clarifier, an aeration tank, a secondary clarifier and a mechanical dewatering unit. The receiving river has a base flow of 4.5 m³/s that provides a dry weather dilution ratio of approximately 1:15. The river is 45 km in length and is equally divided into 45 reaches. The runoff generated by rainfall on the upstream catchments enters as an additional inflow into the river at reach 4. The four CSOs in the sewer are combined and discharged to reach 7. The intermittent spill from the storm tank and effluent discharge of the WWTP enter the river at reaches 9 and 10 respectively to observe their separate impacts to the receiving river. The dimensions of the catchment, the treatment process units and the river are provided in Table 4.2.

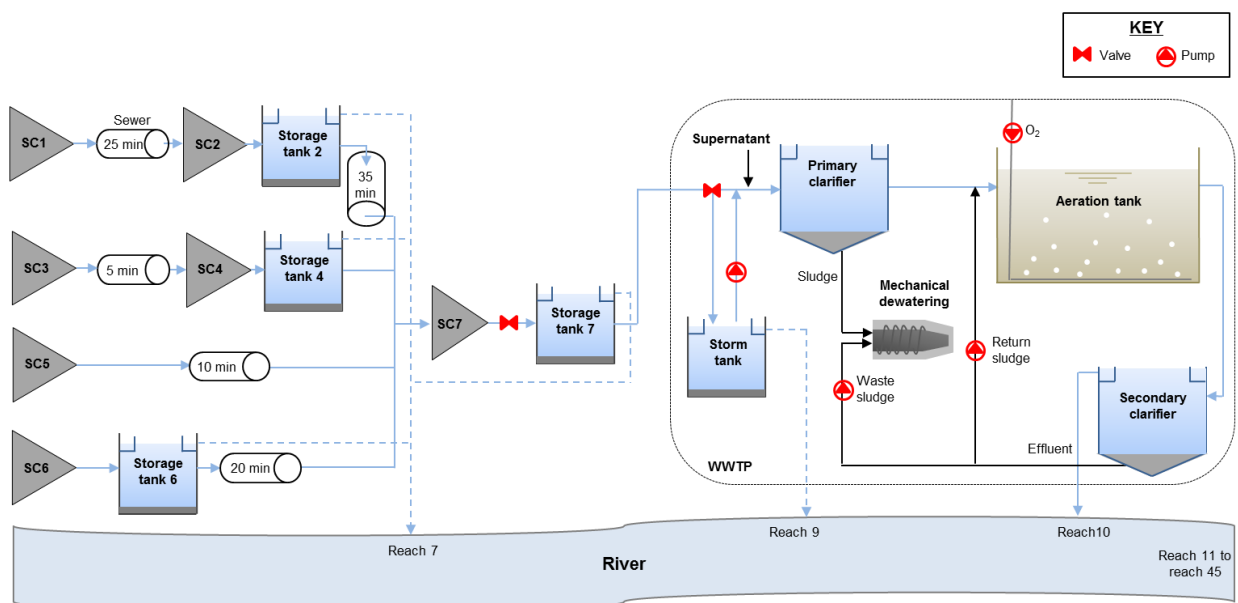


Figure 4.2 Schematic representation of the catchment (SC: sub-catchment)

The flow and water quality data of the DWF in the sewer system (Schütze et al., 2002), rainfall runoff (Schütze et al., 2002) and supernatant flow from the sludge dewatering unit in the WWTP (Lessard and Beck, 1993) are presented in Table 4.1. The values for the runoff and supernatant are assumed to be constant in the simulation, while that for the DWF are average values and are used by multiplying pre-defined diurnal patterns (Schütze et al., 2002).

Table 4.1 Flow and water quality data for dry weather flow, rainfall runoff and supernatant flow of the case study UWWS

	Flow rate (L/s)	Water quality (mg/L)					
		COD	COD _{soluble}	SS	VSS	NH ₄ +NH ₃	NO ₃
Dry weather flow	318.3	606	281	335	245	27.7	0
Rainfall runoff	--	100	46	190	139	2	0
Supernatant	20	8,221	84	7,595	6,155	12	0

4.3.2 Modelling of the Case Study Site

Though the case study site has been used by several studies, the modelling strategies applied may not be the same due to the different simulation platforms used (SYNOPSIS in Lau et al., (2002), Schütze et al. (2002) and Zacharof et al. (2004), SIMBA5 in Astaraie-Imani et al. (2012) and Fu et al. (2008), and SIMBA6 in Casal-Campos et al. (2015) and this work) and diverse modelling techniques provided even by the same simulation platform. The model employed for this work (modelling method for each component summarised in Table 4.2) is similar to the one used in Fu et al. (2008), however a few changes were made to suit the purpose of this study. The major modifications are listed below.

- In previous studies, the sludge treatment unit was not modelled. However, sludge treatment and disposal are one of the most important cost factors. For example, the operational cost for sludge treatment can amount to more than half of the total operational cost for wastewater treatment (Nowak, 2006). Therefore, a mechanical dewatering unit was added to have a more complete representation of the cost consequences of different operational strategies.
- The modelling of the primary clarifier was modified to adapt to the introduction of the sludge treatment unit. Firstly, settled sludge in the primary clarifier was drawn at a constant rate (about 15% of DWF by referring to Schütze et al. (2002), comparable to that reported by Tardy (2011)) to the

sludge dewatering unit. Secondly, the supernatant flow from the sludge treatment unit was added to the front of the primary clarifier for treatment.

- For simplicity, ammonia is the single pollutant investigated in this work.
- The average river flow rate was increased to three times the previous value $1.5 \text{ m}^3/\text{s}$, as a preliminary optimisation run suggested that no operational strategy could meet the environmental standards on total ammonia with the original designated dilution capacity.
- A one-year simulation was set up so that long-term performance of the system can be evaluated. In the previously established models, the evaluation of system performance was rather short-term (e.g. one week) so wastewater temperature and upstream river flow rate and water quality were assumed to be constant. To accommodate long-term simulations, a pattern of seasonal wastewater temperature was defined and one-year input data sets (rainfall and corresponding river data) were incorporated into the model. As no monitoring data on temperature of the Norwich WWTP were available, a seasonal pattern ($18 \text{ }^\circ\text{C}$, $23 \text{ }^\circ\text{C}$, $19 \text{ }^\circ\text{C}$ and $15 \text{ }^\circ\text{C}$ from spring to winter) was assumed by adjusting a WWTP wastewater temperature pattern reported in the literature (Shatat and Al-najar, 2011) to data on the local climate of Norwich (Hughes, 2006). Detailed river water quality data is commonly scarce, thus a hypothetical dynamic river data set was generated by adding an agricultural runoff pollution source to the upstream of the hypothetical river. After finishing the research work for Chapters 4 and 5, however, a detailed data set of river water quality from another area along with the corresponding river flow and rainfall records were acquired from the Environment Agency. Due to time constraints, however, results produced using the semi-hypothetical input data are used to illustrate the proposed method, while the newly acquired real-life data are employed to examine the reliability of the methodology. Details on the two data sets are provided as follows.

Data set 'A':

One year (01/10/2012 to 30/09/2013) 15-minute increment time series of total ammonia concentration and flow rate of the runoff from North Wyke

Farm (Okehampton, UK) were used to generate a dynamic upstream river water quality (annual average about 0.09 NH₃-N mg/L). To reflect the dynamic dilution capacity of the river at different times of the year, monthly river base flow rates were defined (annual average 4.5 m³/s) according to the rainfall data. The one-year rainfall 15-minute increment time series (687.60 mm/year) (Figure 4.3a) corresponding to the agricultural runoff was used in the model to generate urban runoff and stormwater flowing to the sewer system.

Data set 'B':

This data set is from one-year (24/05/2013 to 23/05/2014) records of an online analyser placed downstream of a WWTP in an English Midlands river. The automatic sampler monitors in-river total ammonia concentration every 30 minutes and flow rate every 15 minutes. The flow scale of the river (annual average about 5.6 m³/s) is similar to that of data set 'A', but the river water quality (annual mean: 0.67 NH₃-N mg/L, 90%ile: 0.95 NH₃-N mg/L, 99%ile: 1.84 NH₃-N mg/L) is much worse (for data set 'A', annual mean: 0.09 NH₃-N mg/L, 90%ile: 0.09 NH₃-N mg/L, 99%ile: 0.11 NH₃-N mg/L). Despite the natural decay of ammonia occurring in the river flow, it is impossible, according to a preliminary optimisation run, to achieve at reach 11 (after all wastewater discharges) the environmental standards applied to the first data set (second row of Table 4.3, corresponding to the fourth row of Table B.4) or the less stringent set of standards (third row of Table 4.3, corresponding to the fifth row of Table B.4). The two sets of environmental limits are the standards in England and Wales for different types of rivers (Defra, 2010; Foundation for Water Research, 2012). To demonstrate the benefits of optimal operational design, the monitoring data on total ammonia concentration were downscaled to a level where the environmental standards are still violated but can be met through optimisation of operational strategies. In this study, the downscaling factor is defined to be 0.6 and the more relaxed set of environmental standards was employed. The downscaling factor was deliberately designed to be not too small so as to simulate a very different scenario with a more polluted river. The corresponding one-year rainfall data (868.79 mm/year) are shown in Figure 4.3b at an hourly time step.

Table 4.2 Dimensions of the case study UWWS and the modelling methods

Process unit	Dimension	Hydraulic/pollutant transport model	Models for sedimentation	Models for biochemical reaction processes
Catchment	Total area of 7 sub-catchments: 725.8 ha	Nash cascade	Not modelled	
Sewer	--	Translation		
Storage tank	Tank 2: 2800 m ³ ; Tank 4: 1400 m ³ ; Tank 6: 2000 m ³ ; and Tank 7: 7000 m ³		Simplified model by a coefficient of settling efficiency	Not modelled
Storm tank	6750 m ³			
Primary clarifier	6785 m ³	Completely mixed reactors	Empirical equation as a function of hydraulic retention time (HRT)	
Aerator	10,400 m ³		--	An extension of Activated Sludge Model No. 1
Secondary clarifier	6600 m ³		3-layer model, using exponential function to simulate settling velocity	Not modelled
Mechanical dewatering	--	--	Idealised solid separation	
River	4.5 m ³ /s	SWMM5	Not modelled	Lijklema

Table 4.3 Environmental standards on total ammonia concentration in England and Wales applied to data sets ‘A’ and ‘B’

Data set	90 percentile (mg/L) ^[1]	99 percentile (mg/L) ^[2]
A	0.3	0.7
B	0.6	1.5

Note:

[1] Requirements from the UK regulation transposing the WFD requirement (Defra, 2010);

[2] Requirements from the Urban Pollution Management Manual (Foundation for Water Research, 2012);

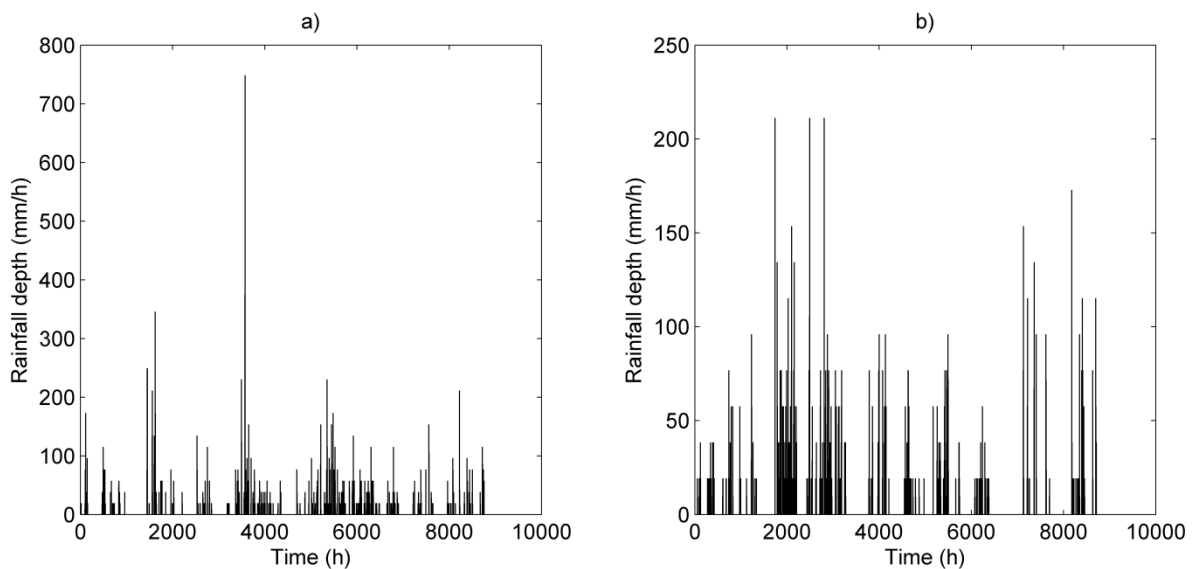


Figure 4.3 Rainfall time series of a) data set ‘A’ (Oct 2012 to Oct 2013) and b) data set ‘B’ (May 2013 to May 2014)

Discrepancies are expected between predictions by the established model and real world data. For one reason, it is impractical, as mentioned in section 3.1, to make extensive simulation of all (possibly known) processes in each treatment unit in the context of integrated modelling. For another, no model exists that can accurately represent a real life system due to limitations in our knowledge and sources of uncertainty that cannot be reduced by more data/studies (see more in section 4.5.2). Hence, some processes are simulated in a simple manner (e.g. mixing, sedimentation in storm/storage tanks) or not accounted in modelling (e.g. biochemical reactions in the sewer) in this case study as no sufficient data is available to identify and calibrate parameters of more sophisticated models. Nevertheless, processes critical for wastewater treatment and its environmental

impacts, namely sedimentation in the secondary clarifier and biochemical reactions in the aeration tank and the receiving river, are modelled in a relatively detailed manner.

4.3.3 Operational Scheme of the Case Study

In the baseline scenario, the settings of the key operational handles in the studied integrated UWS are shown in the second column of Table 4.4 (overflow threshold settings of tanks 2, 4 and 6 are also 5 times the DWF flowing to each tank). According to a one-year simulation with data set 'A', both the 90%ile and 99%ile total ammonia concentration in the downstream river, being 0.38 NH₃-N mg/L and 0.84 NH₃-N mg/L respectively, fail the environmental standard limits (i.e. 0.3 NH₃-N mg/L and 0.7 NH₃-N mg/L). So the operational settings listed in Table 4.4 are optimised to find operational solutions to meet the river target whilst maximising other aspects of system performance. Overflow settings for tanks 2, 4 and 6 are not optimised due to their weak impact to the overall system performance revealed by a sensitivity analysis (more descriptions on sensitivity analysis provided in section 6.3.1).

Table 4.4 Base-low-high values of the operational variables of the case study

Operational variable	Baseline value (m ³ /d)	Lower bound value (m ³ /d)	Higher bound value (m ³ /d)
CSO (tank 7) overflow threshold	137,500 (i.e. 5DWF)	82,500 (i.e. 3DWF)	220,000 (i.e. 8DWF)
Storm tank overflow threshold	82,500 (i.e. 3DWF)	55,000 (i.e. 2DWF)	137,500 (i.e. 5DWF)
Storm tank emptying threshold	24,000	16,800	31,200
Storm tank emptying rate	12,000	7,200	24,000
Aeration rate	720,000	240,000	1,200,000
Return sludge pumping rate	14,400	7,200	24,000
Waste sludge pumping rate	660	240	960

4.4 Results

This section presents the process and results of applying the proposed methodology in section 4.2 to the case study. The presented results are an illustration of the optimisation and permitting processes, and it is not the intention of this research to prescribe a specific operational strategy or permit. The investigated case is used to demonstrate the effectiveness of the newly developed permitting framework and permits will vary from case to case.

4.4.1 Selection of Performance Indicators

For illustration purposes, the following indicators are proposed to describe potential economic and environmental interests.

- a) Energy cost incurred in pumping, aeration and sludge treatment. It is selected to measure economic implications of operation changes. Moreover, it is a good indicator of GHG emissions, especially the amount of emissions under regulation (Parliament of the UK, 2010).
- b) Water quality of the WWTP effluent. Effluent water quality can be described by pollutant concentration levels measured by different statistical parameters (e.g. 95%ile as used in effluent discharge permits), pollutant discharge load, and stability of water quality expressed as standard deviation of effluent water quality time series (Niku and Schroeder, 1981).
- c) Downstream river water quality. Similar statistical parameters, such as 90%ile and 99%ile as required by UK standards and standard deviation derived from long-term simulation data, can be used to represent river water quality.
- d) Environmental risk. A risk indicator is introduced (Equation 4.1) according to the widely used definition as the product of probability and consequences (Liu et al., 2011; Siu, 1994). By definition, it complements other risk-related parameters (e.g. 99%ile river quality limit (Foundation for Water Research, 2012), fundamental intermittent standards (Foundation for Water Research, 2012)) by measuring the probability and consequence of water quality deterioration beyond threshold limits.

$$Risk = \sum (P_j \times (C_j - C_{limit})) \quad (4.1)$$

where C_j (mg/L) is total ammonia concentration in the river at time j ; C_{limit} (mg/L) is the threshold limit which is set as the 90thile river total ammonia standard in this work; and P_j is probability of occurrence of C_j exceeding C_{limit} . P_j is determined by dividing the duration of the consequence ($C_j - C_{limit}$) in a run by the total simulation time. This equation calculates the shaded area of the time series graph of river quality in Figure 4.4.

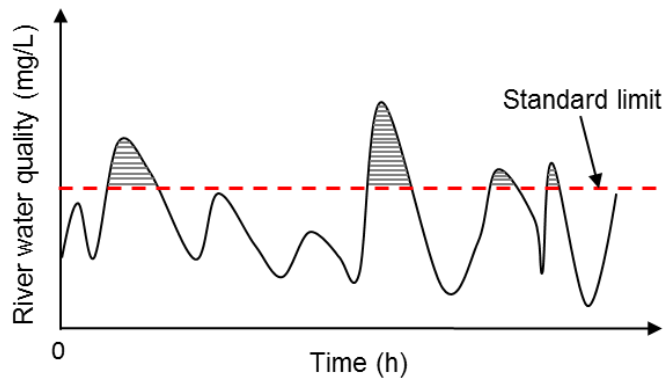


Figure 4.4 Illustration of risk calculated in a time series of river water quality

LHS is used to select representative performance indicators from the variety of proposed ones. As a popular sampling technique of drawing random samples from input probability distributions (Iman et al., 1980; McKay et al., 1979), LHS is employed to generate 1000 operational scenarios by drawing random values of operational settings from the feasible ranges shown in Table 4.4 (a uniform distribution is assumed for the values of each operational variable setting). The proposed indicators are calculated based on one-year simulation results of the 1000 operational scenarios. By analysis of the correlation relationship of the investigated indicators, operational cost, effluent quality standard deviation and environmental risk are selected as representative indicators. The definition and calculation of the three objectives are presented as below.

1) Energy cost:

Energy cost refers to the expenditure incurred in pumping, aeration and sludge treatment as calculated using Equations 4.2-4.5:

$$\text{Operational cost} = C_{pump} + C_{aeration} + C_{sludge} \quad (4.2)$$

$$C_{pump} = 0.1 \times E_{pump} \quad (4.3)$$

$$C_{aeration} = 0.1 \times E_{aeration} \quad (4.4)$$

$$C_{sludge} = 7.95 \times 10^{-5} \times V_{ts} \times C_{ts} \quad (4.5)$$

where C_{pump} (£) is the cost for pumping, E_{pump} (KWh) is the total electricity consumption from pumping within the simulation period, $C_{aeration}$ (£) is the cost for aeration, $E_{aeration}$ (KWh) is the total electricity consumption from aeration, C_{sludge} (£) is the cost for sludge treatment, V_{ts} (m³) is the total volume of thickened waste sludge, and C_{ts} (mg/L) is the concentration of the thickened waste sludge. The constant 0.1 is the electricity tariff rate (£/KWh) defined for pumping and aeration in this study. The constant 7.95×10^{-5} is the mechanical dewatering cost (£) per gram of dry waste sludge (Mamais et al., 2009).

2) Effluent standard deviation:

Percentile values of WWTP effluent total ammonia concentration of the 160 compliant solutions from the 1000 simulated scenarios are summarised by box plots in Figure 4.5, which show that obvious variation of effluent quality lies in high percentile values (90%ile and above). Thus correlation analysis is made between high percentile values and other statistical indicators as presented in Table 4.5.

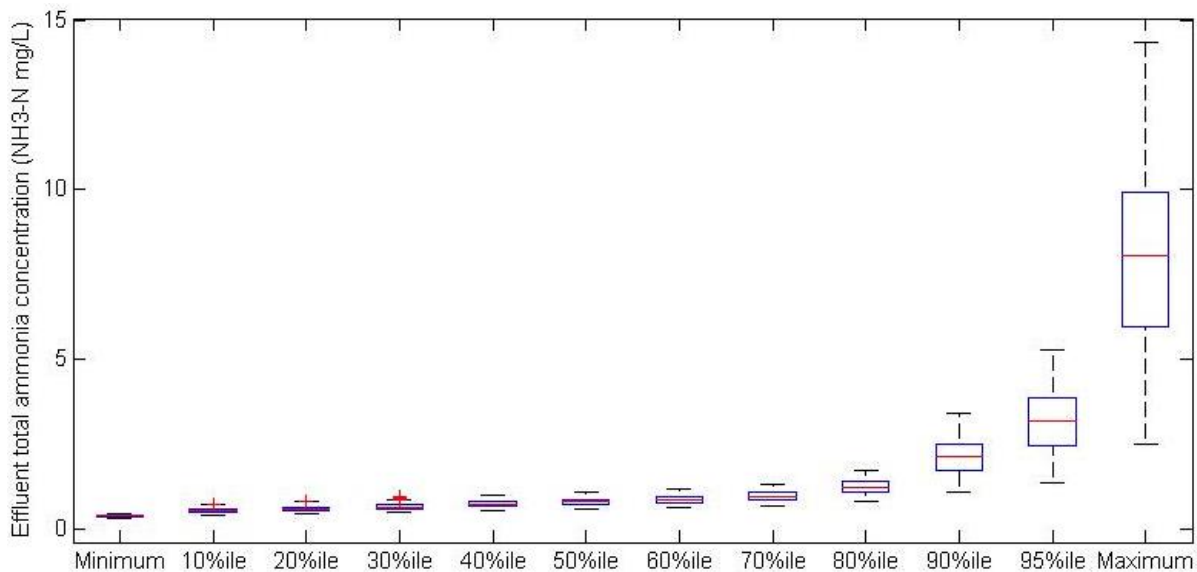


Figure 4.5 Distribution of effluent total ammonia percentile concentration values obtained from 160 simulation scenarios

Table 4.5 Correlation relationships between mean/standard deviation values with percentiles of effluent total ammonia concentration of the 160 simulation scenarios

	Correlation coefficient				
	70%ile	80%ile	90%ile	95%ile	max
Mean value	0.878	0.973	0.981	0.945	0.852
Standard deviation	0.536	0.744	0.925	0.981	0.985

Standard deviation is selected to be the most representative one due to the increasing correlation with higher percentile values. It is also strongly correlated (correlation coefficient: 0.878) with the 90%ile river water quality value as regulated in the WFD. This indicates that stability of the WWTP is highly influenced by events causing deterioration in effluent quality, and it in turn affects chemical quality of the receiving water body.

3) Environmental risk:

The environmental risk indicator is chosen due to the high correlation with the 99%ile and standard deviation of river total ammonia concentration and total discharge load from the UWWS (correlation coefficients: 0.907, 0.984 and 0.921). It is defined according to Equation 4.1.

4.4.2 Multi-Objective Optimisation and Trade-off Analysis

The settings of the seven operational variables are optimised within the reasonable ranges (Table 4.4) to minimise the three objectives described in Equations 4.6-4.8, subject to the legislative constraints listed in Table 4.3.

$$\text{Min } (C_{\text{pump}} + C_{\text{aeration}} + C_{\text{sludge}}) \quad (4.6)$$

$$\text{Min } (STD_{AMM}) \quad (4.7)$$

$$\text{Min } (Risk) \quad (4.8)$$

Given the computational inefficiency of running long-term simulation in SIMBA6, a practical approach is adopted to balance between population size and

generation number of NSGA-II. A widely accepted setting of population size 100 is used in this study (Deb et al., 2002), and a usage of generation number of 15 is found to produce satisfactory Pareto fronts, and thus is used in this study and repeated for ten random seed runs. Default settings of distribution index for crossover (20) and mutation (20) are used.

The optimisation results are projected against the three objectives shown in Figure 4.6a, and separately in three pairs from Figures 4.6b to 4.6d. Solving the three-objective optimisation problem automatically solves three sub-problems at the same time (i.e. non-dominated solutions of two-objective optimisation can be deduced directly from the three-objective optimal solutions, without the need for running three two-objective optimisations), and the results are shown in different symbols from Figures 4.6b to 4.6d.

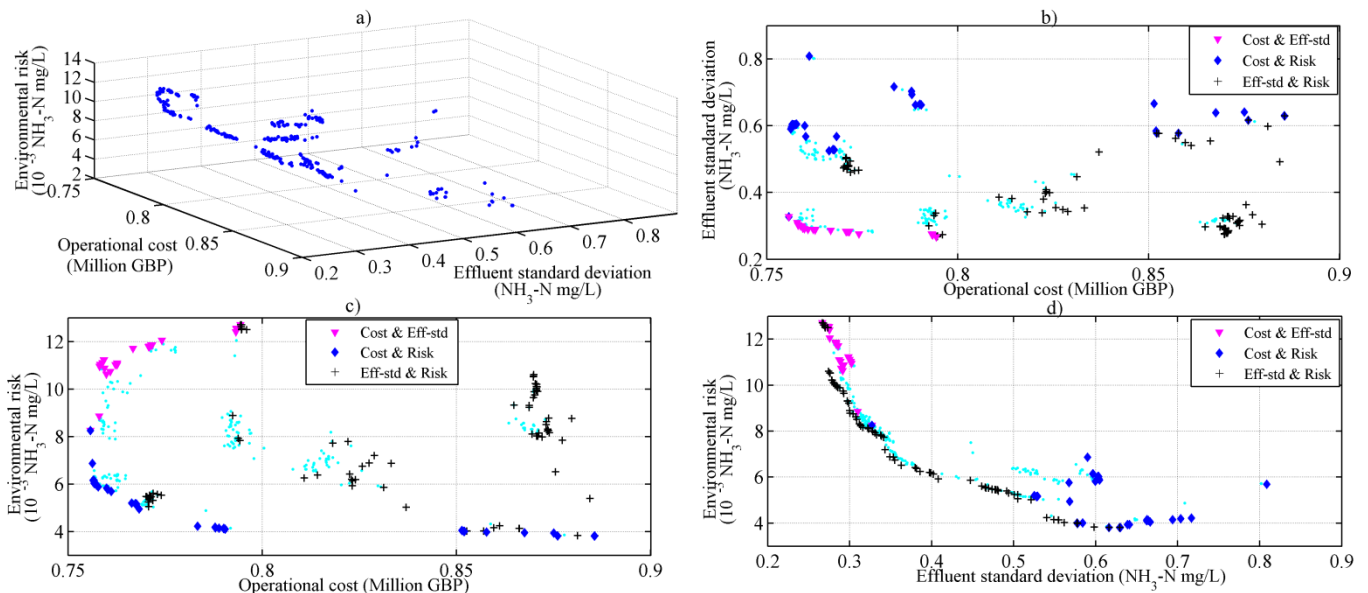


Figure 4.6 Non-dominated Pareto solutions using objectives of operational cost, effluent standard deviation and environmental risk in two- and three-dimensional space (Non-dominated solutions using two objectives are highlighted in different colours than cyan. Cost - operational cost, Eff-std - effluent standard deviation, and Risk - environmental risk)

Each solution on the curve corresponds to an operational strategy set (i.e. seven operational parameter values) and its associated performance. Compared to the baseline scenario results (cost: 0.82 Million £/year, effluent standard deviation: 2.01 $\text{NH}_3\text{-N}$ mg/L, environmental risk: 0.03 $\text{NH}_3\text{-N}$ mg/L), significant improvement is achieved in all three objectives by optimisation. This

agrees with the findings from previous research (Butler and Schütze, 2005; Fu et al., 2008) and demonstrates the advantage of operational optimisation, in particular from an integrated system perspective.

However, as observed from the optimal solutions, there is a trade-off between objectives, especially between effluent standard deviation and environmental risk (Figure 4.6d). High WWTP effluent stability is not only beneficial in itself, but could also improve the water quality status (90%ile value) of the receiving water. The most cost-effective way of achieving high effluent stability is by limiting inflows to the WWTP, compared with other measures such as enhanced aeration supply. Yet reducing WWTP inflow will lead to more overflows which raise the environmental risk. This is demonstrated by the solutions symbolised in magenta triangles which are highly optimised in cost and effluent stability but perform relatively poorer in environmental risk. Therefore, it is essential to use all three objectives for the optimisation so that no key aspect is neglected.

4.4.3 Solution Screening Using Visual Analytics

The screening process is primarily based on visual analytics to explore the complex trade-offs by successively adding more objectives into the trade-offs to aid the decision-maker in better capturing objective interactions and discovering high-performing solutions, which may not be fully captured in a lower-dimensional space (Fu et al., 2013). Other indicators proposed in the first step can also be used if additional information is provided. Colour designation facilitates the screening process by presenting results in an informative way and recording the decision-makers' preferences during the process. Below is an example of how screening is conducted.

- The process started from the trade-off graph between effluent standard deviation and environmental risk as shown in Figure 4.7a. Two cut-off lines were drawn to screen out solutions at both ends coloured in cyan. The top left group of solutions has relatively high environmental risk, while the solutions at bottom right have high standard deviation (i.e. low stability) in effluent discharge without improvement in risk reduction.
- In Figure 4.7b, solutions were projected against risk and a third objective of operational cost, and the screening information in Figure 4.7a was retained

by keeping the colour of the solutions. Cost-effective solutions achieving low environmental risk with reasonably low cost were selected from the chosen solutions from Figure 4.7a and were highlighted in green (they were in blue in Figure 4.7a). Thus the colour of solutions in the current figure is the combination of screening results of the current and previous steps.

- A fourth objective total pollutant discharge load was used in Figure 4.7c to select solutions with low discharge load from the UWWS and the high performing solutions retained were highlighted in magenta.
- A fifth objective, e.g. river standard deviation (Figure 4.7d), river 90%ile quality and river 99%ile quality, was also tested for screening but no additional information was provided, i.e. no solutions were screened out from the high performance solution set. Thus solutions highlighted in magenta are the final selected solutions, which will be used to derive operational strategy-based permits.

The indicators used for screening and the definition of threshold lines are typically determined by regulators negotiated with other stakeholders. In addition to the interactive nature, the screening can also be a cyclic process as preferences may change affected by results in the next screening step.

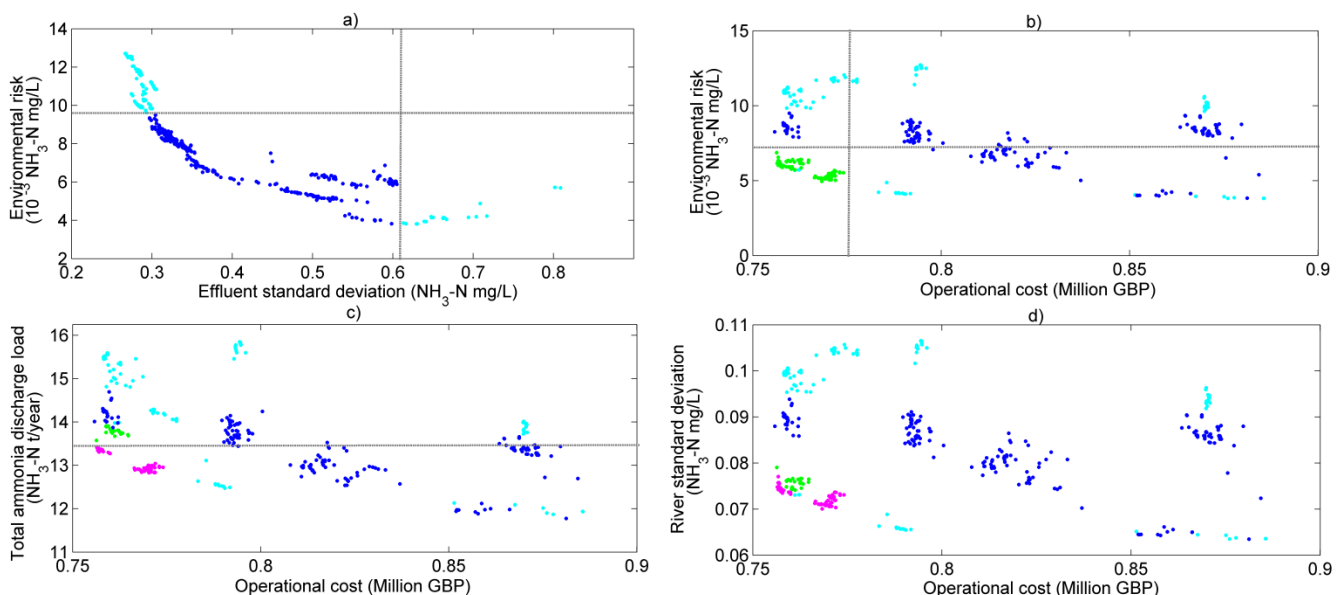


Figure 4.7 Screening of the Pareto optimal solutions through visual analytics (high performing solutions selected in a) to c) are highlighted in blue, green and magenta, respectively)

4.4.4 Permit Derivation Based on High Performing Solutions

Figure 4.8 shows operational variable values and corresponding performance of the Pareto optimal solutions (solid lines, with high performing solutions selected from section 4.4.3 highlighted in magenta) and the baseline case (black dashed line). Values are normalised and the minimum and maximum values are shown at the bottom and top of Figure 4.8. The return sludge pumping rate and waste sludge rate have been highly modified through optimisation, indicating sub-optimal settings during sludge-related operation is a main reason for the poor performance of the baseline case. Despite the highly optimised sludge pumping rates, the optimal solutions display remarkable diversity in other operational settings (reflected in the range of setting values), so does the system performance. However, the high performing solutions selected through screening are very similar in both operation and performance and are divided into two groups. Group 'A' solutions have lower cost than group 'B' but at the expense of lower effluent stability and higher environmental risk. A single solution from the high performing solution set can be chosen for permitting, but to allow for flexibility in practice, the feasibility of using value ranges based on one group is investigated.

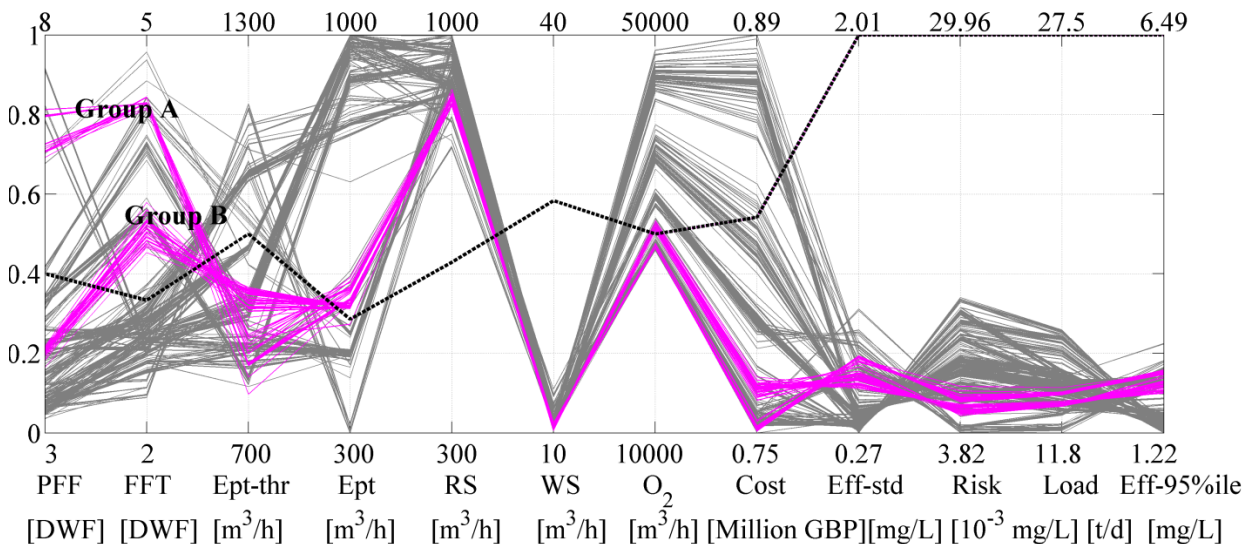


Figure 4.8 Values of operational variable settings, performance indicators and effluent 95%ile concentration of the Pareto optimal solutions (in grey), selected high performing solutions (in magenta) by the screening process and the baseline operational strategy (in black) (operational variables: PFF - pass forward flow, FFT - flow to full treatment, Ept-thr - storm tank emptying

threshold, Ept - storm tank emptying rate, RS - return sludge rate, WS - waste sludge rate and O₂ - aeration rate, and performance indicators: Cost - operational cost, Eff-std - effluent standard deviation, Risk - environmental risk and Load - total pollutant discharge load)

Group 'A' is used here to explain the permit derivation process. Based on the 34 solutions in the group, the minimum and maximum values of the seven operational variables and the five performance indicators are used as boundaries of 12 value ranges. LHS is performed to generate 20,000 operational scenarios within the seven operational value ranges, and the generated operational strategies are evaluated in SIMBA6 to estimate the confidence level of reliable performance if the system operates following the prescribed ranges. Results show that 89% of the 20,000 samples have effluent 95%ile values within the expected range, and the number is 71% if the other four performance indicators are also considered. Considering the high confidence level, the seven operational value ranges based on the 34 selected solutions can be used for permitting. However, if the confidence level is low, the operational ranges can be narrowed and the LHS re-run until an acceptable level of certainty is achieved.

Table 4.6 shows the proposed permit for the investigated case based on the 34 high performing solutions. It includes a set of operational values (taken as average values for illustration purposes) and corresponding ranges set for flexibility.

Table 4.6 Proposed form of operational strategy-based permit

Operational variables	Permit value	Permit range
Pass forward flow (dry weather flow, i.e. DWF)	6.7	[6.4, 7.1]
Flow to full treatment (DWF)	4.4	[4.4, 4.5]
Storm tank emptying threshold (m ³ /d)	19,700	[18,800, 20,600]
Storm tank emptying rate (m ³ /d)	12,700	[11,800, 13,800]
Return sludge pumping rate (m ³ /d)	21,100	[21,000, 21,400]
Waste sludge pumping rate (m ³ /d)	257	[254, 259]
Aeration rate (m ³ /d)	691,200	[685,752, 696,936]

4.5 Discussion

4.5.1 Performance of Operational Strategy-Based Permitting in Comparison with Traditional Approach

To compare with the traditional end-of-pipe permitting approach, a 95%ile permit is derived for this case using the stochastic permitting model RQP. In the baseline scenario, the river water quality at reach 9 after receiving all intermittent wastewater discharges (90%ile: 0.09 NH₃-N mg/L, 99%ile: 0.63 NH₃-N mg/L) complies with the environmental standards, thus no change in the design or operation of the storage and storm tanks needs to be made under the current regulation in England and Wales. Based on the upstream river condition at river reach 9 and WWTP effluent discharge characteristics under the baseline scenario, the derived permit is 1.42 NH₃-N mg/L, which is stricter than the 95%ile values of the operational strategy-based permitting solutions shown in Table 4.6. An experiment is designed, as described below, to investigate whether the tighter 95%ile limit leads to more environmentally protective and/or cost-effective results.

A 10,000-shot LHS was performed to search for compliant operational strategy solutions to achieve the 95%ile permit. Only operational settings in the WWTP were varied in the LHS, while keeping the PFF and FFT settings at the baseline values (i.e. 5DWF and 3DWF). Various combinations of operational settings in the WWTP were found to produce 95%ile values lower than the required level. The value ranges of effluent 95%ile total ammonia concentration as well as results in four performance indicators are presented in Table 4.7. Although effluent standard deviation of the compliant solutions is lower than the operational strategy-based permitting solutions, environmental risk (measured by indicators 'environmental risk' and 'total discharge load') is much higher due to increased overflow caused by lower PFF and FFT settings. Moreover, operational cost of the compliant solutions can be 19% more.

The performance of the LHS samples used for confidence assessment in section 4.3.4 is also shown in Table 4.7 for comparison. Only slight deviation in performance from that of the operational strategy-based permitting solutions is observed (Table 4.6). By contrast, the end-of-pipe permit solutions behave in a diverse manner in operational cost and environmental risk. Hence, despite the

effectiveness in restricting WWTP effluent discharge quality, the end-of-pipe permitting approach is insufficient in controlling other aspects of system behaviour compared to regulation on operation. Faced by the complex environmental challenges and the pursuit of cost-effectiveness, a more stringent regulation by traditional permitting approach may produce undesirable outcomes.

Table 4.7 Comparison of performance by the proposed operational strategy-based permitting approach and the traditional end-of-pipe method

Performance indicator	Operational strategy-based permitting solutions	20,000 LHS samples	End-of-pipe permit compliant solutions
Effluent 95%ile concentration (NH ₃ -N mg/L)	[1.99, 2.06]	[1.96, 2.10]	[1.23, 1.42]
Total operational cost (Million £/year)	[0.75, 0.76]	[0.75, 0.76]	[0.75, 0.90]
Effluent standard deviation (NH ₃ -N mg/L)	[0.58, 0.61]	[0.56, 0.63]	[0.27, 0.35]
Environmental risk (10 ⁻³ NH ₃ -N mg/L)	[5.83, 6.56]	[5.75, 6.59]	[8.34, 11.96]
Total discharge load (NH ₃ -N t/year)	[13.3, 13.4]	[13.2, 13.5]	[12.9, 14.5]

4.5.2 Reliability of the Operational Strategy-Based Permitting Approach

By permitting on operational strategies based on modelled system performance, the success of the newly developed approach relies on a) accuracy of an integrated UWWS model in representing the real world system, and b) good performance of the optimised operational strategies under future environmental conditions. As all models are imperfect abstractions of reality, uncertainty in modelling should (if possible) be considered in model-based decision-making (Carter and White, 2012; McIntyre, 2004; Ragas et al., 2009; Refsgaard et al., 2007). For the employed integrated UWWS model, uncertainty in the model output can result from:

- imperfect knowledge in input data, e.g. the simplified diurnal patterns defined to describe the dynamic wastewater inflow and quality to the WWTP, and assumed seasonal wastewater temperature;
- model structure (i.e. incomplete or simplified description of the modelled process as compared to reality) and model parameter (not all parameters in the model are validated with real-life data);
- computer implementation of the model (e.g. numerical approximations, resolution in space and time); and
- inherent stochastic or chaotic nature of natural phenomena (e.g. rainfall), which is not predictable and is non-reducible by more studies.

Due to the intensive resource and time required to carry out a comprehensive uncertainty analysis, it is not conducted in this study. Yet even if the model can simulate the system accurately, the permitted operational strategies, optimised using a pre-defined input data set, may not be the best solutions for future conditions. This is especially so under the pressure of climate change and the increasingly onerous environmental water quality condition. Hence, another input data set ('B') was used to examine the performance of the permitted operational strategies under a different environment. An extra advantage of using data set 'B' is that the rainfall is 26% more intensive than that of data set 'A' measured by the total rainfall depth and the upstream river condition is much poorer, thus can be deemed as a 'worse' scenario. Optimisation was run to find the optimal operational solutions with data set 'B'. Figure 4.9 shows the optimisation results as compared to the performance of the permitted solution (the strategy corresponding to the second column of Table 4.6) highlighted in a red square. Results show that the permit solution is not dominated by (i.e. no worse than) the optimal solutions and is outstanding in the performance of cost and environmental risk, however, its effluent standard deviation is higher than all optimal solution. This is caused by the heavier rainfall which adversely affects the wastewater treatment efficiency. In comparison, the optimal solutions have lower PFF and FFT settings thus protect the WWTP from overloading. Nevertheless, the permitted operational strategy provides reasonably good and reliable performance. To further ensure the robustness of the derived

operational strategy, more historical data sets should be applied if available or by using hypothetical data generated by stochastic experiments.

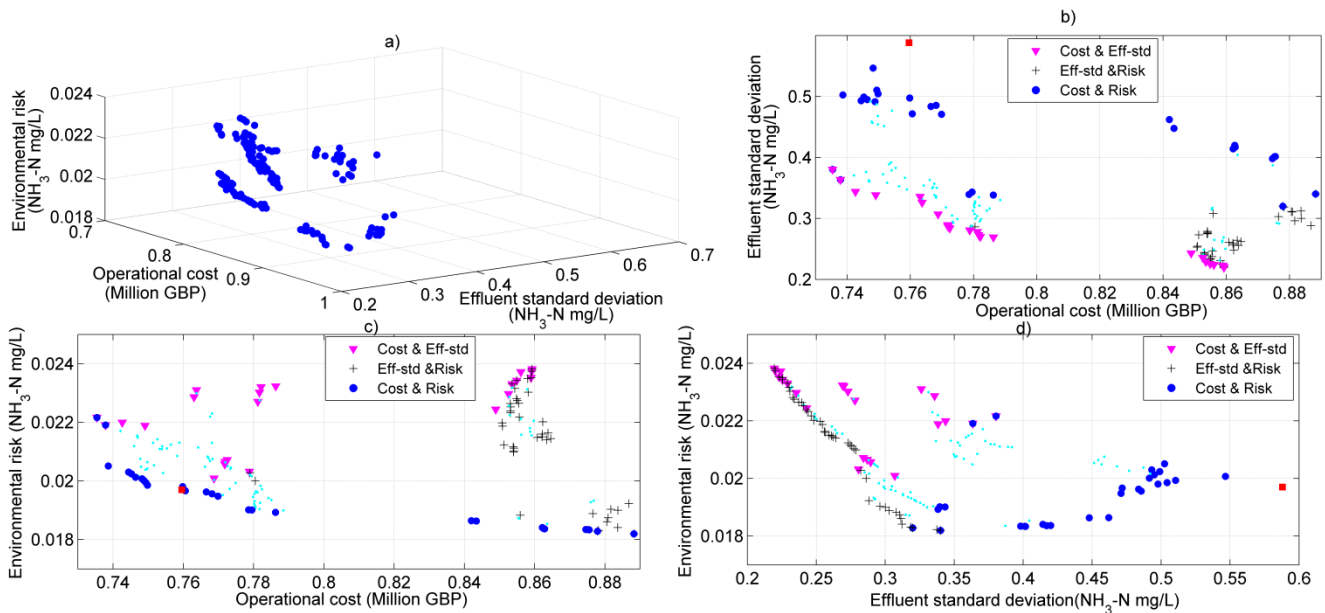


Figure 4.9 Performance of the permitted solution in Table 4.6 under data set 'B' (shown in red square) against non-dominated Pareto solutions optimised using data set 'B' with objectives of operational cost, effluent standard deviation and environmental risk in two- and three-dimensional space (Non-dominated solutions using two objectives are highlighted in different colours than cyan. Cost - operational cost, Eff-std - effluent standard deviation, and Risk - environmental risk)

4.5.3 A Win-Win Solution

By simulating behaviour of the regulated facilities, the integrated UWWS modelling enables regulators to have a better understanding of the economic and environmental impacts of the traditional end-of-pipe permitting approach. So, to respond to a more stringent 95%ile effluent permit, three compliance strategies are possible: a) increase treatment capacity (e.g. elevate the aeration rate, build a new reactor); b) discharge wastewater through other outlets which are weakly regulated and monitored; and c) implement an innovative technological solution. The first option often pushes up the cost, contradicting the interests of the regulated community as well as the aim of sustainable development. Neither is the second option desirable, as implied by the high environmental risk of the operational strategies with low overflow settings (e.g. low PFF and FFT) as presented in section 4.5.1. The third option is favourable

both to the regulators and the regulated parties. As demonstrated by this study and previous research, optimisation of operational strategy based on integrated modelling is, among others, an innovative technological solution. It can achieve environmental quality objectives in a reliable and energy efficient way. In particular, it exploits the potential of the existing system without the need for capital investment in enlarging treatment capacity.

Besides technological innovation, good regulation is also essential for effective risk management. Although a more stringent 95%ile permit can be achieved by a range of operational strategies, the solutions can be of higher environmental risk than other options that produce lower effluent quality. End-of-pipe quality has been used as a surrogate indicator of UWS performance, but is only valid if all discharges in the system are well monitored and controlled. Given the common situation of ineffective control on intermittent spills (e.g. CSOs, storm tank overflows), limited success could be achieved (at least cost-effectively) by over-tightening end-of-pipe limits of WWTP effluent discharges. However, if the end-of-pipe regulation is removed, more environmentally protective operational solutions are achievable. The proposed operational strategy-based regulation approach is an attempt to move away from restrictive and conservative 'outcome-based' permitting to more flexible and responsive 'performance-based' permitting, based on a fuller understanding of the system as a whole.

4.6 Conclusions

An operational strategy-based permitting approach was introduced in this chapter. Results from a case study demonstrate that environmental water quality, operational cost (also an indicator of GHG emissions) and treatment process stability can be simultaneously enhanced by an integrated operational strategy without violating the environmental standards in the UK. To achieve balanced benefits, it is necessary to consider interests of all stakeholders and incorporate the conflicting ones as optimisation objectives of operational strategies. Yet, the potential advantages achievable are likely to abate under the current regulation paradigm, as the WWTP effluent water quality or the number of overflow spills derived by the traditional end-of-pipe permitting approach may not be met by the optimal operational strategies if the overall

impact to environmental water quality, operational cost and/or variability of treatment efficiency is lower.

The newly developed permitting approach is a good complement to the traditional approach in achieving better and balanced system performance. Moreover, it ensures informed and transparent decision-making with stakeholder input at all points in the permitting process, and fits into wider environmental management strategies such as the US Watershed Management Program and European River Basin Management Plan. The four-step decision-making framework, established by using NSGA-II, visualisation tool and LHS, facilitate the complex optimisation and decision-making process. Although the research is based on a semi-hypothetical case with a single pollutant and neglected uncertainty in modelling, it is suggested that the method and at least some of the findings can be generalised for regulatory decision-making.

5 Real Time Control-Based Permitting

5.1 Introduction

Chapter 4 has highlighted the benefits of optimising an integrated operational strategy of UWWs in perspectives of reducing operational cost, variability of wastewater treatment efficiency and environmental risk. However, the operational settings are fixed (except the emptying of the storm tank) against a dynamic environment in the optimisation. This means a balance needs to be sought between maintaining system performance in the worst conditions and not entailing excessive cost. One way to address this is to implement real-time control strategies so that the operation of the system can be varied in response to the environmental change (e.g. assimilation capacity of the receiving water, rainfall, and wastewater temperature) in real-time. Indeed, integrated RTC of UWWs has been identified as a promising approach to improve the receiving water quality, increase levels of service and enhance sustainability (Butler and Schütze, 2005).

The benefits of real-time control of the UWWs in accordance to the receiving river water quality have already been reported in literature (Langeveld et al., 2013; Meirlaen, 2002; Rauch and Harremoës, 1999b; Schütze et al., 2002). However, the previous research was mainly focused on improving environmental water quality, while cost was often neglected in the development of RTC strategies. Driven by the increasingly stringent wastewater discharge permits and the pursuit of cost-effectiveness, there is a growing interest in applying RTC technologies to reduce operational cost by exploiting environmental capacity without violating the permit (Gardner et al., 2010). However, as suggested by Gardner et al. (2010), the benefits gained through the adoption of the technologies will be negated as a new more stringent permit is likely to be set under the current UK permitting regulation due to changes in the operating performance. Yet, as recognised by the report, this finding was made based on simple stochastic models without simulating the RTC process, thus further studies based on mechanistic models are needed to validate the argument.

To fill the gap in research, this chapter aims to explore the potential benefits of implementing RTC strategies to the integrated UWWs in terms of both

operational cost and environmental water quality, and to investigate a reasonable approach of permitting to exploit the full potential of the RTC technology. Four questions will be addressed:

- a) How to explore integrated RTC strategies to maximise cost savings and environmental outcomes?
- b) What are the advantages and disadvantages of applying integrated RTC strategies to wastewater treatment processes?
- c) What is the potential form of permitting to best deliver the benefits brought by the RTC technology?
- d) What are the advantages and disadvantages of the proposed form of permitting compared to the traditional approach?

An integrated RTC-based permitting approach is proposed in this study. The methodology, as described in section 5.2, is similar to that for the proposed operational strategy-based permitting approach in that it is based on optimal control strategies and similar optimisation technique and permit derivation method are used. Descriptions on these techniques are therefore not repeated whilst more details are given on the development of an integrated RTC framework. The proposed RTC-based permitting approach is applied to the same case study used in Chapter 4. Implications of the RTC technology and the proposed permitting approach are discussed in section 5.4.

5.2 An Integrated RTC-Based Permitting Framework

A three step decision-making framework (Figure 5.1) was proposed for the integrated RTC-based permitting as described below.

Step I: Development of an integrated RTC strategy framework.

As mentioned in section 3.2.1, actuators, sensors, wastewater treatment process dynamics and the controller units connecting the sensors and actuators constitute the basic elements of a control system. The development of an integrated RTC strategy framework refers to in this work the definition of actuators (e.g. pumps, valves and gates), sensors, structure of controllers (i.e. which actuators are controlled in accordance to measurement from which

sensor(s)) and controller algorithms (i.e. a set of rules specifying the time sequence of all set-points or control inputs in an RTC system).

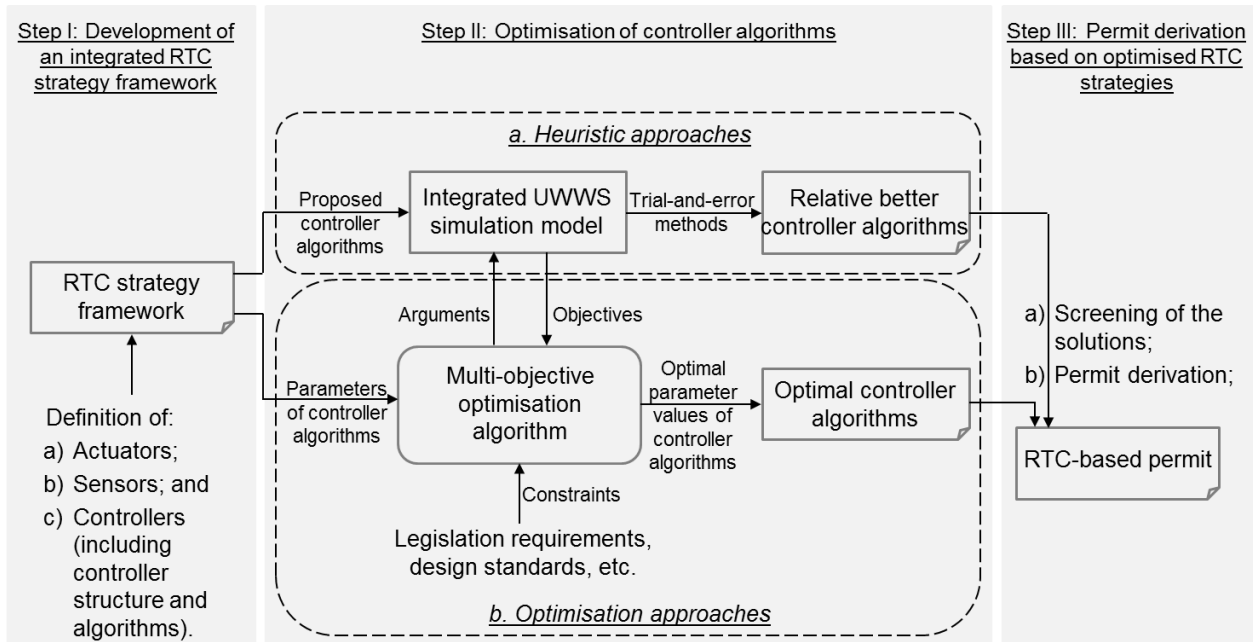


Figure 5.1 Decision-making framework for integrated RTC-based permitting

Examples of state variables monitored by sensors in the case study integrated UWWS are provided in Table 5.1, and variables manipulated by actuators are provided in Table 5.2.

Table 5.1 Examples of variables monitored by sensors in the case study integrated UWWS

Category	Example
Flow rate	a) Wastewater inflow rate to the WWTP; and b) River flow rate
Water level	Water levels at storage/storm tanks
Physical, chemical or biological variables in treatment units and rivers	a) Wastewater temperature; b) DO and ammonia concentrations in the aeration tank; c) Mixed liquor suspended solids (MLSS) concentration in the aeration tank; and d) DO and ammonia concentrations in the river
Rainfall	Rainfall in the catchment

Table 5.2 Examples of variables manipulated by actuators in the case study integrated UWWS

Category	Example
Wastewater flow rate	Storm tank emptying rate
Sludge flow rate	a) Return sludge flow rate; and b) Waste sludge flow rate
Compressor speed or air valve opening	Air flow rate

The determination of the controller structure can be difficult. For example, assuming each of the x actuators can be controlled according to information from each of the y sensors, a total of x^y possible structures exist for a hierarchical control. Despite a few studies to derive the structure in an automatic way, e.g. a self-learning expert system was applied by Almeida and Schilling (1993) to optimise the structures for individual events, heuristic approaches by experience and/or trial-and-error techniques are a more widely practised method. In this work, the controller structure is determined by experience in accordance with the objectives of the RTC scheme and case-specific needs and conditions.

Model-based predictive control is applied to the integrated RTC in this work. If-then rules are used as the controller algorithm. The if-then rules allow the performance of control actions defined in the consequence (i.e. 'then') statement based on criteria in the conditional (i.e. 'if') statement. Values of parameters and control variables in the rules are determined by offline optimisation because: a) the integrated UWWS model used for the control is highly non-linear and complex; and b) multiple objectives are used in the optimisation of the control. The optimisation of the controller algorithm is conducted in step II.

Step II: Optimisation of controller algorithms.

After the establishment of the strategy framework in step I, the numeric parameters in the controller algorithm (if-then rules) are quantified towards maximising pre-defined objectives. This can be performed by heuristic

approaches or optimisation tools as illustrated in Figure 5.1. Both approaches need integrated UWWS models to evaluate the performance of the potential solutions by calculating the objectives based on simulation results. To improve the computational efficiency, a combined approach is adopted in this work. That is parameters in the condition of the rules are determined by heuristic method, while the control variables in the consequence statement are quantified by optimisation tools.

Step III: Permit derivation based on optimised RTC strategies.

As multiple objectives are used for the optimisation in step II, more than one integrated RTC strategy solution (refers to integrated RTC framework with quantified parameters in the controller algorithm) would be produced. Hence, a screening procedure, similar to that in section 4.3.3, is employed to screen high performance RTC solutions in accordance to site-specific needs and decision-makers' preference. Permit is derived based on the selected RTC strategy solutions.

5.3 Case Study

5.3.1 Development of an Integrated RTC Strategy Framework

In the studied case, there are five key actuators (one valve, three pumps and one blower) controlling the emptying rate of the storm tank, air flow rate, return sludge flow rate and waste sludge flow rate in the WWTP. In this study, air flow rate is the only controlled variable while the other four manipulated variables are set to the optimised values obtained using NSGA-II in Chapter 4. Aeration control is chosen because it is found to be the most influencing factor in terms of operational cost and water quality as revealed in a sensitivity analysis to be described in section 6.3.1. Return sludge rate and waste sludge rate, also showing great impact (Figure 6.6), are not selected due to the long response time (in days to weeks) to operational changes thus are considered unsuitable for the RTC application.

The number of monitored variables can be numerous as sensors can in theory be placed at almost any position of the integrated UWWS. This work focuses on monitoring of physical parameters, as the sensors are easier to install and maintain and generally cost less than that for water quality and biological

parameters. Three variables are selected for the case study, which are upstream river flow rate (at river reach 2, before all wastewater discharge points), wastewater inflow rate to the WWTP and wastewater temperature. River flow rate is chosen as an indicator of river assimilation capacity, and the latter two represent critical factors influencing wastewater treatment efficiency.

Following the selection of the controlled and monitored variables, the control strategy is structured as “if-then” rules or scenarios as shown below:

“If river flow rate $\geq a$, wastewater inflow rate $\geq b$ and temperature $\geq c$, then aeration rate = x . “

The number of rules or scenarios depends on how the value of each monitored variable is classified. For example, if the river flow rate is classified into three classes ‘low’, ‘medium’ and ‘high’, and wastewater inflow rate and temperature both into two classes ‘low’ and ‘high’, there would be $3 \times 3 \times 2 = 18$ scenarios. For simplicity, the value of each monitored variable is classified into only ‘high’ and ‘low’ in this study. The threshold values 41,250 m³/d (equals to 1.5DWF), 15 °C and 300,000 m³/d (about 10DWF) were used to classify dry/wet weather, winter/non-winter time, and low/high river flow. The three threshold values are determined by trial-and-error method facilitated by model simulation.

To improve the computational efficiency further, the eight aeration rate values for the control actions defined in the ‘if-then’ rules are simplified into three tiers X, Y and Z ($X < Y < Z$). By trial-and-error method, X, Y or Z is assigned to each of the eight scenarios based on model simulation. For example, in summer time and with no or light rainfall and high river flow rate (i.e. S2 in Table 5.3), the lowest aeration rate X would be enough, as the wastewater treatment efficiency is relatively high due to the higher temperature and lower loading to the treatment process and the assimilation capacity of the receiving water is higher. The if-then rules for the real-time aeration control are summarised in Table 5.3. The values of the three aeration tiers are to be optimised in the next step of the decision-making framework.

Table 5.3 RTC rules for aeration rate control in accordance to wastewater inflow rate, temperature and upstream river flow rate

Scenario	Wastewater inflow rate to the WWTP (m ³ /d)	Temperature (°C)	Upstream (reach 2) river flow rate (m ³ /d)	Aeration rate tier (m ³ /h)
S1	> 41,250	> 15	> 300,000	X
S2	<= 41,250	> 15	> 300,000	X
S3	> 41,250	<= 15	> 300,000	Z
S4	> 41,250	> 15	<= 300,000	Y
S5	<= 41,250	<= 15	> 300,000	X
S6	<= 41,250	> 15	<= 300,000	X
S7	> 41,250	<= 15	<= 300,000	Z
S8	<= 41,250	<= 15	<= 300,000	X

5.3.2 Optimisation of Integrated RTC Strategies

To examine the effect of RTC against fixed operation, only the aeration rate varies with the environmental changes, whilst keeping the other settings in the integrated UWWWS fixed and at the same values as optimised in Chapter 4 (i.e. second column of Table 4.6). The optimisation of the three aeration tiers is formulated in a similar way as in Chapter 4 though there are minor differences. For example, the number of the optimisation objectives is reduced to two as the values of the stability of the treatment process and environmental risk are found to be positively linearly related when only aeration rate is optimised. The generation number for the optimisation run by NSGA-II is larger than in Chapter 4 as no satisfactory results were found with 15 generations. Details of the simulation and optimisation modules in comparison with that of Chapter 4 are summarised in Table 5.4.

Table 5.4 Comparison of the definition of optimisation for operational strategy-based permitting and RTC-based permitting

Module	Parameter	Operational strategy-based permitting	RTC-based permitting
Simulation module	Model input		Same
	Configuration of the UWWS		Same
	Operational and control settings	Fixed operational settings	Aeration rate controlled by “if-then” rules as in Table 5.3, while other settings fixed at the same values as optimised in Chapter 4
	Objectives	<ol style="list-style-type: none"> 1) Total operational cost; 2) Stability of the treatment process; and 3) Environmental risk 	<ol style="list-style-type: none"> 1) Total operational cost; and 2) Environmental risk
Optimisation module	Optimisation variables	Three threshold settings and four manipulated variable values (Table 4.4)	Three aeration rate tiers
	Value ranges of the optimisation variables		Same
	Constraints		Same
	Optimisation algorithm settings	<ol style="list-style-type: none"> 1) Population number: 100; 2) Generation number: 15; and 3) Four batches 	<ol style="list-style-type: none"> 1) Population number: 100; 2) Generation number: 50; and 3) Four batches

The non-dominated optimal results from the four-batch optimisation are shown in black dots against the two objectives in Figure 5.2 (each dot represents one RTC strategy). The results of the fixed aeration setting strategy are also plotted in the figure (marked by a red square) for comparison. Despite the much larger generation size for the optimisation than that in Chapter 4, none of the Pareto optimal solutions dominate the fixed operational solution. This result is somewhat counterintuitive, thus the formulation of the integrated RTC framework and the optimisation was checked first before drawing any conclusion. The poor performance of the optimised RTC strategies can be caused by a number of reasons, for example:

- a) The computational efficiency is low, thus further optimisation is needed;
- b) There are no benefits in applying the defined form of RTC strategies;
- c) The monitored variables used for the control of aeration are inappropriate and need to be re-selected;
- d) The threshold values for the monitored variables in the conditions of the if-then rules are not reasonable and need to be changed;
- e) The number of scenarios and/or aeration rate tiers need to be modified; and
- f) The assignment of aeration tier to each scenario should be adjusted.

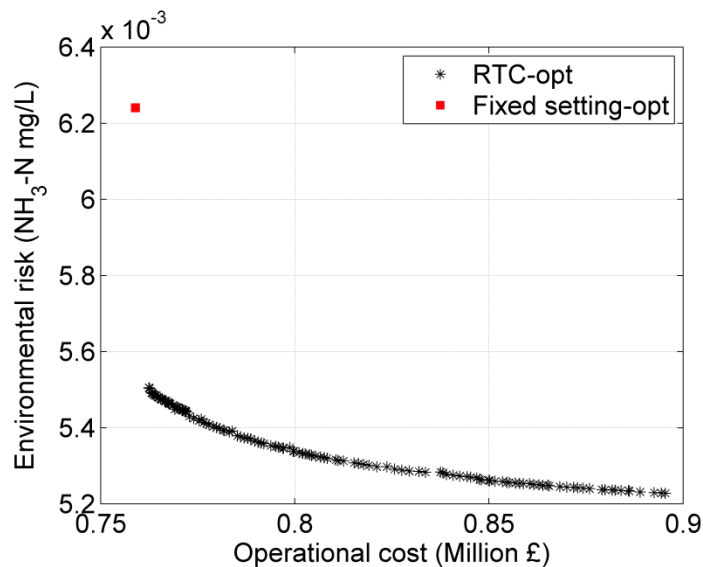


Figure 5.2 Optimised RTC strategy solutions in comparison with the optimised fixed operation solution against the objectives of operational cost and environmental risk

As a much higher generation number and smaller number of optimisation variables and objectives have been used for the optimisation than in Chapter 4, further optimisation does not seem likely to produce better results at least in short term. The argument 'b)' should be justified with more experiments, at least by investigating arguments 'c)' to 'f)' first, before any final conclusion can be made. It is straight-forward to address arguments 'c)' and 'd)' by choosing other variables and monitored variables for the optimisation. In comparison, the methods to examine 'e)' and 'f)' are not straightforward. For example, it is unclear whether the number of scenarios should be increased or reduced. An increased number of scenarios should in theory lead to more cost-effective solutions with more accurate air supply to satisfy the demand. However, the advantage could not be delivered if optimal solutions cannot be efficiently found. Thus the optimisation results are analysed first.

Figure 5.3 shows the variable values of the optimal solutions in Figure 5.2 (from left to right). Each solution corresponds with one set of X, Y and Z values. As shown in the figure, the values of Y and Z for most solutions are close, suggesting only two aeration tiers could be sufficiently enough.

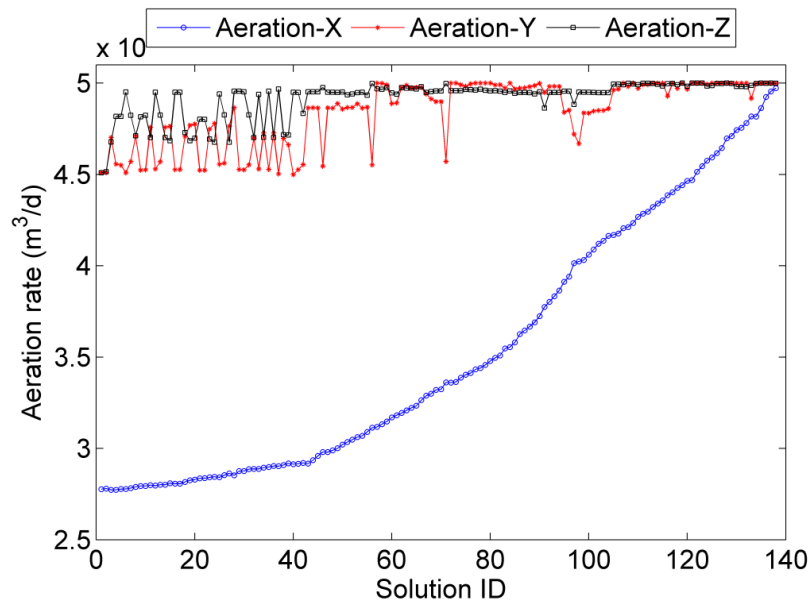


Figure 5.3 Operational variable values of the optimised RTC solutions with three aeration tiers

By using two aeration tiers (i.e. X and Y values of the optimal solutions), the argument 'f', i.e. suitability of the assignment of the aeration tier for each scenario, is tested. As it is certain to assign Y to the 'worst' environmental

condition and to assign air flow X to the 'best', S2 and S7 need not to be examined. For the rest of the scenarios, the assignment of aeration tier is tested by changing it to the alternative option (i.e. from X to Y, or Y/Z to X) and checking if great improvement in system performance can be achieved. The changes in the two objectives are presented in Figure 5.4. By altering the aeration tier from Z/Y to X for S3 and S4, cost reduction can be achieved but with a disproportionate increase in risk. Similarly, disproportional cost is increased if the aeration tier X is changed to Y for S5, S6 and S8. It is uncertain however of whether the aeration tier for S1 needs to be changed from the produced results. The slope of the curve suggests more percentage of risk can be reduced by a lower percentage of cost increase. Nevertheless, the rule is not changed because a) the amount of change is marginal and b) the reduction in operational cost is harder to achieve for this case compared to the environmental risk. Note that if the aeration tier of S1 is changed, the framework of the RTC strategies will be altered, as the condition of river flow rate will be redundant for the "if-then" rules. Therefore, the suitability of parameters selected for the RTC rule conditions can be checked through the optimisation of the controlled variable values.

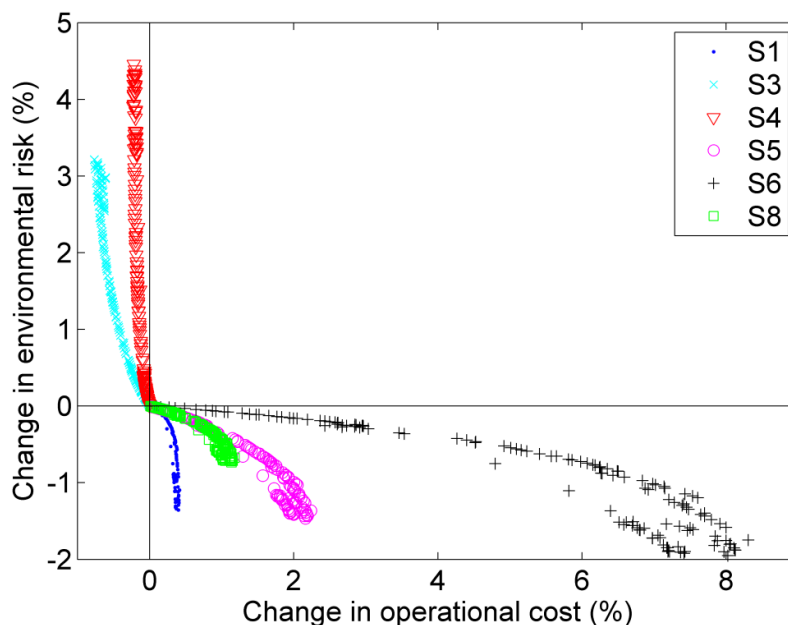


Figure 5.4 Changes in operational cost and environmental risk by varying aeration tiers from X to Y or Y/Z to X of S1, S2-6 and S8

After the series of test, the optimisation is re-run with the same framework in Table 5.3 but changing tier "Z" to "Y" to examine if better results can be

produced. As the number of optimisation variables is reduced to two, LHS instead of NSGA-II is employed for the search of better solutions as it can be more efficient than heuristic search algorithms in finding satisfactory results in 2-D solution spaces. A 5000-shot LHS was run and results are shown in cyan dots in Figure 5.5 as compared to the previous optimisation results (black asterisks) and fixed operation solution (red square) in Figure 5.2 . Among the 5000 samples, 1153 of them dominate the fixed setting solution and are highlighted in blue dots in the figure. The operational cost can be reduced by 4% and with the same level of environmental risk as compared to the fixed aeration solution; or the effluent standard deviation can be decreased by 11% entailing no more cost. The improved solutions demonstrate the benefits of adopting RTC strategies for aeration operation for this system.

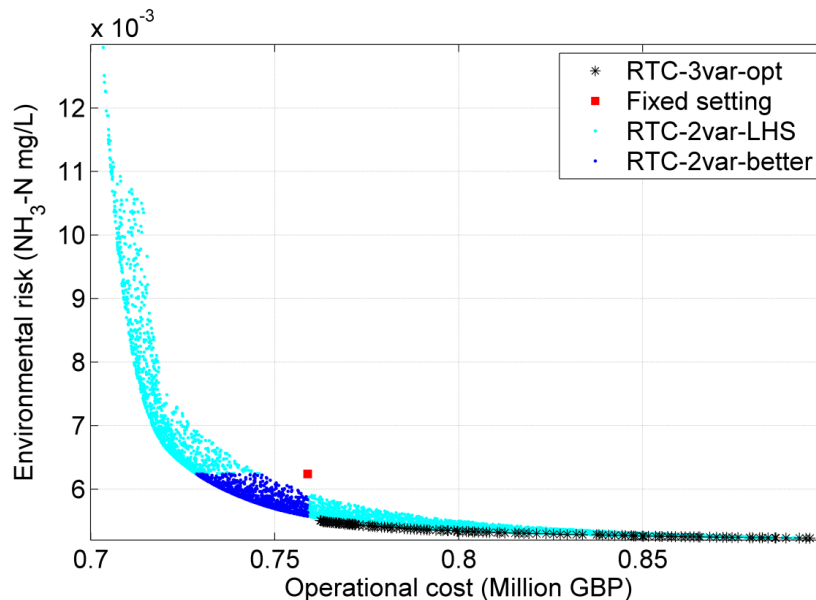


Figure 5.5 RTC strategies (with two aeration tiers) generated by a 5000-shot LHS as compared to RTC strategies (with three aeration tiers) optimised by NSGA-II and the optimal fixed operation solution produced in Chapter 4 (two aeration tiered RTC solutions dominating the fixed setting strategy are highlighted in blue)

5.3.3 Permit Derivation Based on Optimal RTC Strategies

To enable RTC strategies-based permitting, the variable values of the better performing strategies (than the fixed operation solution) are examined first to find out if any common features exist. The control variable values and corresponding performance of the 1153 dominating RTC strategies are shown

in grey lines in Figure 5.6, with values normalised and the minimum and maximum values shown at the bottom and top of the figure. It can be seen that the values of the higher aeration tier ('Y) vary greatly, so as that of the performance indicators. Thus a screening procedure similar to that applied in Chapter 4 is employed to select high performing solutions.

- Figure 5.7a shows the solutions to be screened, which are the 1153 RTC strategies dominating the fixed operation solution.
- In Figure 5.7b, solutions were projected against operational cost and a third objective downstream river 90%ile total ammonia concentration. Cost-effective solutions achieving lower river 90%ile value with reduced cost than the fixed operation scenario were selected and highlighted in green.
- A fourth objective downstream 99%ile total ammonia concentration was used in Figure 5.7c. It can be seen that all solutions perform much better than the fixed operation strategy in the fourth objective, and no solutions can be clearly screened out in this step.
- A fifth objective total pollutant discharge load was used in Figure 5.7d to select solutions with low discharge load from the UWWS. Solutions with lower discharge load than fixed operation solution were retained and highlighted in magenta.

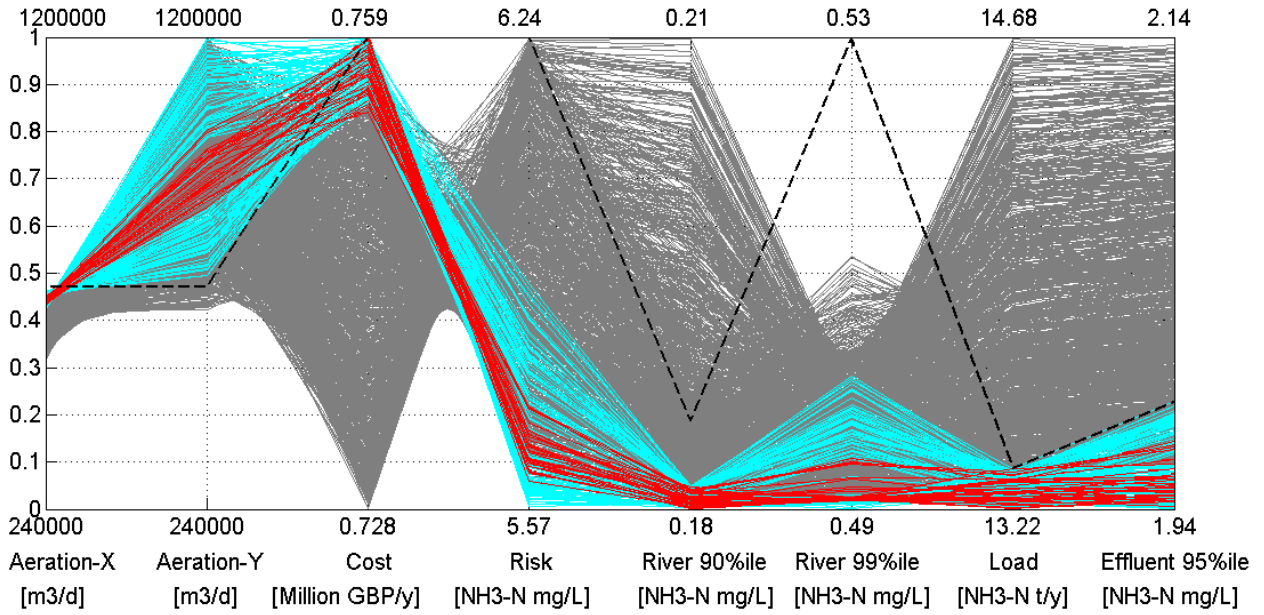


Figure 5.6 Values of aeration tiers and performance indicators of 1153 RTC strategies (in grey) generated by LHS dominating the fixed operational setting solution, 152 high performing solutions (in cyan) selected by the screening process as illustrated in Figure 5.7, and 30 optimal solutions (in red) used for permitting

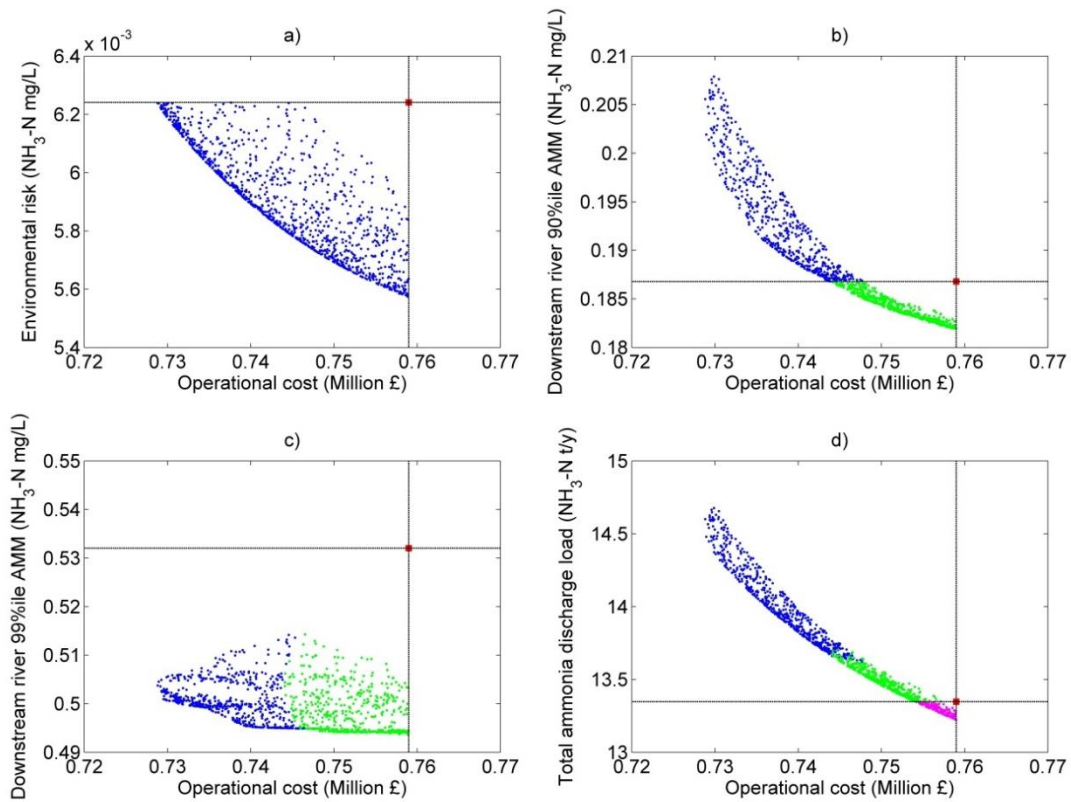


Figure 5.7 Screening of high performing RTC strategies from solutions produced in section 5.3.2 (high performing solutions selected in b) and d) are highlighted in green and magenta, respectively)

The 152 selected solutions by the screening procedure are marked in Figure 5.6 in cyan lines. As the selected solutions still perform diversely in some indicators, the 30 Pareto solutions that are relatively better in terms of cost and discharge pollutant load as highlighted in red in Figure 5.6 are used for permitting.

Similar to the permit derivation procedure in section 4.4.4, the boundary values of the aeration tiers of the 30 selected solutions are first identified, which are [653,000 m³/d, 673,400 m³/d] and [855,100 m³/d, 1,009,600 m³/d] respectively. As there are only two variables, it would not be necessary to employ LHS to characterise the performance of control solutions generated within the two value ranges, but rather by running only two ‘extreme condition’ scenarios with minimum and maximum aeration tier values. Results of the two scenarios are presented in Table 5.5 as compared to the performance of the fixed operation solution.

Table 5.5 Comparison of results of scenarios with minimum, maximum, average and lowest cost aeration tier values and the fixed optimal operation solution

	Minimum aeration solution	Maximum aeration solution	Average aeration solution	Lowest cost solution	Fixed operation solution
Aeration X (m ³ /d)	653,000	673,400	664,600	653,000	691,200
Aeration Y (m ³ /d)	855,100	1,009,600	928,500	955,100	691,200
Operational cost (Million £/year)	0.753	0.760	0.757	0.754	0.759
Environmental risk (10 ⁻³ NH ₃ -N mg/L)	5.75	5.59	5.66	5.67	6.24
Downstream 90%ile concentration (NH ₃ -N mg/L)	0.18	0.18	0.18	0.18	0.19
Downstream 99%ile concentration (NH ₃ -N mg/L)	0.50	0.49	0.49	0.49	0.53
Total discharge load (NH ₃ -N t/year)	13.39	13.20	13.28	13.35	13.35
Effluent 95%ile concentration (NH ₃ -N mg/L)	1.98	1.94	1.96	1.97	1.99

As shown in Table 5.5, the scenarios with minimum and maximum aeration rates are no better than the fixed operation solution, as the former one causes more pollutant discharge load and the latter entails more operational cost. This indicates that solutions generated within the two value ranges of aeration rate do not necessarily dominate the fixed operation solution. Hence, the 30 RTC solutions should be further screened if permit is to be set by aeration value ranges as in section 4.4.4. Alternatively, a single solution can be selected as the permit. Two potential solutions are presented in Table 5.5, one based on average aeration tier rates and the other by the RTC strategy producing lowest cost among the 30 selected solutions. Table 5.6 shows the form of the RTC-based permitting based on the lowest cost solution.

Table 5.6 Proposed form of RTC-based permit based on the lowest cost solution

Operational variable	Permit value (m ³ /d)	Condition
Aeration rate	653,000	If WWTP influent flow rate \leq 41,250 m ³ /d; or WWTP influent flow rate $>$ 41,250 m ³ /d, wastewater temperature $>$ 15 °C and upstream river flow rate $>$ 300,000 m ³ /d
	955,100	Other conditions

5.4 Discussion

5.4.1 Benefits of Real-Time Control Technology

Figure 5.5 has presented the advantages of applying the proposed form of real-time aeration control strategy over the fixed operation solution in terms of reduction in energy cost and environmental risk. By examining more aspects of environmental impact, the screening procedure ensures, with a high confidence level, that the final selected solutions are more environmentally protective. Figure 5.8 shows the impact of the permitted real-time aeration strategy (Table 5.6) on effluent total ammonia concentration, which in turn influences river water quality. As shown in Figure 5.8c, the change in effluent water quality when applying 6% reduced aeration rate is hardly noticeable except in wet weather; but the environmental impact of the obviously worse effluent quality is marginal

as shown in Figure 5.8d due to the high dilution capacity of the receiving water. By supplying 38% more air flow rate in less favourable conditions, the spikes of effluent and downstream river total ammonia concentration are decreased.

Associated with the higher environmental benefits, however, is the diminished advantage in cost savings. The permitted RTC strategy in Table 5.6, which is the cheapest solution among the final selected strategies, costs only 0.6% less than the fixed operation solution. In comparison, the figure is 4%, as mentioned in section 5.3.2, for the lowest cost solution (aeration tier rates: 549,000 m³/d and 1,022,600 m³/d) among the strategies before the screening procedure. Figure 5.9 plots the river total ammonia concentration under the two RTC strategies as well as the fixed operation scenario. Though the latter RTC strategy entails less energy cost resulted from the 21% lower aeration rate during moderate conditions, it also leads to the worsening of the environmental water quality. Moreover, the improvement of total ammonia concentration in downstream river under adverse conditions is less effective despite a higher air flow rate is applied (48% more intensive than the fixed operation strategy). Hence, the proposed form of RTC strategies offers mainly environmental benefits. If cost savings are to be sought, compromise in environmental quality, in particular total ammonia discharge load, needs to be made.

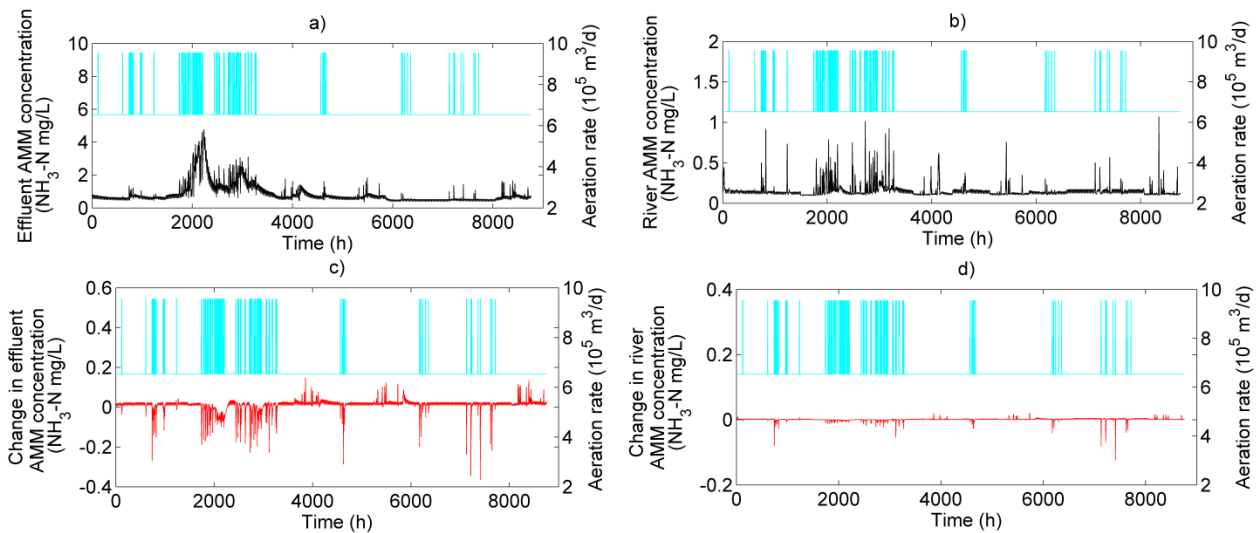


Figure 5.8 Time series of air flow rate (in cyan) under the RTC strategy in Table 5.6 with effluent (in a)) or downstream river total ammonia concentration (in b)) or the changes after applying the RTC strategy (in c) and d))

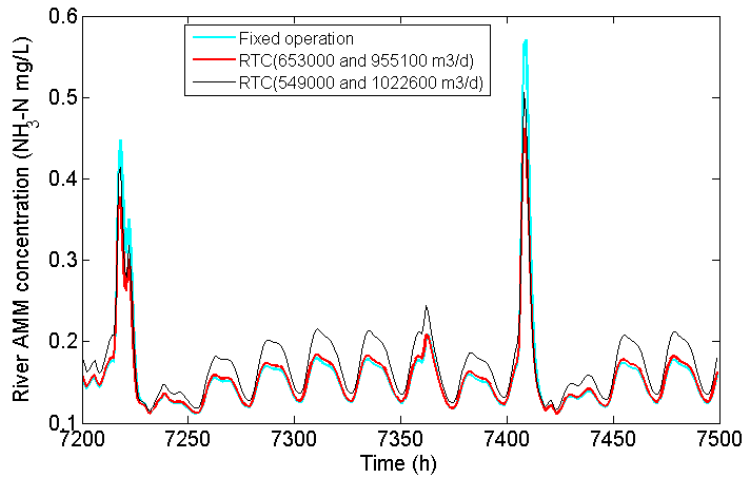


Figure 5.9 Comparison of time series of downstream river total ammonia concentration under fixed operational strategy and two RTC strategies

5.4.2 Need for Regulation on Control Strategy

The effluent 95%ile total ammonia concentration of the permitted RTC strategy (1.97 $\text{NH}_3\text{-N}$ mg/L) is only slightly smaller than the fixed operation solution (1.99 $\text{NH}_3\text{-N}$ mg/L) as shown in Table 5.5. Moreover, the probability distributions of effluent quality of the two scenarios are almost the same, especially by comparing with the change of the PDF from the baseline scenario (defined in section 4.3.3) to the optimal fixed operation scenario as plotted in Figure 5.10. However, their performance in operational cost and environmental risk are quite different. It is therefore essential to permit and regulate on the control strategies rather than an effluent 95%ile value to achieve the desirable system performance.

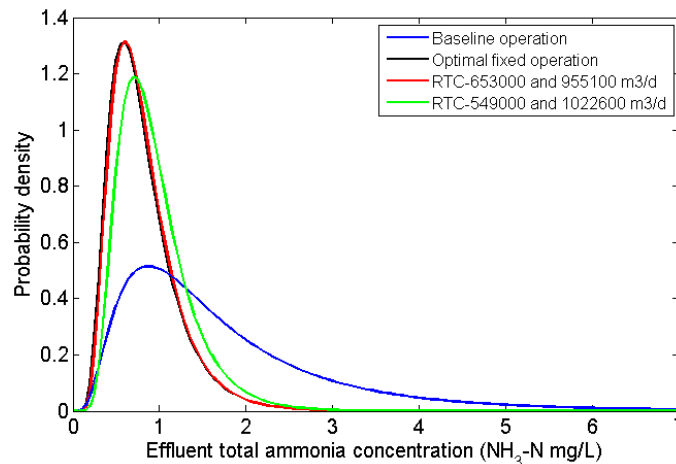


Figure 5.10 Comparison of PDFs of effluent total ammonia concentration under scenarios with baseline operation, optimal fixed operation and the two RTC strategies in Figure 5.9

The necessity of regulation of the system control is further explained by the example of the RTC strategy (aeration tier rates: 549,000 m³/d and 1,022,600 m³/d) that has a 3.4% lower cost than the permitted strategy. It can be used to simulate the behaviour of the WWSP to reduce energy cost by implementing real-time aeration control strategies. Though it may be perceived that insignificant relaxation of treatment effort under moderate conditions would not affect compliance of 95%ile permit as it is a measure of effluent quality in more extreme conditions, the effluent 95%ile value under this RTC strategy (2.14 NH₃-N mg/L) actually violates the permit limit (1.99 NH₃-N mg/L). The change in the PDF of effluent total ammonia concentration by this strategy is obvious as shown in Figure 5.10 in a green curve. Furthermore, this more cost-efficient RTC strategy also results in deterioration in river water quality as already mentioned in section 5.4.1. Hence, utilising RTC technology under the traditional permitting regime may offer no benefits to either the WWSP or to the environment, which corresponds well with the finding in Gardner et al. (2010). Under the proposed RTC-based permitting approach, more rational decisions, which are either more stringent environmentally protective or more cost-effective, can be made based on the full appraisal of the consequence of the compliance strategies.

5.4.3 Reliability of the RTC-Based Permitting Approach

Similar to section 4.5.2, the permitted RTC strategy in Table 5.6 was applied to another data set of rainfall and river flow and water quality (i.e. data set 'B') to test the reliability of the permitting approach. The impact of the RTC strategy is presented in Figure 5.11. The improvement in effluent water quality is clear as shown in Figure 5.11c, however, the contribution to river water quality betterment is smaller compared to data set 'A' due to the higher upstream total ammonia concentration.

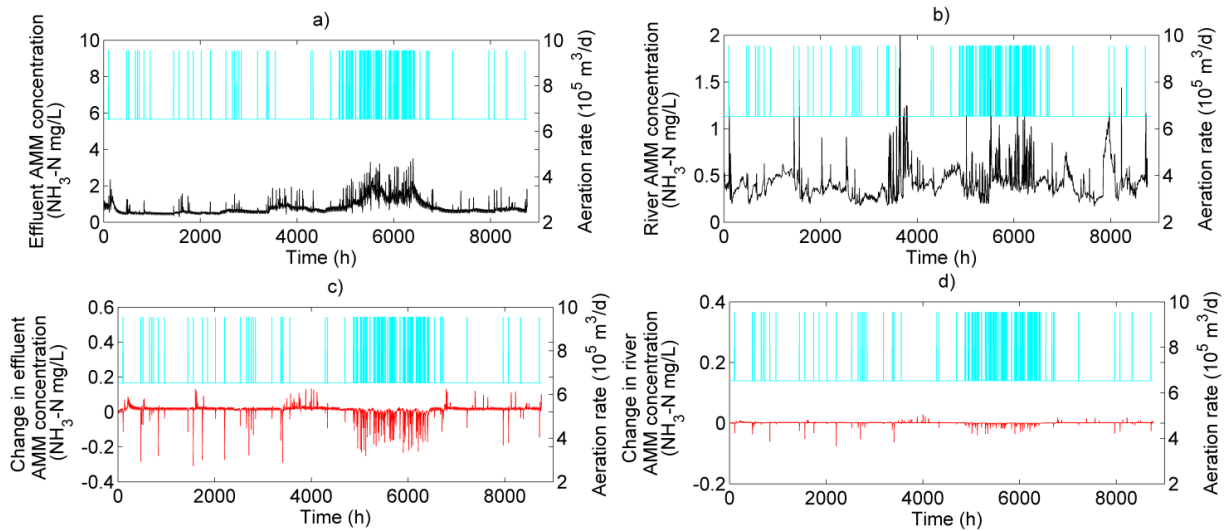


Figure 5.11 Time series of air flow rate (in cyan) under the RTC strategy in Table 5.6 with effluent (in a)) or downstream river total ammonia concentration (in b)) and the changes after applying the RTC strategy (in c) and d)) using data set 'B'

The result of the permitted RTC strategy is plotted in Figure 5.12 (marked as a red diamond) with results of optimal fixed operation solutions produced in section 4.5.2 using data set 'B' and the permitted fixed optimal operation solution using data set 'A' (marked as a red square). Compared to the optimal fixed operation solution marked as the red square, the RTC strategy slightly reduces energy cost (0.8%) and environmental risk (1%), and greatly enhances process stability as shown in Figures 5.12b and 5.12d.

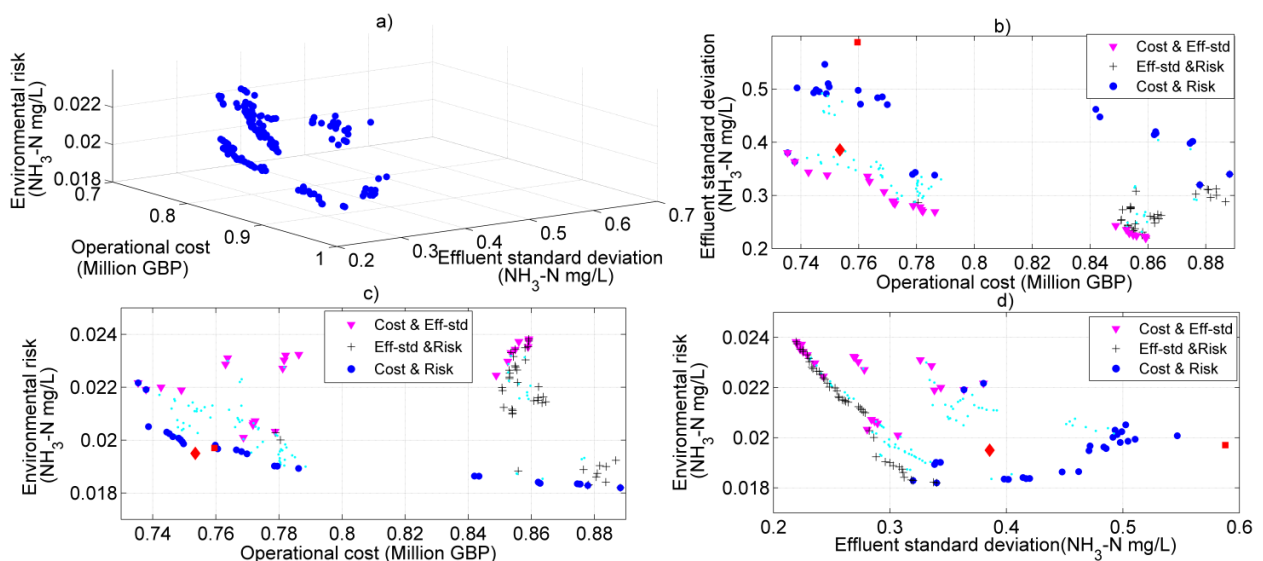


Figure 5.12 Performance of the permitted RTC strategy in Table 5.6 and the permitted fixed operation solution in Table 4.6 under data set 'B' (shown as red

diamond and red square, respectively) and non-dominated Pareto fixed operation solutions using data set 'B' with objectives of operational cost, effluent standard deviation and environmental risk in two- and three-dimensional space (other symbols the same as in Figure 4.9)

5.5 Conclusion

An RTC-based permitting approach was proposed in this chapter to identify cost-effective RTC strategies based on a systematic assessment of the economic and environmental performance of the UWWS. This regulation approach should encourage the uptake of RTC technology in the wastewater industry, which offers benefits in operational savings without compromising the environmental quality by exploiting the dynamic assimilation capacity of the environment. For the integrated real-time aeration control scheme investigated in this chapter, further improvement in environmental benefits can be achieved compared to the optimal operational strategy in Chapter 4. However, a trade-off between energy cost and environmental quality exists, and marginal saving in energy cost can be realised if discharge pollutant load from the UWWS is not allowed to be increased. Thus more tests on other controller structures or algorithms or control types (e.g. flow control) are needed to find if integrated RTC strategies could enable significant cost savings without deteriorating the environmental quality. Nevertheless, the proposed three-step RTC-based permitting framework is a useful tool for exploring cost-effective RTC strategies and permitting according to solutions with high performance.

6 Risk-Based Cost-Effective Permitting

6.1 Introduction

Two innovative permitting approaches have been introduced in Chapters 4 and 5 to deliver maximised and balanced benefits by implementing optimal integrated operational or control strategies. An integrated UWWS model was used to evaluate the multiple impacts of an integrated operational/control strategy, and NSGA-II was employed to efficiently search for optimal operational/control solutions to simultaneously minimise energy cost and adverse environmental impacts. Whilst great benefits can be achieved by the two proposed approaches, they may be too comprehensive and costly to be implemented in some cases due to the amount of data, technique and effort required to establish an integrated UWWS model and to master the optimisation tools.

Though integrated UWWS modelling is rarely used except in academic research, it is common to model subsystems of an UWWS (i.e. sewer or WWTP) for planning, design or process control purposes. To adapt to the current common practice, an innovative permitting model, similar to that of Chapters 4 and 5 but simpler and needs no additional modelling and complex optimisation techniques, is developed in this work. This is achieved by establishing an integrated cost-risk analysis framework coupling the use of a dynamic wastewater system model and a modified statistical permitting model. The former model is used to explore a cost-efficient operational/control strategy to achieve an effluent quality, while the latter to evaluate the environmental impact related to a specific level of effluent quality. An integrated analysis of the results from the two models enables identification of the operational/control strategy for cost-effective permit. The modelling strategy of this method, as compared to that of the two previously introduced permitting approaches and the traditional policy, are illustrated in Figure 6.1.

The risk-based cost-effective permitting approach is introduced in this chapter by using permitting of a WWTP effluent discharge through optimisation of an operational strategy, though it can be extended without major modifications to permitting of intermittent wastewater discharges and/or use of optimisation of RTC strategies. In the following sections, the method is described in section 6.2, and the application to a case study site is presented in section 6.3. A discussion

is provided in section 6.4 on the uncertainty of the proposed permitting model, and the potential of using the model for catchment-based permitting.

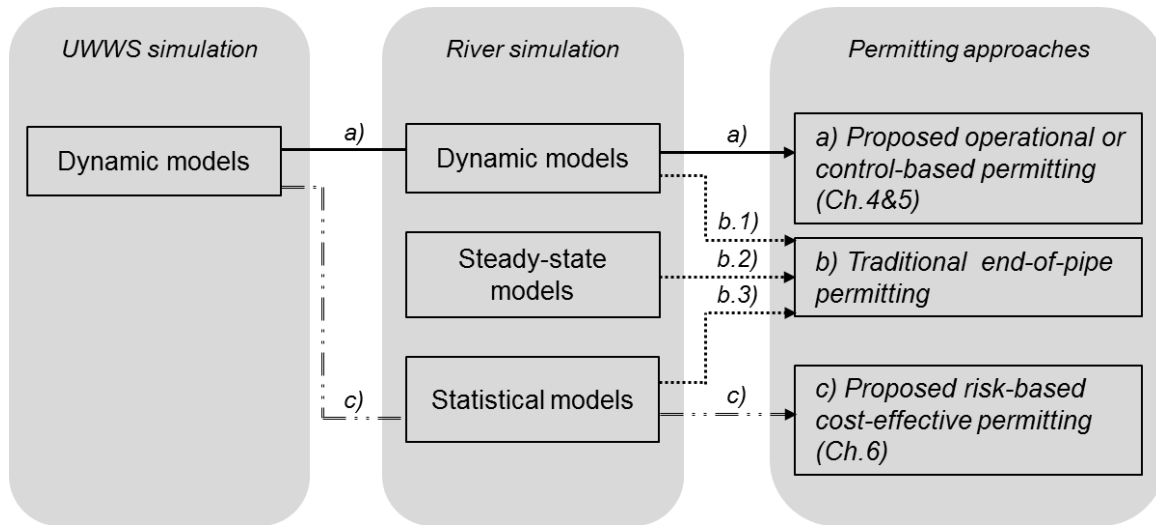


Figure 6.1 Illustration of the modelling strategies for different permitting approaches

6.2 Methodology

The integrated cost-risk analysis framework for the risk-based cost-effective permitting is presented in Figure 6.2. The three modules for permit development, i.e. cost calculation, risk calculation and cost-risk analysis, are represented in grey boxes in the figure and are explained in detail as follows.

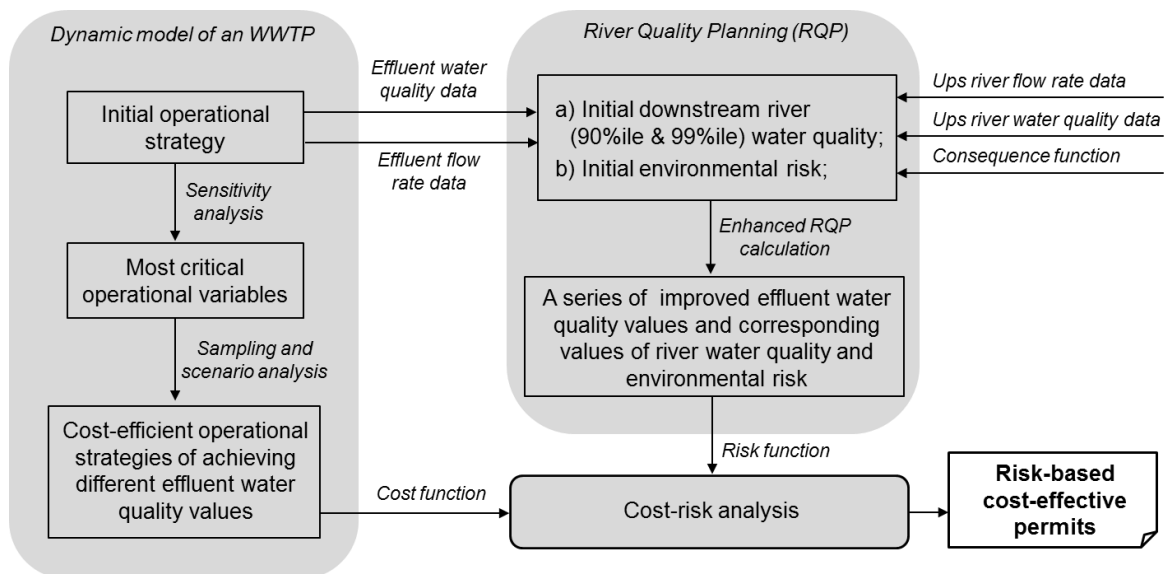


Figure 6.2 Integrated cost-risk analysis framework for risk-based cost-effective permitting

i) Cost calculation module:

A dynamic WWTP model is used in this module to measure the operational cost and effluent water quality (described in 95%ile) associated with an operational strategy by long-term simulation. The strategy that achieves an effluent 95%ile water quality value with minimum cost is searched by optimisation techniques. The optimisation needs to be conducted for a range of effluent 95%ile values as the environmental impact of a wastewater discharge is unknown (in other word, what level of effluent water quality would be of concern is unclear) from a WWTP model. Hence, despite the fact that only a single objective (i.e. minimising operational cost) is considered, the optimisation is still very challenging if carried out in a “top-down” manner, i.e. define n effluent 95%ile water quality values, and for each one of them, optimise settings of operational handles to achieve the (almost exact) effluent quality value with minimum cost. As such, a “bottom-up” approach is used by varying the settings of the operational handles first, and finding the lowest-cost operational solutions of achieving different effluent 95%ile values from candidate solutions. To ensure the quality of the optimal solutions yielded by this approach, a large quantity of candidate operational strategies would be necessary particularly when generated using random sampling. Thus to improve the searching efficiency, a one-at-a-time (OAT) sensitivity analysis is employed in this work to identify operational variables that have a significant effect on the system performance. Sampling is then focused on the most critical operational variables without sacrificing much the quality of the optimisation. Figure 6.3 shows an example of the cost function produced by this module.

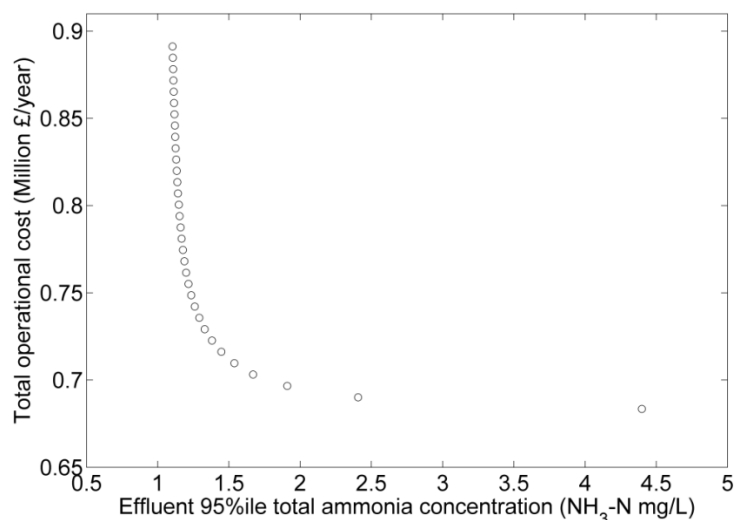


Figure 6.3 An example of cost function

ii) Environmental risk calculation module:

The statistical permitting model RQP reviewed in section 2.2.1 is employed to evaluate the environmental impacts of the wastewater discharge corresponding to the initial operational strategy (flow and water quality described in statistical parameters), and calculate the change in the impact values by improving effluent quality until the environmental standard is just met. As the original code of RQP is not available for this work, the river water quality modelling and permitting processes of RQP are reproduced according to descriptions in the guidance (Environment Agency, 2011b; Murdoch, 2012) on the platform of MATLAB. To fit for the research need of this work, the (reproduced) original RQP model is modified to a) calculate environmental risk based on information already provided by the current modelling, and b) record the results from the iterative calculation process of permitting (i.e. successively reducing/increasing standard deviation and average values of effluent water quality) to facilitate cost-effective permitting analysis.

To calculate environmental risk in RQP, a consequence function is introduced to measure the impact of river water quality deterioration (Botto et al., 2014). Though it can be defined in various linear or non-linear functions, a piecewise linear function (Equation 6.1) is developed here to be consistent with the definition in Equation 4.1. As illustrated in Figure 6.4a, the consequence value is zero below a threshold and increases linearly afterwards. By integrating the product of the consequence function and the probability distribution of downstream river water quality (blue solid line in Figure 6.4a from zero to infinite, one environmental risk value can be derived (Equation 6.2).

$$E_c = \begin{cases} 0 & (C < C_T) \\ C - C_T & (C \geq C_T) \end{cases} \quad (6.1)$$

$$\text{Risk} = \int_0^{\infty} E_c \cdot P_r dC \quad (6.2)$$

Where C is the downstream river water quality, C_T is the threshold limit, which is the 90%ile river water quality standard (same as in Equation 4.1), E_c is the consequence value corresponding to river water quality C , which is $(C - C_T)$ if the river water quality is above C_T and zero otherwise, and P_r is the related probability.

Each environmental risk value, computed based on one set of consequence and probability functions, corresponds to one effluent water quality distribution, thus one effluent 95%ile value. For example, the risk value calculated by the two functions in Figure 6.4a is marked in Figure 6.4b as a red square. Located at the lower end of the risk function curve, the red square represents the compliance solution, i.e. the environmental standard is satisfied if the effluent 95%ile water quality value is equal to or lower than the value corresponding to the red square. The other end of the curve is the result from the initial data set and the points between the two ends are the intermediate results recorded in the permitting process. By the traditional regulatory approach, permit would be the effluent 95%ile value of the compliance point. Yet under the proposed permitting approach, the entire risk function is used for cost-effective permitting in the next module.

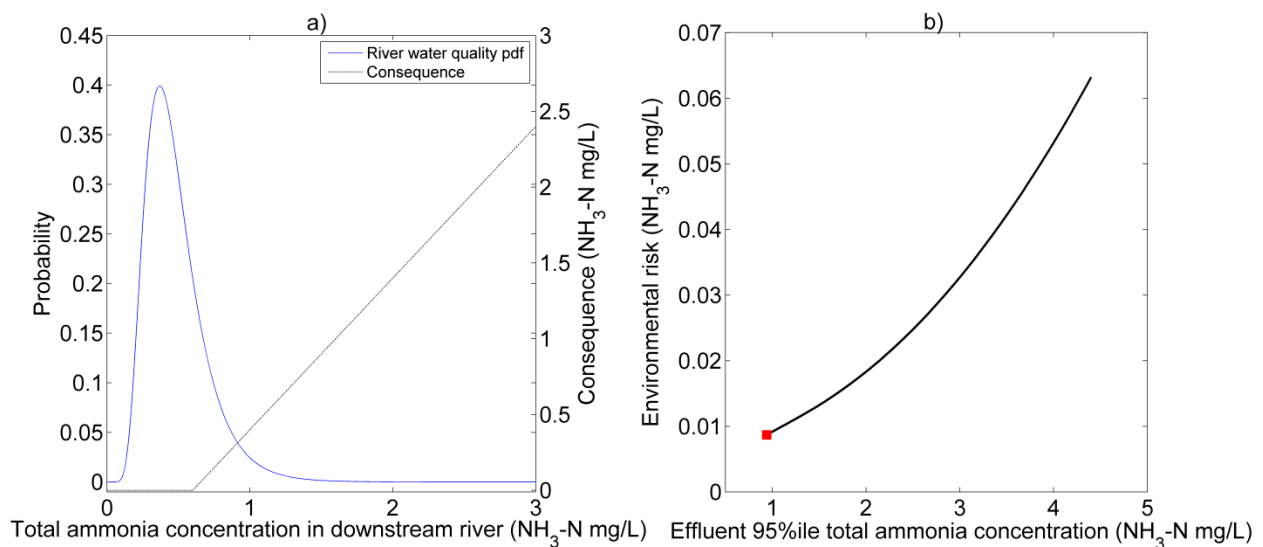


Figure 6.4 a) River water quality probability distribution function and consequence function for production of environmental risk, and b) environmental risk function with the risk value calculated for the functions in a) highlighted as a red square

iii) *Integrated cost-risk analysis module:*

A cost-effective permit is derived by integrating the cost and risk functions produced in the first two modules and evaluating the investment (or increase in GHG emissions) needed to achieve a certain target of risk reduction (or river water quality improvement). For example, the cost function in Figure 6.3 and risk function in Figure 6.4b are plotted together in Figure 6.5. By observing the slopes

of the two curves, the three points in the red circle (corresponding to three operational strategies) are shown to be good options for cost-effective permits, as compared to the starting point ('S') the environmental risk is greatly reduced without entailing excessive cost. Yet compliance of the environmental standards (i.e. the left end point of the risk function) needs also to be considered in permitting. Figure 6.5 shows three possible compliance points: 'C1', 'C2' and 'C3'. If the compliance point is at 'C3', it would suggest the three circled solutions are preferable to the traditional permit as they are more environmentally protective without laying much economic burden to the WWSP. At compliance point 'C2', disproportionate cost would be needed to meet the environmental standards. In this scenario, whether the three circled solutions are more reasonable permit options depends on whether sacrifice in the environmental water quality is acceptable to the specific site. For compliance point 'C3', though the corresponding effluent 95%ile value is only 0.2 mg/L more stringent than that of 'C2', this effluent quality is technically unachievable through optimising operational strategies with the existing system and other (possibly much more expensive) compliance strategies would be required. Informed by this, the regulators may consider other cost-effective measures at a catchment scale to protect the water environment rather than putting too much pressure on the WWSP.

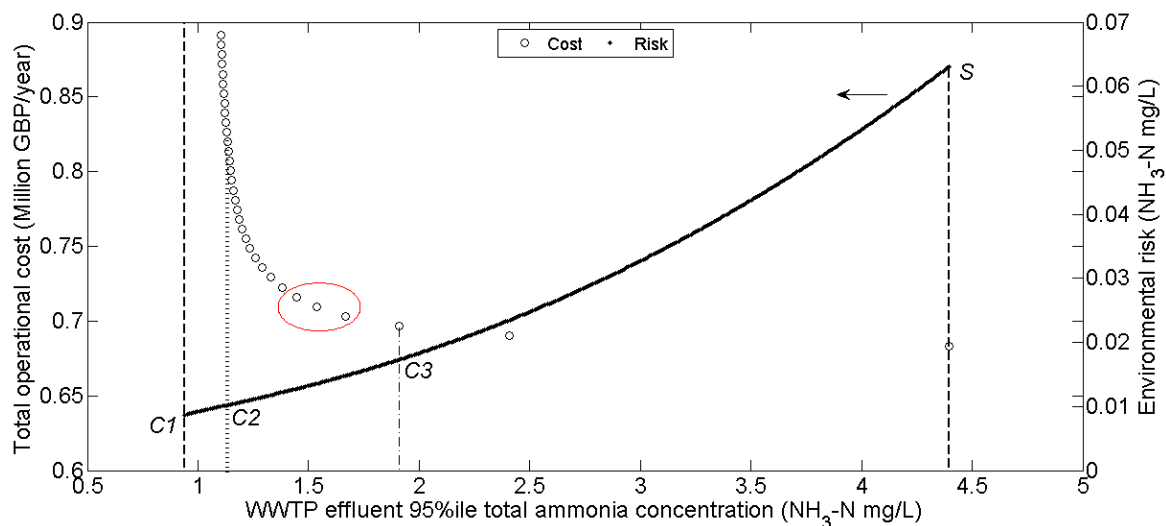


Figure 6.5 Integrated cost-risk analysis for the derivation of cost-effective permits with three promising solutions marked in a red circle (S - system performance before permitting, C1 to C3 – three possible compliance points)

6.3 Results

The proposed method is applied to the same case study WWTP in Chapters 4 and 5 but with rainfall and river data set 'B' (described in section 4.3.2) as it is from detailed monitoring records of a real river. Results from the three permitting modules are presented as follows.

6.3.1 Calculation of Cost Function

The dynamic WWTP model used in previous chapters is employed in this module to produce the cost function. As a first step, the OAT sensitivity analysis is performed to screen out less critical operational variables. It is conducted by changing the setting of only one operational variable at a time to the low or high bound value and evaluate the system performance by one year simulation, while keeping the other five variables in the WWTP at their baseline values (the baseline and low and high bound values are also listed in Table 4.4); the variable is then returned to its baseline value, and the process is repeated for each of the other variables in the same way. Sensitivity, measured as the changes in the output values (i.e. total operational cost and effluent 95%ile total ammonia concentration in this case) to that of the baseline scenario, are shown in decreasing order in Figure 6.6. It can be seen from the figure that aeration rate is the most critical operational variable affecting both the operational cost and effluent quality. For example, the total operational cost increases by 15.7% when the aeration rate is changed from the baseline setting (720,000 m³/d) to the high bound value (1,200,000 m³/d), and reduces by 18.1% when changed to the low bound value (240,000 m³/d); and the effluent 95%ile total ammonia concentration deteriorates by 385.8% when the aeration rate is set to the low bound value. The return and waste sludge pumping rates are the second most sensitive variables following the aeration rate. In addition, effluent total ammonia concentration is shown to be sensitive to the flow to full treatment setting as well shown in Figure 6.6b. This is expected as it determines the maximum amount of wastewater flow to the WWTP thus directly affects the treatment efficiency in the plant. In this work, however, FFT is not considered in the following analysis as the change in its setting will affect the volume of wastewater overflowing to the river, the environmental impact of which cannot be evaluated together with the WWTP effluent discharge by the single discharge permitting model RQP (the catchment

permitting model SIMCAT would be suitable, however, it is out of the scope of this study).

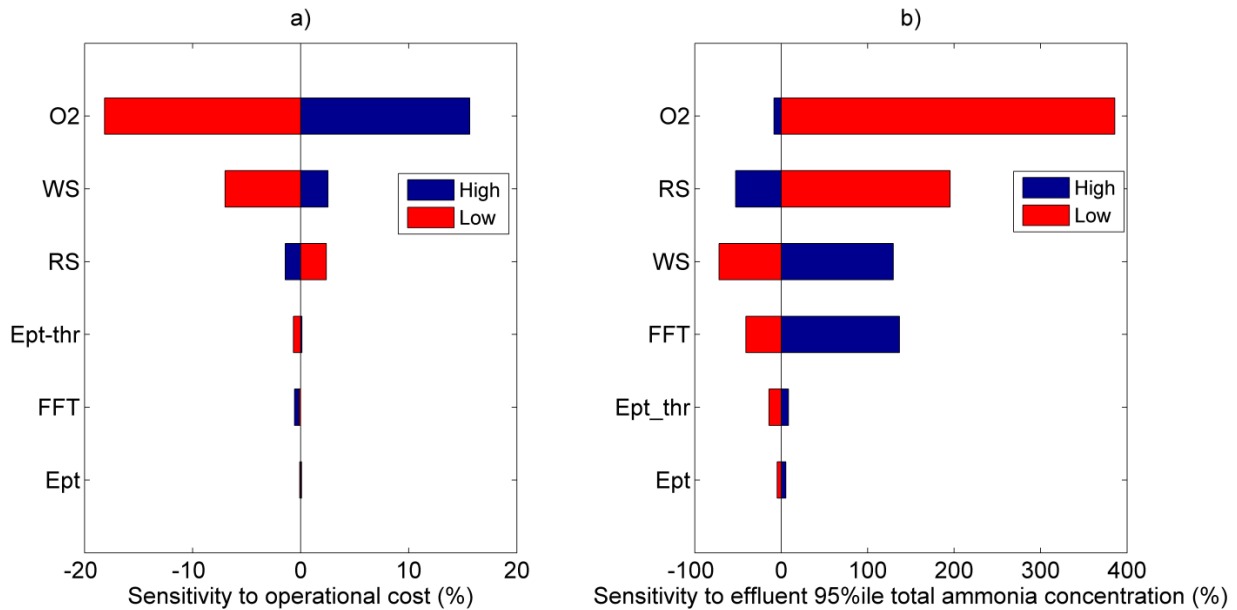


Figure 6.6 Criticality of operational variable settings to a) operational cost and b) Effluent 95%ile total ammonia concentration (FFT - flow to full treatment, Ept-thr - storm tank emptying threshold, Ept - storm tank emptying rate, RS - return sludge rate, WS - waste sludge rate and O2 - aeration rate)

It is quite straight-forward to select the most critical operational variables for this case study based on results in Figure 6.6. However, the screening process can be much more complicated for cases with a large number of operational variables and/or system performance indicators of concern. Definition of a sensitivity threshold would be useful in these situations to facilitate the screening. For instance, if the output value changes by more than 50% (the threshold limit) when operational variable setting is varied to either the low or high bound, the operational variable is considered to be critical. Caution should be taken in defining the sensitivity threshold as well as the low and high bounds of each variable due to the direct and great impact on the final results. A major limitation of the OAT sensitivity analysis is the inability to identify correlations between variables, thus more advanced methods such as variance-based methods (Sweetapple et al., 2014b, 2013; Zhang et al., 2013) should be taken if a deeper analysis is needed.

Besides the level of criticality, results of the sensitivity analysis could also indicate the direction of how settings of the operational variables could be improved thus facilitate the following search for the optimal operational strategy. As shown in Figure 6.6, the two system outputs conflict with each other when the aeration rate is changed, e.g. by changing the aeration rate to the high bound value the operational cost increases but the effluent 95%ile total ammonia concentration decreases, suggesting a trade-off analysis is necessary to determine the aeration rate value. In contrast, both two performance indicators are improved when the return sludge rate is set at the high bound value and the wastewater sludge rate at low bound value, suggesting no optimisation on the two operational settings is needed. However, this conclusion is only valid if the system performance changes monotonically when varying the operational setting from the low bound value to the high bound one. As such, detailed scenario analysis was run for each of the three operational variables by varying the setting at a range of equally distributed values within the feasible ranges.

The scenario analysis results, as shown in Figure 6.7a, confirm the presumptions above. The cross point of the three curves is the baseline scenario (with settings of return sludge rate 14,400 m³/d, waste sludge rate 660 m³/d and aeration rate 720,000 m³/d) as marked in Figure 6.7b point 'A'. Starting from this point, the two performance objectives can be simultaneously improved by changing the setting of sludge pumping rates to that of the left end points of the curves (i.e. points 'B' and 'C' in Figure 6.7b) without varying the aeration rate. The curve for the aeration rate (sludge pumping rates for all points in the curve are the same as the baseline settings) clearly shows the trade-off relationship. Moreover, the slope of the curve rises slowly at the beginning from the right end (the biological treatment process is impaired by too limited air supply as reflected by the unreasonably high effluent water quality at the right end) and steepens towards the other end, suggesting increase in aeration intensity is cost-effective in improving effluent quality only up to a certain level. This is expected for biological treatment processes, as the dissolved oxygen concentration gradually approaches the saturation point with increasing aeration rate, and the growth rate of microorganisms slows down with increasing dissolved oxygen (Tchobanoglous et al., 2004).

Informed by the scenario analysis results on the individual operational variables, another scenario (point 'D' in Figure 6.7b) was run to combine the optimal settings of return and waste sludge rates. This achieves further benefits - the effluent water quality is better and the operational cost is lower than either 'B' or 'C'. Based on the optimised sludge pumping operation, another set of scenarios are conducted to regenerate the curve for the aeration rate and the results are presented in Figure 6.7b marked in blue diamonds. The advance of the curve on aeration rate towards the left corner is the result of optimisation of sludge pumping operation. Though there might be an opportunity for further improvement in system performance by optimising settings of other operational variables, the potential seems marginal in this case due to their weak impact on the system performance as revealed by the sensitivity analysis (Figure 6.6). Moreover, the change in other operational settings would affect the volume of wastewater overflowed in a direct or indirect manner, thus this circumstance is not considered here but discussed further in section 6.4. Hence, the regenerated curve on aeration rate is the cost function used for cost-effective permitting of this study.

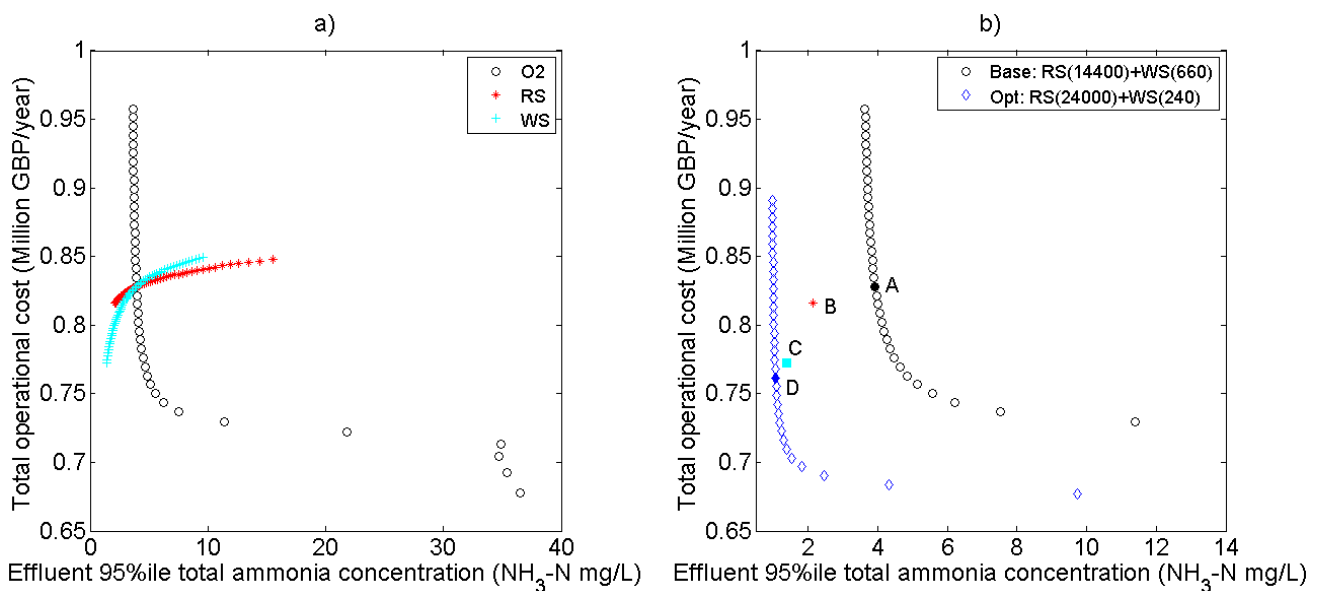


Figure 6.7 a) Scenario analysis by individually varying settings of aeration rate, return sludge rate and waste sludge rate; and b) optimisation of operational strategies and development of cost function (A – baseline scenario, B and C – scenarios shown as the left end points of the curves on return sludge rate and waste sludge rate in a) respectively, D – scenario combining the settings of scenarios B and C)

6.3.2 Calculation of Environmental Risk Function

The WWTP effluent data under the baseline operational strategy produced by the dynamic WWTP model is fed to this module to produce the environmental risk function by the enhanced RQP. The calculation procedure is summarised in Figure 6.8 with modifications to the original RQP highlighted in bold italic or in shade. The five major steps are explained as follows.

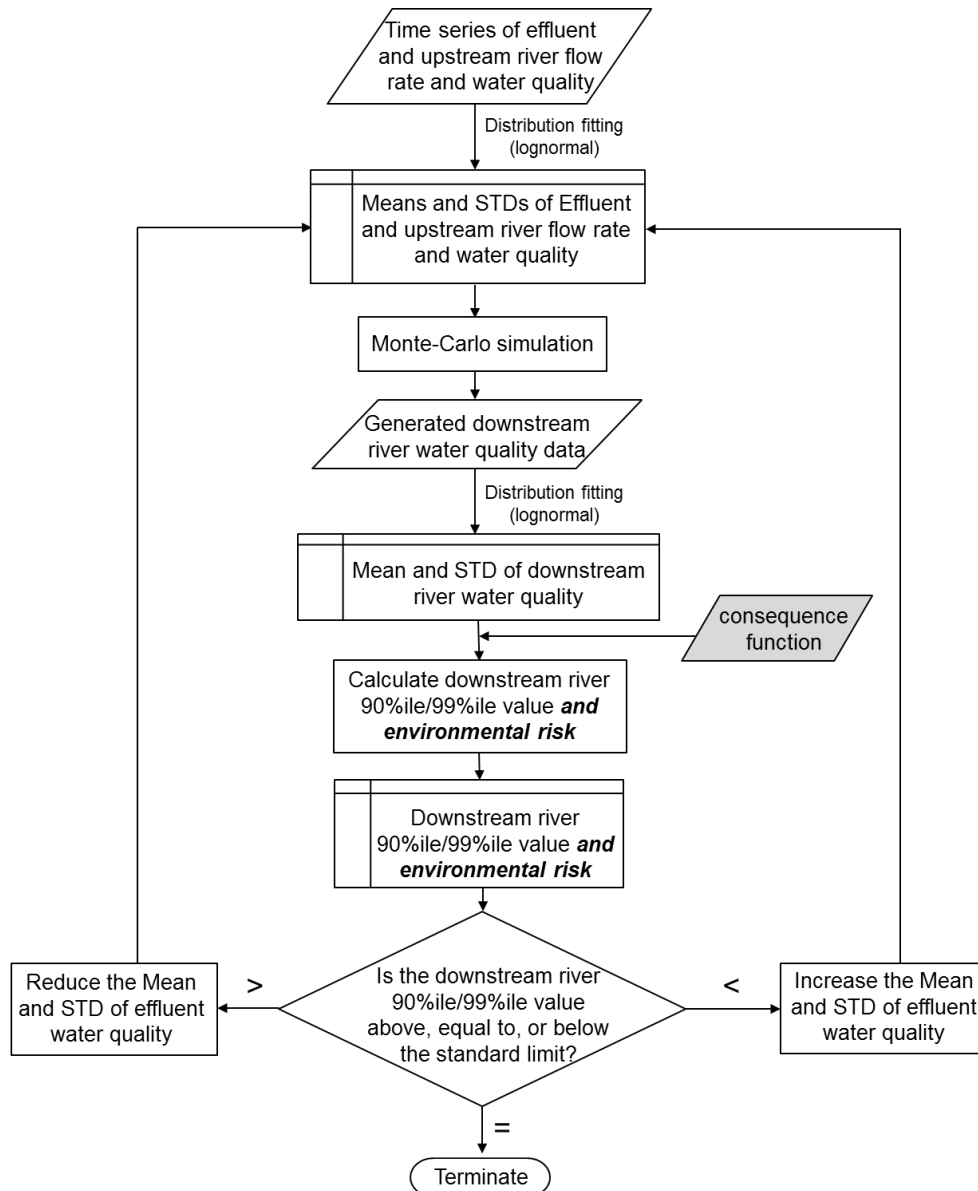


Figure 6.8 Calculation procedure by enhanced RQP with modifications to the original RQP highlighted in bold italic or in shade

Step I: Distribution fitting to input data sets. As mentioned in section 2.2.1, values of river and wastewater effluent flow and water quality are assumed to follow lognormal distributions in RQP. Thus the probability distributions for the

four input variable values, characterised by means and standard deviations (STDs), are determined by fitting lognormal distributions to the one-year 15-minute historical monitoring data on upstream river flow rate and 30-minute increment data on total ammonia concentration, and 1-hour increment simulation data on effluent flow and water quality produced by the dynamic model using baseline operational strategy.

Step II: Monte-Carlo simulation. Monte-Carlo sampling (similar to the LHS used in previous chapters) is employed by RQP to estimate the initial downstream total ammonia concentration by drawing random samples from the four lognormal distributions generated in step I and calculate the downstream river water quality by solving mass-balance equations. The input variables may be correlated in real-life, e.g. the river flow rate and effluent flow rate are often positively correlated due to the influence from the same rainfall events. By designating correlation coefficients between the variables, the correlation can be taken into account in the random sampling. In this work, the default correlation coefficient settings in RQP are used, which are 0.6 for the river flow rate and effluent flow rate, and zero for all others. To achieve reproducibility in random sampling, seeds for generating random numbers for the samples can be controlled in RQP. This is also adopted for this work so that the same downstream river water quality results will be produced for the same input data series. More details of the Monte-Carlo method can be found in section 2.2.1.

Step III: Distribution fitting to the generated downstream river water quality data. Similar to step I, a lognormal distribution is fitted to the downstream total ammonia concentration values generated from Monte-Carlo simulation in Step II.

Step IV: Calculation of downstream river water quality and environmental risk. The 90%ile and 99%ile downstream total ammonia concentration values are derived based on the mean and standard deviation of the lognormal distribution determined in step III. The environmental risk is calculated according to Equation 6.2 based on the probability distribution generated in step III and the predefined consequence function defined in Equation 6.1. The threshold limit (C_T) takes the value 0.6 NH₃-N mg/L (Table 4.3) as explained in section 4.3.2.

Step V: Development of the environmental risk function by iterative calculation. If the 90thile or 99thile total ammonia concentration in downstream river calculated from step IV is higher or lower than the environmental standard limit, the probability distribution for the effluent water quality is modified by successively reducing or increasing standard deviation and mean values while keeping the CV value constant, and the calculation from step II to V is repeated until the standard limit is just being met. By recording results from this iterative calculation for traditional permitting, the environmental risk function is developed. In this case study, the 99thile environmental standard limit is satisfied (1.04 NH₃-N mg/L) while the 90thile limit is violated (0.72 NH₃-N mg/L), so to be environmentally protective, the 90thile standard limit is used for developing the environmental risk function.

Figure 6.9 presents the produced environmental risk function (Figure 6.9a) as well as results for the CV, mean and standard deviation of effluent total ammonia concentration (Figures 6.9a and 6.9b), and the 90thile and 99thile total ammonia concentration in downstream river (Figure 6.9c). The calculation terminates when the effluent 95thile total ammonia concentration is reduced to 0.78 NH₃-N mg/L. Yet results beyond that point, though not to be used for permitting purposes, are also presented in this figure (coloured in cyan) to show the trends. The evolution of effluent water quality along the iterative calculation is illustrated by presenting probability density functions (PDFs) of five representative points as shown in Figure 6.9d, with effluent 95thile values being 3.93 NH₃-N mg/L (starting point), 2.00 NH₃-N mg/L, 0.78 NH₃-N mg/L (compliant point) and 0.51 NH₃-N mg/L, respectively.

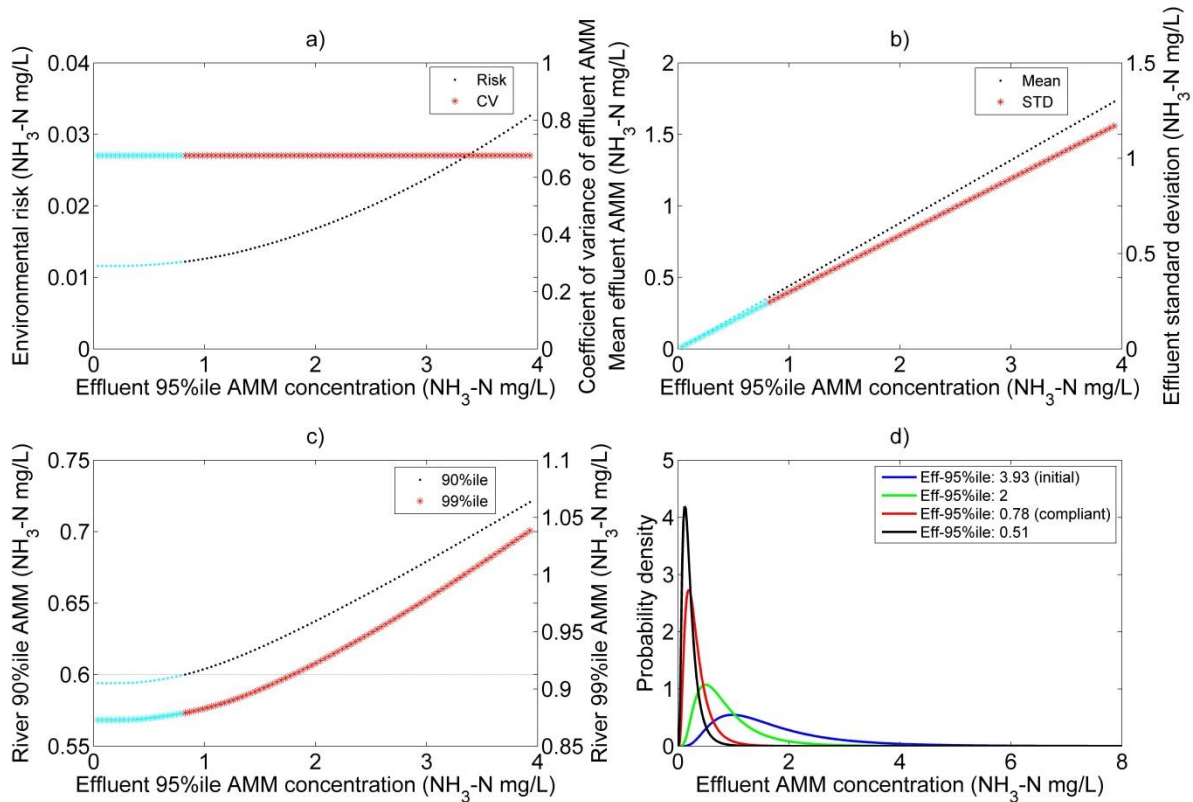


Figure 6.9 Functions of environmental risk (a) and water quality-related parameters (i.e. coefficient of variance, mean and standard deviation of effluent water quality shown in a) and b), 90%ile and 99%ile of river water quality shown in c), and five PDFs on effluent water quality with the same CV value but different 95%iles) (results compliant of environmental standards shown in cyan in a) to c))

6.3.3 Integrated Cost-Risk Analysis for Cost-effective Permitting

Informed by the added cost analysis and environmental risk assessment, risk-based cost-effective permits can be derived. There are no criteria for ‘cost-effectiveness’ of wastewater discharge permits, as it would depend on the expectation on the environmental quality and sufficiency of budget which vary from case to case. However, by providing a holistic view of the environmental and cost impacts of a wide range of effluent 95%ile values, flexibility is given to the regulators in making decisions suited to the need and priority of local situations.

The cost and risk functions produced in the first two modules are plotted together in Figure 6.10a. It can be seen that the effluent 95%ile value needed to achieve the environmental standard, i.e. permit ‘C’ derived by the traditional approach, is not achievable even with optimised sludge pumping rates and highest aeration

rate. To comply with permit 'C', therefore, investment in upgrading the treatment process is likely to be incurred.

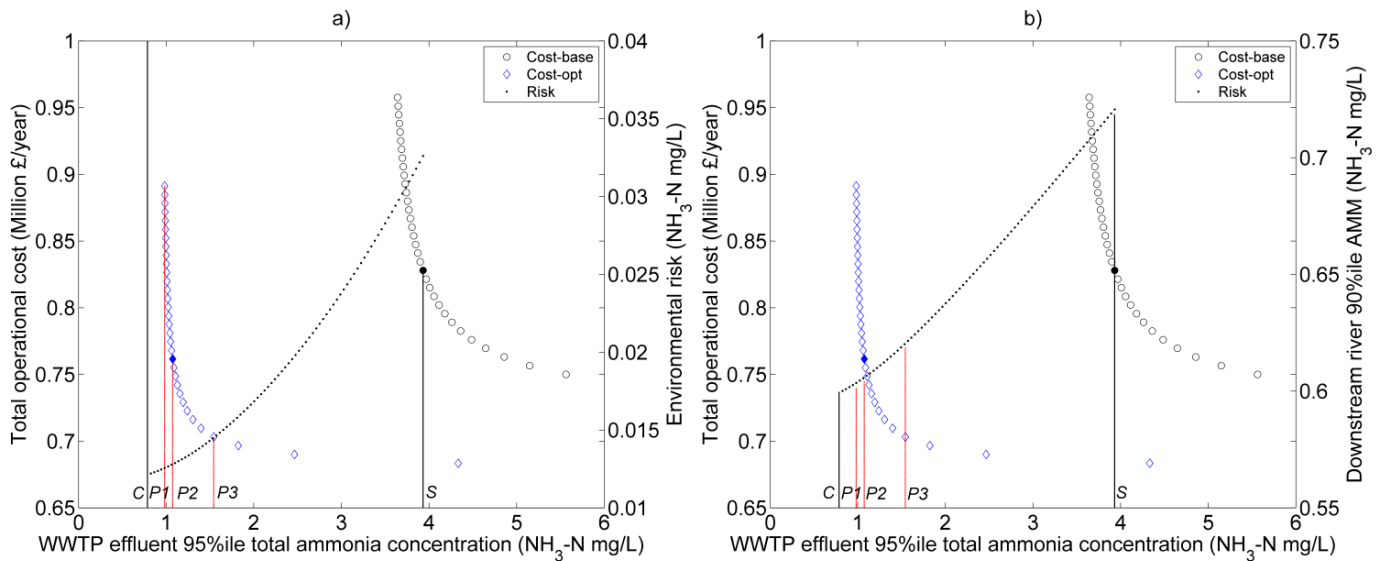


Figure 6.10 Integrated analysis of cost and risk (a) and river 90%ile water quality (b) functions for the derivation of cost-effective permits (S – starting point, i.e. before permitting, C – point where the environmental standards are just met, P1 to P3 – potential cost-effective permits for the case study)

If the compliance with environmental standard can be compromised, three representative candidate permits can be imposed, which are:

- 1) Best achievable permit ('P1'): This is based on the maximum operational capacity of the existing treatment process. It offers a minimum deviation from the environmental standard as indicated in Figure 6.10b but entails 7.2% higher cost than the baseline scenario ('S').
- 2) Least operational change permit ('P2'): The corresponding aeration rate is the same as the baseline scenario, thus only the sludge pumping rates need to be changed. Nevertheless, it could achieve a 8.4% cost reduction than the baseline scenario 'S' with only a slight increase in environmental risk and downstream river 90%ile total ammonia concentration compared to 'P1'.
- 3) Economically achievable permit ('P3'): This option could make the best use of financial resources in improving environmental quality as reflected by the slope of the cost function curve. Though it violates the environmental 90%ile limit by 0.02 NH₃-N mg/L, its environmental performance is still much better than the baseline scenario.

Detailed information related to the three alternative permits, the traditional permit and the baseline scenario is provided in Table 6.1. Percentage of change compared to that of the baseline scenario is calculated as shown in brackets. By comparison, 'P3' seems a better option for cost-effective permitting than 'P1' and 'P2'. However, the final permit is to be determined by the decision-makers according to their preferences and local needs, and the integrated cost-risk analysis provides a useful tool assisting informed decision-making.

Table 6.1 Comparison of performance of the three potential permits

Performance indicator	P1	P2	P3	C (By traditional approach)	S (Baseline scenario)
Effluent 95%ile value (NH ₃ -N mg/L)	0.98 (-75.1%)	1.07 (-72.8%)	1.54 (-60.8%)	0.78 (-80.2%)	3.93
Total operational cost (Million £/year)	0.89 (7.2%)	0.76 (-8.4%)	0.70 (-15.7%)	--	0.83
Environmental risk (10 ⁻³ NH ₃ -N mg/L)	12.6 (-61.3%)	12.8 (-60.7%)	14.5 (-55.5%)	12.1 (-62.9%)	32.6
Downstream river 90%ile value (NH ₃ -N mg/L)	0.60 (-16.7%)	0.61 (-15.3%)	0.62 (-13.9%)	0.60 (-16.7%)	0.72

After the effluent 95%ile value is selected for the permit, the corresponding operational strategy is set as the permit, whilst the effluent 95%ile value is also listed in the permit as a reference. The control-based permit based on 'P3' is presented in Table 6.2.

Table 6.2 Proposed form of risk-based cost-effective permit

Variable	Permit value
Waste sludge pumping rate (m ³ /d)	240
Return sludge pumping rate (m ³ /d)	24,000
Aeration rate (m ³ /d)	504,000
Effluent 95%ile total ammonia concentration (NH ₃ -N mg/L)	1.54

6.4 Discussion

6.4.1 Opportunities for Catchment-Based Permitting

Section 6.3.3 has discussed the methods of deriving cost-effective permits on condition that exceedance of the environmental standard is acceptable. If no violation in the standard limit is allowed, however, additional compliance strategies would be necessary. Upgrade of the wastewater treatment process is an intuitive strategy, yet it may cause too much than necessary burden on the WWSP. Indeed, it may not be a favourable solution under the catchment management policy, which aims to achieve maximum environmental benefits with least cost for all regulated parties in the catchment based on systems thinking. Hence, this section discusses the opportunity of achieving the environmental standard by catchment-based permitting, i.e. permitting of operation in a WWTP in coordination with the regulation of other pollution sources in the catchment.

Table 6.3 shows three pollution control strategies targeted at wastewater discharges from different sources in a catchment. Strategy a) aims to balance the intermittent and continuous wastewater discharges by changing the overflow settings. If it is to be adopted, both the cost and risk functions for WWTP effluent discharge permitting should be reproduced as the strategy will alter the flow to the treatment process as well as the upstream river condition for the change in the volume of overflow discharges. As reproduction of the cost function is somewhat time-consuming by repeating the calculation in section 6.3.1, strategy a) is not considered in the following discussion. With regards to strategies b) and c), only the risk function needs to be recalculated as the control measures of the two strategies have little or no interference to the wastewater treatment process, hence they are used for the discussion of catchment-based permitting. It should be noted that the following discussion is a purely hypothetical analysis, and reasonable assumptions were made on the cost and environmental impact of the upstream improvement strategies. This is considered to be acceptable for this work as the intention is to illustrate the methodology rather than to prescribe a specific catchment-based permit.

Table 6.3 Examples of water pollution control measures in a catchment

Strategy	Targeted system
a) Balancing continuous and intermittent wastewater discharges, such as by optimising: <ul style="list-style-type: none"> ▪ FFT; ▪ overflow thresholds for CSOs and storm tanks; and ▪ emptying rate of storm tanks 	UWWS
b) Reducing pollution from urban runoffs (e.g. by building SuDS); and c) Reducing pollution from agricultural runoffs (e.g. by efficient use of fertiliser)	Upstream areas of the UWWS

A series of tests were conducted to calculate the impact of upstream improvement (expressed as percentage reduction of total ammonia concentration in the upstream river) on the change of effluent 95%ile total ammonia concentration required to achieve the downstream environmental standard (i.e. 'C' in Figure 6.10), and savings in operational cost compared to the baseline scenario ('S') and environmental risk corresponding to the new permit 'C'.

The experiment is carried out by iterative calculations following the steps below.

- 1) Reduce the upstream total ammonia concentration time series by 1% of the original values.
- 2) Develop the risk function based on the new upstream river water quality data, as well as the original river flow data and effluent flow and water quality data.
- 3) Check if the derived permit 'C' (i.e. the left end of the risk function generated in step 2)) can be achieved by the maximum operational capacity of the WWTP (i.e. 'P1' in Figure 6.10). If yes, record the effluent 95%ile value of permit 'C' and the corresponding risk value and the savings in operational cost, and repeat the calculation from step 1); if not, do not record the results and repeat the calculation from step 1).

- 4) Terminate when the environmental standard can no longer be met with further relaxation of effluent water quality.

Results on the evolution of permit 'C', environmental risk and operational cost saving are presented in Figures 6.11 and 6.12 against percentage of upstream river water quality improvement. It can be seen that with only 2% improvement in upstream river water quality, the environmental standard can be met by an optimised operational strategy in the WWTP. Continued betterment in upstream river water quality can allow the WWTP effluent quality to be further relaxed to about 8 NH₃-N mg/L for further energy savings in the WWTP without violating the environmental standard. However, despite the fact that the downstream river 90%ile total ammonia concentration values of all plotted data points are the same as being the standard limit (0.6 NH₃-N mg/L), the environmental risk values marked in red asterisks in Figure 6.11 vary and rise quickly after about 30% of upstream improvement with only marginal savings in the WWTP (black dots in Figure 6.12).

To derive a cost-effective catchment-based permit, the investment for upstream improvement should be considered along with the operational cost savings in the WWTP, and with regards to the environmental consequences as well. As no suitable data were found to develop a cost function for this case study, a cost curve for the control of N_r (i.e. reactive nitrogen, including NO_x, NH₃, N₂O and NO₃⁻) leaching reported in a study (Sutton et al., 2011) is adapted for illustration purposes. For completeness, both capital and operational costs for the investment need to be calculated, however, only capital cost is considered here for a simplified analysis as strategies b) and c) in Table 6.3 incur mainly one-off costs. The stair-shape of the curve is a reflection of the different levels of capital cost needed for various pollution control measures.

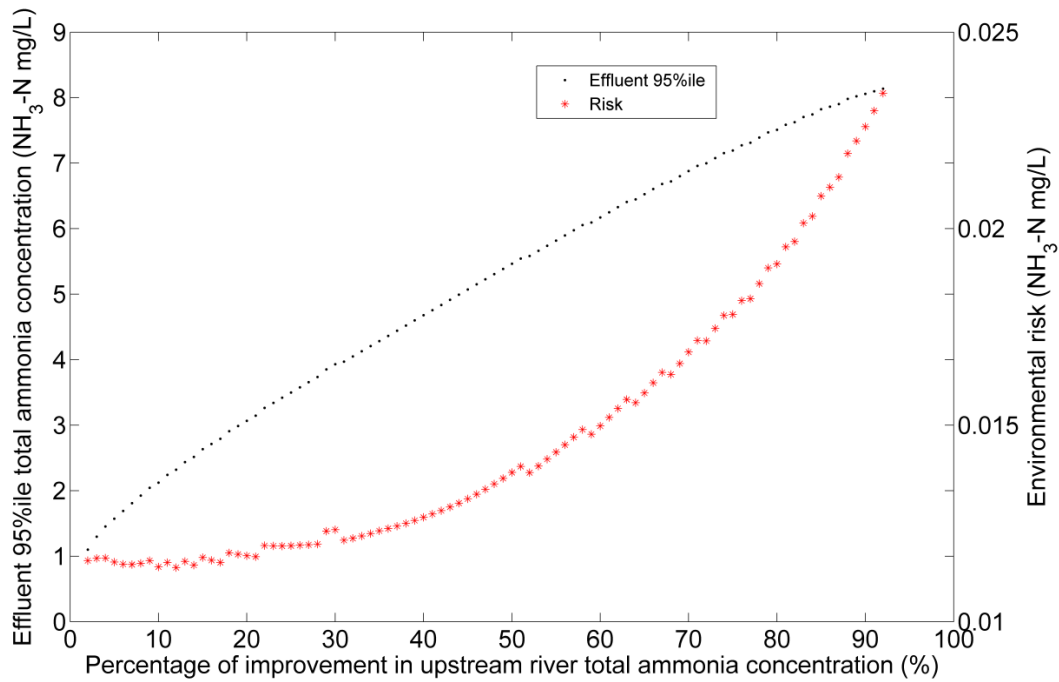


Figure 6.11 Change in effluent 95%ile total ammonia concentration required to meet the environmental standard and corresponding environmental risk value subject to different levels of improvement in upstream river water quality

As a five-year payback time is a widely used indicator (Feng Liu et al., 2012; Georges et al., 2009), the scaling of the right axis (capital cost for upstream improvement) of Figure 6.12 is set to be five times the left axis (savings in operational cost of WWTP) to facilitate the analysis. Two possible relationships of the benefit and cost functions are shown in Figure 6.12. For the first possible case where the two curves intersect (Figure 6.12a), it is cost-effective to implement a catchment-based regulation approach, as the investment for upstream improvement can be paid back within five years. The best solution is where the curve of operational cost saving exceeds the capital cost most, which in this case is at 7% upstream improvement as marked by a dashed line. For the latter case (Figure 6.12b), longer payback time than five years is needed. If this is acceptable, the cost-effective catchment-based permit should be determined at the point where the difference between the two curves is the smallest, which is at 5% upstream river improvement as marked by a dashed line. Note that if a rigid cost-benefit analysis is to be made, all costs and benefits as well as the discount for time value and adjustment for uncertainty and risk-attitude should be considered.

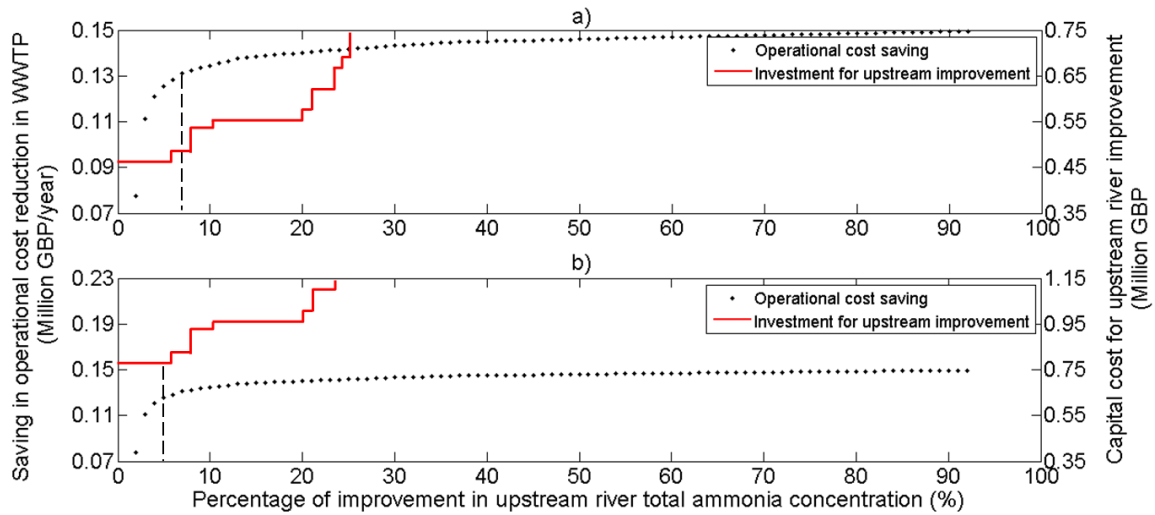


Figure 6.12 Cost-effectiveness analysis for catchment-based permitting when a) the investment can be paid back in five years; and b) the investment can be paid back by more than five years

Though the results of the two scenarios, expressed in percentages of upstream improvement, are very close, the implications for the pollution control measures are different. For the first scenario, the second level of control measure is more cost-effective, while the first level of control measure is advisable for the second scenario. The cost-effective catchment-based permits for the two scenarios are summarised in Table 6.4.

Table 6.4 Catchment-based permits for the two scenarios in Figure 6.12

Permit	Scenario A	Scenario B
WWTP effluent 95%ile total ammonia concentration (NH ₃ -N mg/L)	1.81	1.57
Aeration rate in WWTP (m ³ /d)	480,000	504,000
Upstream improvement	7%	5%
Upstream pollution control measure	Level 2	Level 1

6.4.2 Development of the Risk Function Based on Output Data from Dynamic Model

In the previous section, a risk function produced by the enhanced RQP was used to estimate the environmental impact of different operational strategies in the WWTP. The function was developed by using the assumption of RQP on how the mean and standard deviation of effluent water quality are changed during effluent discharge permitting, rather than using the effluent data associated with the operational strategies generated by dynamic models. In this section, the significance of this approximation on the permitting results is examined.

Figure 6.13a shows the probability distribution plots of effluent total ammonia concentration from two scenarios with the same operational strategies but different sludge pumping rates marked as the solid blue diamond and solid black dot in Figure 6.10, respectively. The variation in the sludge pumping operation results in a big change in not only the effluent 95%ile values as marked in the figure, but also the CV values, which is 0.68 (1.17 NH₃-N mg/L/1.73 NH₃-N mg/L) for the baseline scenario and 0.32 (0.22 NH₃-N mg/L /0.68 NH₃-N mg/L) for the other. If to reduce the effluent 95%ile of the baseline scenario to the same level as of the optimised sludge pumping scenario according to the RQP assumption, the standard deviation and mean need to be decreased from 1.17 NH₃-N mg/L and 1.73 NH₃-N mg/L to 0.32 NH₃-N mg/L and 0.47 NH₃-N mg/L, respectively (both with a 73% reduction). The probability distribution based on the assumed mean and standard deviation is plotted in Figure 6.13b as the thick black line, and the PDF of the optimised operation scenario in Figure 6.13a is also plotted in Figure 6.13b for comparison. Though having the same 95%ile value, the effluent PDF generated by the RQP assumption and the one based on dynamic simulation data are different, which would yield a gap in the prediction of environmental impact. As such, the environmental risk and river 90%ile total ammonia concentration are re-calculated for each scenario on the cost function ('cost-opt' in Figure 6.10) based on simulation data from the dynamic model. Results are presented in Figure 6.14 against the previously used functions. It can be seen that the environmental impacts were underestimated by the previous function. The updated results for 'P1' to 'P3' are listed in Table 6.5 with the previous values shown in brackets. Though the deviations are not significant, the gap becomes larger towards larger effluent water quality values.

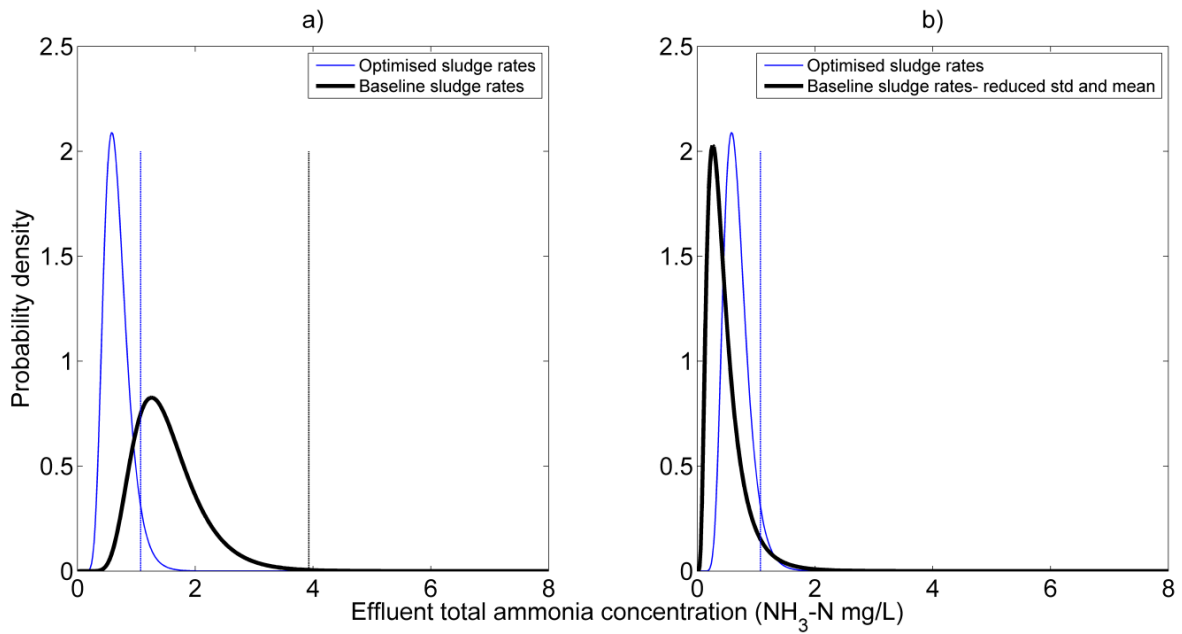


Figure 6.13 a) PDFs of two scenarios with different sludge pumping rates; and b) the modified PDF generated by RQP assumption from the PDF with baseline sludge rates to achieve the same 95%ile value as of the scenario with optimised sludge rates

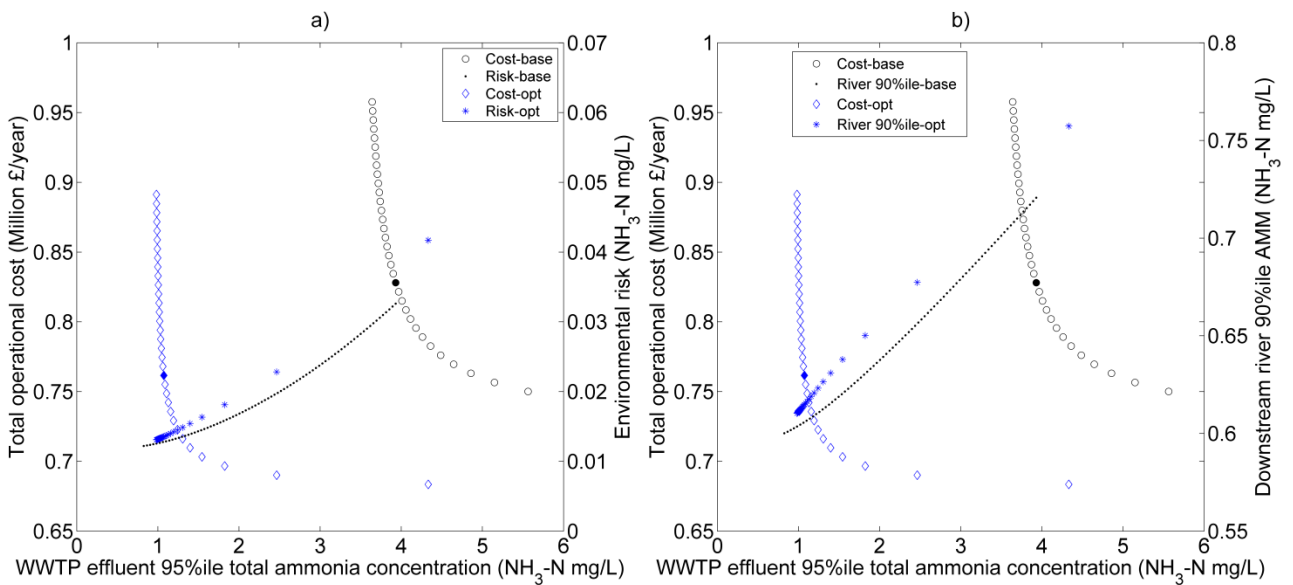


Figure 6.12 Reproduced a) environmental risk values and b) downstream 90%ile total ammonia concentration based on simulated effluent data from dynamic model

Table 6.5 Reproduced environmental risk values and river 90%ile total ammonia concentration values for 'P1', 'P2' and 'P3' in comparison with the previous values in Table 6.1

Performance indicator	P1	P2	P3
Environmental risk (10^{-3} NH ₃ -N mg/L)	13.1 (12.6)	13.6 (12.8)	16.3 (14.5)
Downstream river 90%ile total ammonia concentration (NH ₃ -N mg/L)	0.61 (0.60)	0.62 (0.61)	0.64 (0.62)

6.4.3 Uncertainty Analysis

The uncertainty in the dynamic wastewater system modelling was briefly discussed in section 4.5.2, yet it was not investigated by a detailed study due to the comprehensive efforts necessary for the analysis. The stochastic model RQP is much simpler than deterministic models. Still, there can be many sources of uncertainty. A primary analysis is made in this section to examine the impact of uncertainty in RQP modelling on the permitting results.

Uncertainty in model outputs, as mentioned in section 4.5.2, can result from imperfect knowledge in input data, model structure and parameters, computer implementation of the model, and the chaotic nature of natural phenomena. Three examples of uncertainty associated with RQP are listed below.

- a) The flow and water quality variables are described in RQP by lognormal distributions. Despite the fact that this is a widely used form in environmental engineering, the most accurate distribution type for specific cases might be different.
- b) As correlation coefficients need to be designated for input variables for RQP simulation, uncertainty may arise if default settings in the model are used without validation from case-specific monitoring data.
- c) Though 1-hour incremental time series of monitoring/modelling data have been used for the case study, applied data in most cases (especially on river water

quality) is much less detailed. This may result in great uncertainty due to the inadequate representation of the river/effluent flow.

A range of tools are available to characterise the uncertainty, such as error propagation equations, inverse modelling, scenario analysis, sensitivity analysis and Monte-Carlo simulation. A simple sensitivity analysis is used here to examine the impact of the three mentioned sources of uncertainty on the results of environmental risk. To be more efficient, the risk function is produced according to the procedure in section 6.3.2 rather than using simulated effluent data for the calculation of each risk value as described in section 6.4.2.

To examine the impact of the lognormal distribution assumption, the risk function was re-calculated using the best fitting distribution types in step I for upstream river flow and water quality and effluent flow rate, which are 3-parameter lognormal distribution, 4-parameter Dagum distribution and Cauchy distribution, respectively. The 2-parameter lognormal distribution is still used for effluent and downstream river water quality for the ease of permit calculation using RQP assumptions. The newly generated results on environmental risk and downstream river 90%ile total ammonia concentration are shown in Figure 6.14. The marginal difference to the previous results suggests the insignificant impact of the lognormal distribution assumption (at least in this case).

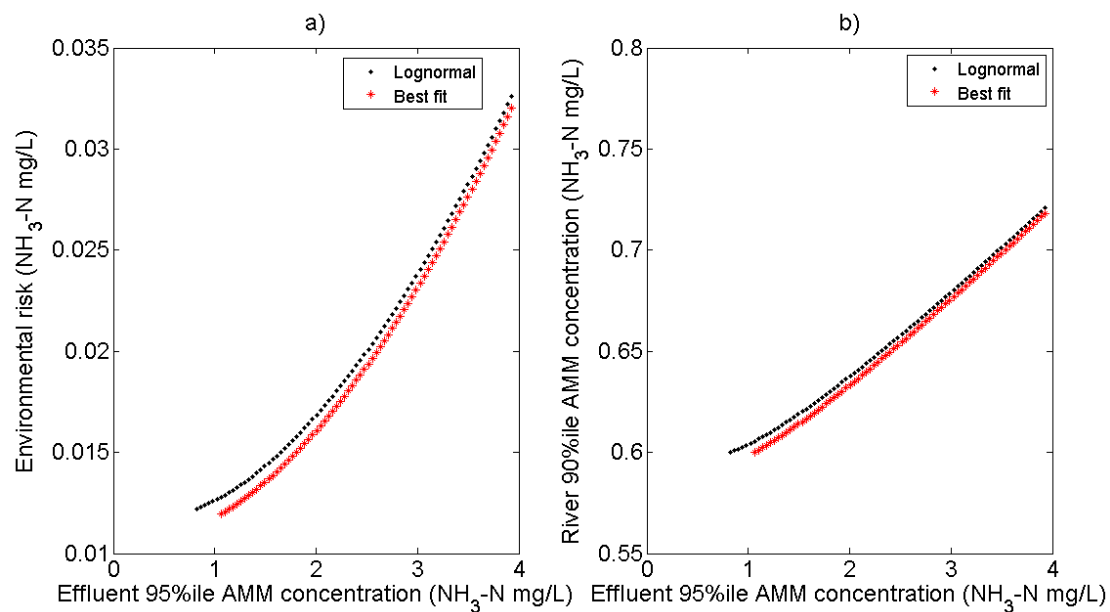


Figure 6.14 Comparison of a) risk functions and b) downstream river 90%ile total ammonia concentration produced assuming lognormal distribution and using best fitting distributions

Another test was made to change the correlation coefficient between the river flow rate and effluent discharge flow rate from 0.6 to 0.42 derived based on monitoring data. The re-produced results are presented in Figure 6.15, which show only minor deviations from the originally produced results.

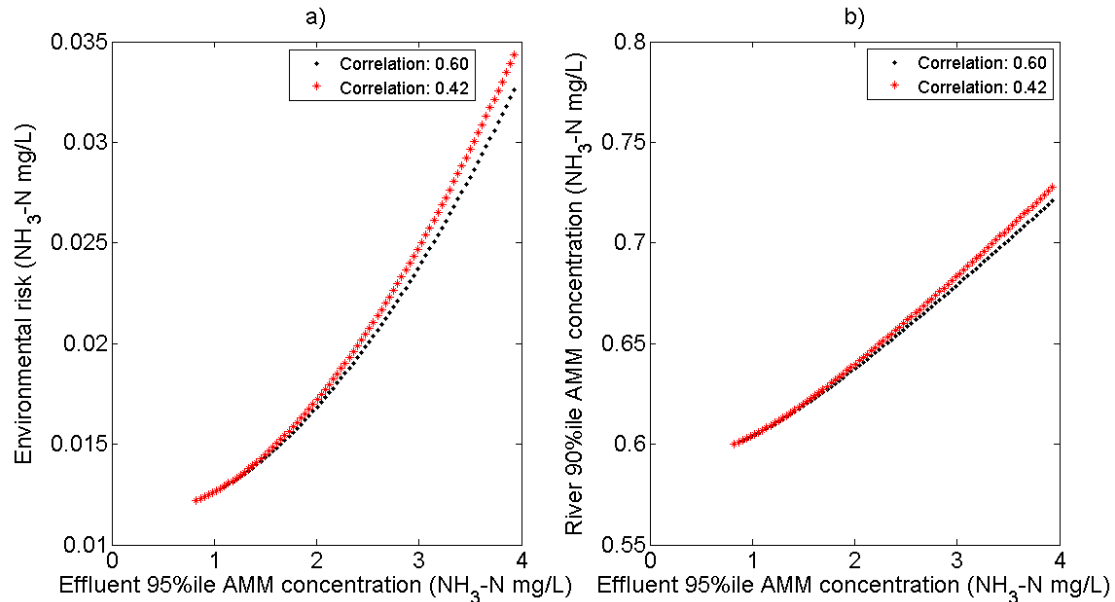


Figure 6.15 Comparison of a) risk functions and b) downstream river 90%ile total ammonia concentration produced assuming the river flow rate and effluent discharge flow rate are correlated with coefficients of 0.60 and 0.42

The third test was to examine the sensitivity of the results to the sampling frequency or time of the input data variables. Weekly and daily sampling scenarios were set up to simulate the less frequent sampling practice in real-life. To achieve this, only part of the 1-hour increment time series data sets were used for the calculation of the risk function. In this work, data records at 9 am every Monday or everyday were used. The re-generated risk functions based on daily and weekly sampling data are presented in Figure 6.16 along with the original one. It can be seen that the results based on weekly samples are more conservative than the other two, because: a) the starting points (i.e. the right end of the curves) have worse risk/river water quality values; and b) the left ends of the weekly frequency curves reach the y axis, suggesting that the environmental standard cannot be met even when the effluent total ammonia concentration is zero. However, it cannot be generalised that less frequent sampling would produce more conservative results. In this case, the permit based on hourly

sampling is more stringent than on daily sampling. Nevertheless, the results demonstrate the big influence of sampling frequency on the final results.

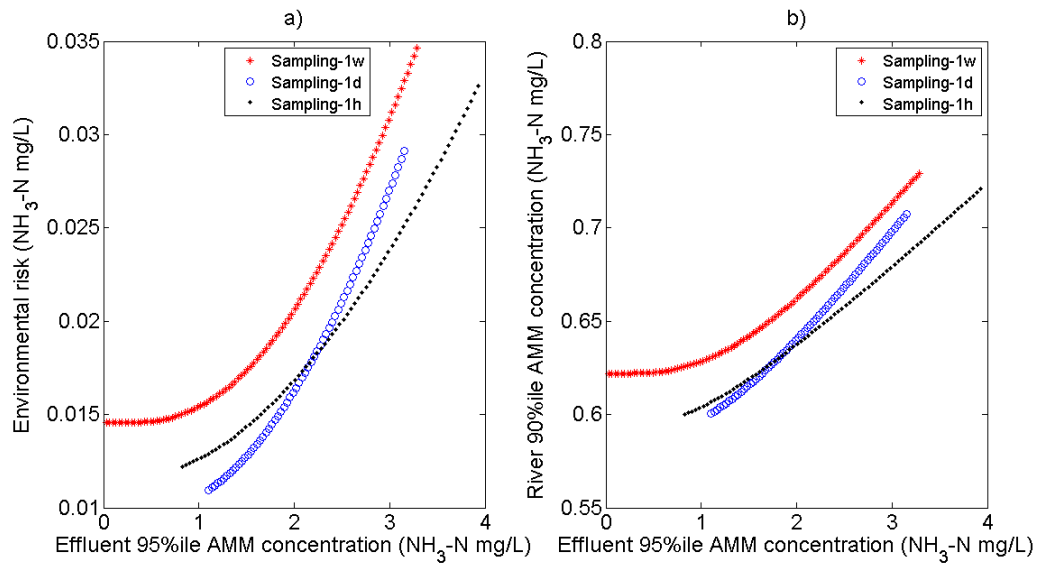


Figure 6.16 Comparison of risk functions (a) and downstream river 90%ile total ammonia concentration (b) produced based on weekly, daily and hourly sampling frequency

For daily sampling frequency, further scenarios are made to assess the influence of different sampling time to the final results by changing the sampling time to 5pm and 1am respectively, and the results are shown in Figure 6.17. It is suggested from the left ends of the curves that the environmental standard can be met based on daily samples taken at 9 am, but cannot if derived from samples taken at 1 am or 5 pm. This demonstrates the importance of considering sampling time in permitting.

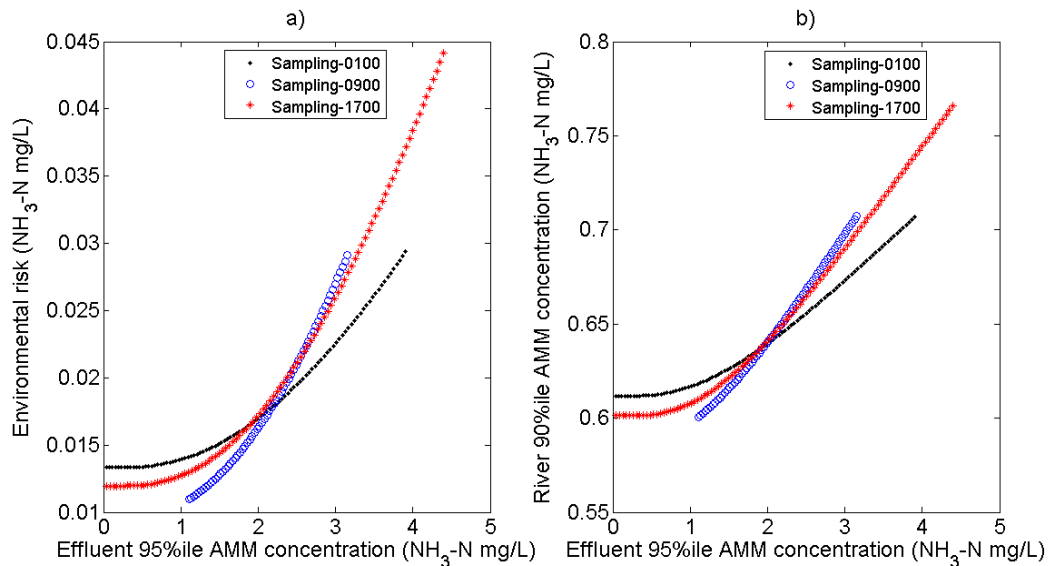


Figure 6.17 Comparison of risk functions (a)) and downstream river 90%ile total ammonia concentration (b)) produced based on daily samples taken at 1am, 9am and 5pm

6.5 Conclusions

A simpler decision analysis framework is introduced to optimise system operation and derive risk-based cost-effective permits accordingly. The integrated cost-risk framework for permitting is demonstrated to be a valuable tool in evaluating technical feasibility, economic efficiency and environmental impact of different operational compliance strategies. The trade-off analysis between operational cost and environmental risk facilitates the derivation of cost-effective effluent permitting. The permitting framework can potentially be extended to catchment-based permitting for more cost-effective environmental protection strategies in a wider scale. Through uncertainty analysis, sampling frequency and time were found to have a big impact on model results.

7 Roadmaps to Proposed Innovative Permitting Approaches

7.1 Introduction

By multi-objective optimisation of compliance strategies on dynamic modelling platforms, more rational decisions can be made based on the capacity of the wastewater system and trade-offs between various environmental and economic benefits. A performance-based permitting approach is demonstrated to be more effective and reliable in achieving balanced system performance than the traditional outcome-based end-of-pipe permitting. Three forms of performance-based permitting were introduced in Chapters 4-6 to derive the best permits which maximise the performance of the existing system by optimisation of operational or control strategy against multiple criteria whilst meeting the environmental standards. The three proposed methods as well as the traditional approach are shown in the order of increasing complexity from left to right in Figure 7.1. By employing more sophisticated modelling strategy or RTC schemes, the complexity rises so well as the potential benefit achievable. However, it does not necessarily mean the RTC-based permitting is the best permitting choice, as the cost and risk implications and practical issues for the implementation need also to be considered. This chapter investigates the advantages and disadvantages of the three proposed approaches and discusses the pathways for the implementation.

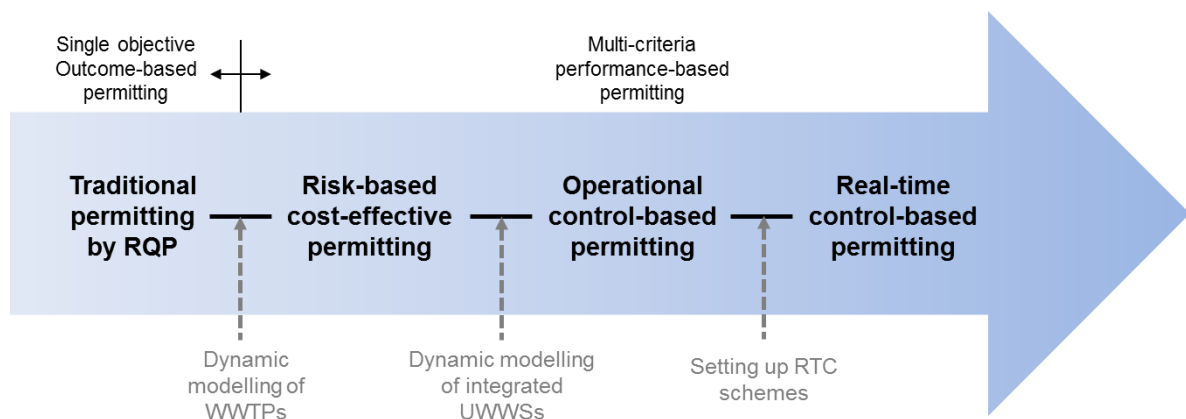


Figure 7.1 Illustration of the three proposed permitting approaches

In the following sections, the roadmaps for the implementation of the performance-based permitting approaches are outlined in section 7.2. Current practices such as performance-based regulation methods and self-monitoring schemes serve as good examples to visualise the new way of permitting. The

three proposed permitting approaches are then appraised in aspects of cost, risk and benefits. The core competencies of modern policy-making are also discussed in section 7.3 to assess the viability of the proposed methods as alternative regulation approaches.

7.2 Implementation of Performance-Based Permitting

7.2.1 Definition of Outcome-Based, Performance-Based and Prescriptive Regulation Approaches

Outcome-based and prescriptive regulation methods are the two most commonly used approaches across various disciplines (May, 2010; Office of the Australian Buildings Codes Board, 2000; Spady, 1994). For the wastewater industry, it is prevailing to practise the former approach, such as the setting of end-of-pipe standards for the control of wastewater discharges. By outlining a clear expectation on what needs to be accomplished, the outcome-based method is specific, observable and comparable. It encourages innovation and offers flexibility on the selection of pathways to achieve the goal. However, criticism has been raised on the inability of this approach to adequately quantify or measure the outcome (May, 2010), which in wastewater management can refer to the limitation in monitoring intermittent wastewater overflows and GHGs emitted from the treatment process.

In contrast, the prescriptive method is the practice of thinking and working in terms of means rather than ends. Applications of this approach are the regulation on wastewater treatment technologies, such as secondary treatment technology for domestic wastewater (Council of the European Communities, 1991a) and Best Available Technology (BAT) for industrial wastewater (Council of the European Union, 1996), and RTC dosing of UV disinfection (Environment Agency, 2011a) as reviewed in section 2.4.3. This approach is considered easy to follow as it clearly lays out what needs to be done, and simple to verify compliance for it can be visually confirmed during plan review and site inspections (Spataro et al., 2011). Yet, this regulatory method is insufficient to be practised alone for urban wastewater pollution control, as the wastewater inflow to the UWWS is highly dynamic and unpredictable thus satisfactory effluent quality cannot be guaranteed by just implementing a specific treatment technology.

Performance-based regulation approach falls within the spectrum between the two aforementioned methods, and has been used in building energy codes (Office of the Australian Buildings Codes Board, 2000; Spataro et al., 2011) (note that in some disciplines, performance-based method is another word for outcome-based method). Performance-based building energy codes contain broad, qualitative energy efficiency goals that require computer modelling to verify compliance. Building data are entered into preapproved modelling software and components and systems are manipulated until the desired efficiency outcome is met (Spataro et al., 2011). This is an expensive regulatory option, as it requires specialty software, trained modellers, and staff expertise in the regulatory agency to review modelling submittals in a meaningful way. Another challenge of this approach is the accuracy of the model in predicting the actual performance of the system, as model is often an incomplete or simplified description of the regulated system and assumption is often made on perfect installation and operation of equipment which is clearly not the case in real life. Nevertheless, this approach allows for innovation and promotes systems thinking in producing cost-effective compliance strategies.

The proposed operational strategy or control-based permitting approaches are performance-based, as models are employed to estimate the performance of different compliance strategies in achieving multiple goals. Yet as demonstrated in section 4.5.1, prescriptive regulation specifying the selected compliance strategy is a necessary complement to ensure the system operates as predicted by the model. The risk of inaccurate modelling can be to some extent addressed by incorporating uncertainty analysis in the decision-making process. Moreover, permits need to be timely reviewed and re-issued if necessary to accommodate to changes in the treatment works, catchment and/or climate.

7.2.2 Roadmaps to Performance-Based Permitting

Some current regulation practices provide good examples of how the performance-based permitting can be applied. As reviewed in section 2.4.3, RTC strategies of sewer system operation are already allowed by the UK permitting policy for the control of intermittent wastewater overflows. Sewer models are employed to derive the RTC strategies to meet emission-based standards on overflow spill frequency or environmental quality standard of the receiving water.

The computational tools are described in the regulation guidance as ‘invaluable design tools’ that can be used to ‘gain an understanding of the way in which the system works’ (Environment Agency, 2011a). As such, integrated UWWWS modelling could gain acceptance by the regulators, although simplified model versions, such as what proposed in Chapter 6, would increase the viability of practical application.

As it is the interest of the WWSPs to operate the wastewater systems in a reliable and cost-effective manner, they should take the initiative to apply for regulation under this new way of permitting. Moreover, the wastewater sector will need to take the responsibility to develop model of the regulated wastewater system and to self-monitor the system operation. As such, most of expense will fall to the WWSPs, yet this may still be a favourable permitting option if the estimated benefits exceed the costs. Detailed analysis on benefits and costs are provided in section 7.3.

Despite the greatly reduced regulatory burden by the shared responsibility from the WWSPs, efforts are also needed from the regulators to enforce and implement the comprehensive permitting approach, e.g. auditing of the wastewater system model, and setting up the measurement scheme for compliance analysis, etc. Some current regulation practices, as to be cited in this section, provide good examples on how effective management schemes can be set up. This section discusses how the performance-based permitting can be implemented at different stages of a permitting practice. Figure 7.2 shows a schematic summary of the procedure.

1) Preparation

The model for permitting is developed, validated and audited in the preparation stage. Historical monitoring data on rainfall, flow rate and water quality data on upstream river flow and WWTP influent and effluent and parameters of the wastewater system configuration, needs to be collected and processed to build the model. A different set of data should also be prepared for calibration and verification of the model.

Guidance on dynamic modelling of the wastewater system can be developed based on the state-of-the-art knowledge. Reports used to guide the data

collection and modelling of sewer systems for overflow discharge permitting are good reference examples (CIWEM, 2009; WaPUG, 1999). The instruction for auditing the sewer system modelling (Environment Agency, 2011) can be adapted for the appraisal of the dynamic wastewater system modelling.

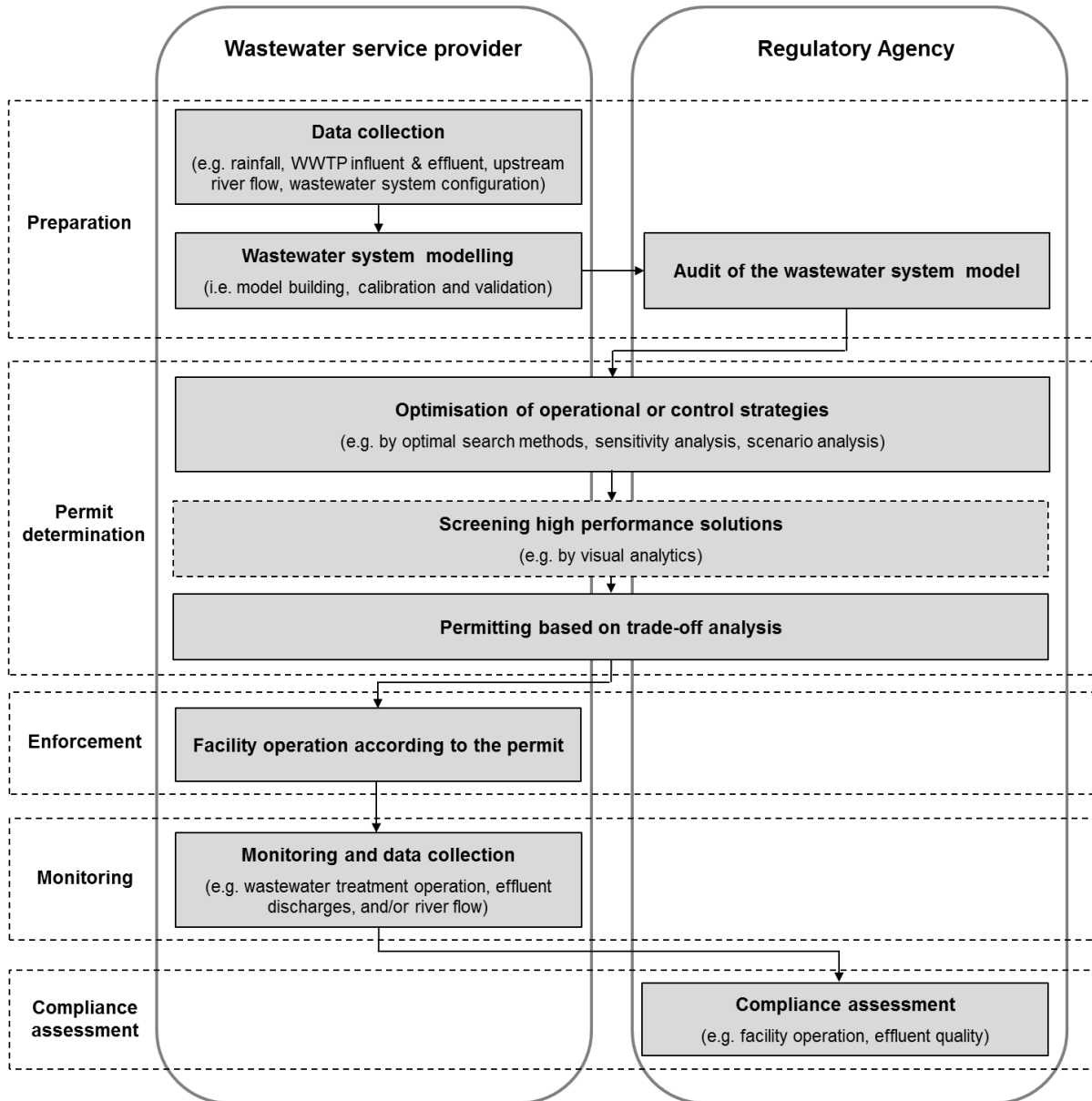


Figure 7.2 Implementation of the performance-based permitting approaches

2) Determination

Tools of varying complexity can be employed to optimise the operational or control strategy, such as NSGA-II, sensitivity analysis and scenario analysis as exercised in Chapters 4-6. For multi-objective optimisation problems, a screening procedure would be useful to identify desirable solutions among the diverse optimal options. As trade-offs exist, compromise is usually necessary to reach a final

decision. Stakeholder engagement would add value and facilitate informed decision-making.

As a typical wastewater discharge permit in the UK already has a section on 'operations' (see section 2.3 of Appendix A), the performance-based permit can be easily adapted to the current permit format by providing details of the operational or control strategy in the 'operating techniques' section.

3) Enforcement

If flexibility is allowed in the permit to vary the operational or control settings within narrow ranges (e.g. the permit example in Table 4.5), a choice needs to be made by the water service provider for daily operation. Once determined, it is essential to keep the operation as set and ensure a robust performance within a dynamic environment. Measures such as providing standby equipment and planning for emergency situations could increase the confidence of compliance.

4) Monitoring

Monitoring equipment needs to be installed following each regulated operational or control variable (e.g. pumps, blowers) to record performance of the equipment. To reduce the regulatory burden, a self-monitoring scheme similar to the Operator Self Monitoring (OSM) system in the UK can be introduced to make the operator, rather than the regulatory agency, responsible for collecting and analysing discharge effluent samples (Environment Agency, 2011a). The MCERTS scheme is set up by the Environment Agency to provide a framework of standards to ensure the monitoring can meet the requirements for compliance assessment and water quality planning. According to the scheme, the installation or upgrade of monitoring equipment should be inspected by a MCERTS inspector, and a suitable management system should also be in place for the monitoring (Environment Agency, 2011a). This scheme is currently used for flow monitoring of WWTP effluent discharge, but there is a potential to extend it to other flow or water quality-related monitoring.

5) Compliance assessment

Based on detailed monitoring data, the compliance of the permit can be assessed by examining whether the operational equipment runs properly. An

allowance can be made, such as a 5% deviation rate, if it does not result in severe consequences as reflected in the effluent water quality records. Though effluent water quality is not the key criteria for the assessment, it should be examined as well for it offers valuable information for post-construction evaluation of the effectiveness of the permitting decision. Monitoring data of good quality provides insights on how to improve the permitting process if needed.

7.3 Appraisal of Proposed Permitting Approaches

Regulatory impact assessment (RIA) is a required practice on policy makers in the UK and many other countries before taking action which has a regulatory impact on business (Regulatory Impact Unit, 2003). A key element of RIA is the assessment of cost, risk and benefits of a proposed regulatory measure to promote economic efficiency of policy. The assessment can be conducted by a comprehensive cost-benefit analysis which quantifies and monetises all aspects of costs and benefits and measures whether the benefits outweigh the costs or by a cost-effectiveness analysis if a simpler assessment is sufficient or certain benefits are not monetary. As it is time inhibitive to perform a comprehensive appraisal in this work, a qualitative and simplified assessment is carried out in section 7.3.1 to analyse the advantages and disadvantages of the proposed permitting approaches as compared to the traditional regulatory approach. The assessment against seven principles of modern policy-making is presented in section 7.3.2.

7.3.1 Impact Assessment of Cost, Risk and Benefit

Though the three proposed approaches differ in details, they are similar in many aspects when compared to the conventional permitting method. Table 7.1 summarises the cost, risk and benefits of the performance-based permitting associated with each stage of permitting.

It is evident that the newly developed permitting approaches are resource intensive due to the comprehensive permitting models and methods and monitoring (and control) devices required to be set up. Among the three proposed approaches, cost may vary greatly due to the different levels of complexity of the permitting method. Table 7.2 lists some major sources that contribute to the cost differences.

As the performance-based permitting relies on comprehensive wastewater system models, there is a risk that not all model parameters are identifiable which may result in misleading results. For RTC-based permit, compliance depends on the reliability of monitoring and control devices, thus the risk of control system failure (e.g. sensor fouling) should be controlled by adequate maintenance and regular calibration of the equipment.

Table 7.1 Appraisal of the proposed performance-based permitting

Permitting stage	Cost	Risk	Benefits
Preparation	Data collection for the development, calibration and validation of dynamic models	<ul style="list-style-type: none"> a) Model parameters not identifiable if the collected data are insufficient or not of good quality; and b) Low potential for operation or control optimisation 	<ul style="list-style-type: none"> a) Invaluable information provided by a well-calibrated model in understanding the system; and b) Potential use of the calibrated model for other purposes
Permit determination	<ul style="list-style-type: none"> a) Administrative burden on auditing the dynamic model; b) Regulatory efforts for the complex decision-making process; and c) Enhanced public participation 	<ul style="list-style-type: none"> a) Decision-making more time consuming; and b) Misleading results produced if the model is badly calibrated, or uncertainty in modelling not considered 	<ul style="list-style-type: none"> a) Enhanced transparency; b) Promotion of adoption of innovative operational or treatment technologies; and c) Enhanced stakeholder engagement in decision-making
Enforcement	<ul style="list-style-type: none"> a) Maintenance of equipment; and b) Purchase of standby equipment; and/or c) RTC devices 	Failure of equipment if not properly maintained and calibrated regularly	<ul style="list-style-type: none"> a) Increased clarity on what needs to be done; and b) Reduced energy cost; and c) Reduced environmental risk
Monitoring	<ul style="list-style-type: none"> a) Purchase of monitoring equipment; and b) Maintenance and calibration of monitoring equipment or RTC devices 	<ul style="list-style-type: none"> a) Failure of equipment if not properly maintained and calibrated regularly; and b) Lack of robustness/reliability of sensors for RTC 	Track of system performance and post-construction evaluation by sound monitoring data
Compliance assessment	Administrative efforts on interpreting and evaluating a large amount of monitoring data	Improper handling or misinterpretation of the monitoring data	Easier identification of reasons for under-performance assisted by more detailed monitoring data

Table 7.2 Sources for the difference in cost of the permitting approaches

	RTC-based permitting	Operational strategy-based permitting	Risk-based cost-effective permitting	Traditional permitting by RQP
Permitting model	Integrated UWWS model	Integrated UWWS model	Dynamic WWTP model + RQP	RQP
Optimisation technique	NSGA-II	NSGA-II	Sensitivity analysis + scenario analysis	--
Data collection for establishing the permitting model	<ul style="list-style-type: none"> a) Rainfall; b) Water quality of urban runoff; c) Flow rate and water quality of WWTP influent, effluent and internal flow within the treatment plant; and d) Upstream river flow rate and water quality 		<ul style="list-style-type: none"> a) Flow rate and water quality of WWTP influent, effluent and internal flow within the treatment plant; and b) Upstream river flow rate and water quality 	<ul style="list-style-type: none"> a) Flow rate and water quality of WWTP effluent; and b) Upstream river flow rate and water quality
Devices to be added	<ul style="list-style-type: none"> a) Water or air flow meters after each permitted control variable; and b) Sensors and controllers for the RTC system 		Water or air flow meters after each permitted operational variable	--

Despite the costs and risks in using complex wastewater system models (in particular integrated UWWS models), a well calibrated model is an invaluable tool in exploring cost-effective compliance strategies based on a comprehensive knowledge of the regulated system and an integrated view of the system performance in different aspects. The interactive permitting framework supported by the wastewater system model and optimisation and screening techniques ensures that the stakeholders' interests are incorporated in the decision-making process. Monitoring and data collection and processing are expensive as well and should be conducted with cautious and techniques. Yet effective monitoring not only tracks the actual performance of the system but also provides useful data in the post-construction evaluation of the effectiveness of pollution control measures.

7.3.2 Nine Principles of Modern Policy Making

With the increasingly complex, uncertain and unpredictable environment we live in and the rising expectations from the customers, the future policy making needs to adapt to the fast-moving and challenging environment to remain credible and effective. As stated in the section of the Modernising Government White Paper (Blair and Cunningham, 1999) covering better policy making: “*the Government expects more of policy makers: more new ideas, more willingness to question inherited ways of doing things; better use of evidence and research in policy making... this means developing a new and more creative approach to policy making*”. A subsequent report by the Cabinet Office on this subject *Professional Policy Making for the Twenty First Century* (Strategic Policy Making Team, 1999) identifies seven core competencies of professional policy making. The performance of the proposed permitting approaches against the seven principles is discussed below.

1) Long-term and forward looking

In traditional permitting, effluent discharge permits are developed based on historical data without forecast of the future. On the platform of dynamic wastewater system modelling, future challenges such as climate change and urbanisation can be represented and long-term environmental and economic performance of the system using different compliance strategies can be assessed through scenario analysis. As such, the proposed permitting approaches could develop permits based on long-term estimated performance of the system, thus contribute to the pursuit of sustainable development.

2) Outward-looking

This refers to policy making that takes account of factors in the national, European and international situation and communicates policy effectively. The proposed performance-based permitting promotes outward-looking policy making as reduction of GHG emissions is considered in the water pollution control regulation, which echoes the call for integrated and coherent policy-making at the European level (European Commission, 2014). Furthermore, stakeholders are engaged at an early stage and across all points of the permitting process. The enhanced involvement of stakeholders and the transparency of the permitting

process facilitate the communication of the regulation to audiences in the wider world beyond the civil service.

3) Innovation and creativeness

Being performance-based, the proposed permitting approaches are different from common regulations on wastewater pollution control which are outcome focused. The unconventional permitting method was found to be effective in delivering multiple environmental and economic benefits. However, closely tied to innovation is the issue of risk, and no pre-determined expectation can be made on how the programme will turn out. Therefore, risk should be identified, assessed and properly managed in the implementation of the newly developed regulation approaches. It would also be helpful to start from a field trial to test the idea and find out what works.

4) Use of evidence

The Government's declaration of 'what counts is what works' highlighted the role of evidence in policy-making. The evidence can be derived from a variety of sources, such as expert knowledge, existing domestic and international research, new research, stakeholder consultation, and evaluation of previous policies, etc. The permitting approaches developed in this work are based on state-of-art research in integrated UWWS modelling and multi-objective optimisation, and innovative decision-making frameworks were established to produce cost-effective compliance strategies and to feed evidence from stakeholder consultation to decision-making. A comparison analysis demonstrated the advantages of the performance-based regulation approach over the traditional way of permitting.

5) Inclusiveness

The proposed permitting approaches boost inclusiveness of policy-making by taking account of the impact on the needs of all those directly or indirectly affected by the policy in the decision-making process.

6) Joining up

In the UK, the environmental and economic behaviour of WWSPs are regulated by different departments (i.e. Environment Agency and Ofwat) or different sectors in the same department such as for the regulation on GHG emission and water pollution control. The multi-criteria decision-making framework provides an opportunity of joined-up policy making that crosses beyond institutional boundaries to achieve the Government's strategic objectives in a coordinated and coherent manner.

7) Learning lessons

Policy making should be a learning process which involves finding out from experience what works and what does not. The intensive monitoring scheme required by the proposed permitting approaches provides invaluable information for systematic evaluation of early outcomes. The newly learned evidence can be fed back to the cyclic permitting framework to update the optimal operational or control strategies.

8 Conclusions and Recommendations

8.1 Thesis Summary

The traditional end-of-pipe permitting policy on urban wastewater discharges is being challenged by the increasingly stringent environmental demands and the pursuit of cost-effectiveness. Flexible permitting approaches have been introduced to coordinate the regulation of wastewater discharges from WWTPs with other pollution sources in the catchment, and/or to allow tiered treatment intensity according to the dynamic environmental demand. However, the current permitting policy is still fragmented in the regulation of WWTP effluent discharges and CSO spills, and lacks coherence on the control of water pollution and GHG emissions.

The aim of this work was to explore innovative permitting policy from an integrated UWWS perspective based on optimal operation and control strategies rather than new treatment processes or technologies. Three permitting approaches were proposed and the advantages and disadvantages were analysed.

An integrated UWWS model from previous studies was used and modified to enable assessment of integrated operation/control strategies in long-term economic and environmental performance of the UWWS. Based on the established model and by using the multi-objective optimisation tool NSGA-II, the operation of the integrated system was optimised to reduce annual operational cost, variability in effluent water quality and environmental risk whilst meeting the environmental standards. Significant improvement was achieved in all three objectives simultaneously after the optimisation, which demonstrated the potential of integrated system operation in addressing environmental issues in a cost-effective manner. Furthermore, the results revealed trade-off relationships between the three objectives, showing the need to represent the interests of all stakeholders in the decision-making process. Yet to ensure the delivery of the balanced and best achievable benefits, permitting on the optimal operational strategies was found to be more reliable and effective than the conventional approach by end-of-pipe effluent limits.

The second proposed permitting approach was based on integrated RTC strategies to optimise the temporal allocation of treatment efforts. Compared to the first approach, further benefits were achieved by utilising the dynamic dilution capacity of the receiving water. A three-step permitting framework similar to that of the previous approach was established for the decision-making.

A similar but simpler permitting method than the first two approaches was also developed to facilitate the implementation of the innovative permitting to real-life. It was based on an integrated cost-risk decision-making framework coupling the use of the stochastic permitting model RQP with a dynamic model of the wastewater system. It requires less comprehensive modelling and optimisation resources and skills, thus may produce more proportional benefits in certain cases; or it can be used as an intermediate step before the full implementation of the other two approaches.

Lastly, the three proposed permitting approaches were appraised and compared in aspects of cost, risk and benefits, and the roadmaps for the implementation were also discussed.

8.2 Conclusions

The main findings with respect to the research chapters are summarised in this section.

1) Operational Strategy-Based Permitting

- Integrated UWWS modelling is an invaluable tool for environmental management, as it facilitates the exploration of sustainable wastewater management strategies by providing with a holistic view of the system performance in multiple aspects.
- To improve the environmental performance of an UWWS, optimisation of the system operation (in particular integrated operation) could be investigated first, as it can be a more cost-effective option than upgrade of the existing treatment process or introducing new treatment technologies.
- It is important to consider GHG emission in wastewater discharge permitting, as conflict was found between environmental water quality and carbon

emission from energy consumption in the operation of the UWWS. Trade-off also exists between stability of wastewater treatment process (which also affects the water quality status required by the WFD) and the proposed indicator environmental risk which is highly influenced by CSO spills, indicating the necessity to consider the interaction between the sewer system and the WWTP for a balanced outcome.

- Compared to the traditional end-of-pipe limits, the proposed operational strategy-based permitting approach is more cost-effective and reliable in delivering optimal and balanced performance of the integrated UWWS in both environmental water quality and GHG emissions. The four-step decision-making analysis framework facilitates the identification of desirable operational strategies by a) engaging stakeholders at all points of the decision-making process, b) embedding the stakeholders' interests in the optimisation and selection of high performing operational strategies, and c) using popular multi-objective optimisation algorithm and visual analytics tool to promote the efficiency of the permitting process.
- Permitting on value ranges of operational settings, derived by the established permitting framework, provides flexibility for real-life implementation without compromising the reliability of the system performance.
- The operational strategy-based permitting approach was found to be reliable as the permitted strategy remained optimality by using another input data set with heavier rainfall and worse river water quality.

2) Real Time Control-Based Permitting

- By applying integrated real-time aeration control, cost savings can be achieved from reducing air flow rate under favourable conditions and the environmental performance be enhanced by intensifying treatment efforts when the wastewater loading to the treatment plant is high or the assimilation capacity of the environment is low. Though operational cost and environmental risk can be simultaneously reduced compared to fixed system operation, there is a trade-off between the two objectives.

- It is time consuming to optimise the values of control variables and parameters of the controller algorithm all in an automatic way, thus heuristic methods can be incorporated to define parameter values to efficiently find satisfactory solutions. As such, the optimisation of the RTC strategies may be a cyclic process, as the values set by heuristic methods may need to be adjusted according to the initial optimisation results.
- The environmental standard limits (e.g. 90%ile and 99%ile concentration limits) are only partial descriptions of environmental water quality. It is necessary to use additional indicators for the evaluation of the RTC strategies to be environmentally protective. For example, if pollutant discharge load to the receiving waterbody is considered, real-time aeration control could provide limited benefits in cost savings as the air flow rate under moderate conditions cannot be reduced much if not to increase the annual pollutant discharge load.
- Application of the RTC technology under the traditional permitting regime may cause environmental deterioration as only 90%ile and 99%ile environmental pollutant concentration limits are assessed. Neither could it be appealing to the WWSP as the pursuit of cost efficiency may lead to permit violation; while to keep a high confidence level of permit compliance, cost savings can be quite limited.
- The proposed RTC-based permit encourages the adoption of cost-effective solutions. By the three-step decision-making analysis framework, the RTC strategies are optimised in terms of operational cost and environmental risk; cost-effective permit can then be determined by analysing the trade-off relationship between the economic and environmental benefits and selecting a solution that protects the environment without entailing excessive cost. As such, the newly developed regulatory approach promotes the uptake of the RTC technology, as the interest of the WWSP is considered in the decision-making and balanced environmental benefits can be achieved by enforcing the optimal RTC strategy developed based on systems thinking.

- The RTC-based permitting approach was found to be reliable as the permitted strategy presented better performance by using another input data set with heavier rainfall and worse river water quality.

3) Risk-Based Cost-Effective Permitting

- Cost-effective permits based on operational strategies can be derived by coupling the traditional permitting model RQP and a dynamic wastewater system model. Permit is derived by a decision-making analysis framework composed of three modules:
 - a) Risk calculation module. The enhanced RQP utilises the information already provided by the permitting model and calculates environmental risk as a function of effluent water quality.
 - b) Cost calculation module. It calculates the minimum operational costs associated with a series of effluent water quality through optimisation of operational strategies. The optimisation is conducted by sensitivity analysis and scenario analysis that do not require comprehensive skills for the use.
 - c) Cost-risk analysis module. Cost-effective permit is derived by integrating the risk and cost functions produced in the first two modules and evaluating the investment needed to achieve a certain target of risk reduction.
- The permitting framework can be extended for catchment-based permitting by adding a cost function on the investment needed for different levels of upstream river (to the UWWS) water quality improvement. It enables more cost-effective solutions by coordinating the treatment efforts of an UWWS with control measures for other pollution sources in a catchment.
- The frequency and timing of sampling on flow rate and water quality has a large impact on the permitting results. Thus to reduce the uncertainty in the final results, it is suggested to set up a more frequent (at least daily sampling) and automatic sampling scheme so that values out of the working hours can also be measured.

4) Roadmaps to Implementation of the Proposed Permitting Approaches

- The three proposed permitting approaches differ from the traditional end-of-pipe regulatory method in that it prescribes detailed compliance strategies. Though the conventional outcome-based approach offers more flexibility for the operation of the UWWS, the performance of the system cannot be well controlled other than effluent water quality. In contrast, the newly developed permitting approaches impose optimal operation or control strategies based on estimated system performance in multiple aspects over long-term simulation.
- Permitting on CSOs by dynamic sewer models and some other current regulatory practices can be learned for the application of the performance-based permitting approaches. Guidance on auditing of dynamic models can be developed to control the quality of the complex models; self-monitoring scheme, overseen by a programme similar to the MSCERTS scheme, can be set up to ensure reliable implementation of the required operational/control strategies.
- Increased investment would be needed to practise the performance-based permitting approaches due to the establishment of comprehensive dynamic models, more intensive monitoring schedule and more complex decision-making framework. Risk may arise if parameters in the comprehensive model are not identifiable, the monitoring equipment is not reliable, the monitoring data are not handled properly, or not all stakeholders are engaged in the decision-making process. Yet if the risks are properly controlled and managed, the performance-based permitting approaches can provide great benefits due to the potential of exploration of cost-effective solutions on the platform of integrated UWWS modelling, better knowledge of the process evolution by the detailed monitoring data and the enhanced stakeholder engagement.
- The proposed permitting methods are viable options of modern policy-making as it is based on *long-term and forward thinking* assisted by integrated UWWS modelling, *outward-looking* by considering the impact of GHG emission control in wastewater discharge permitting, *innovative and creative*

by promoting the adoption of cost-effective and innovative solutions, *evidence-based* by utilising expert knowledge, stakeholder consultation and state-of-art research, *inclusive* by enhanced stakeholder involvement, *joined-up* by promoting coherent and integrated regulation on water pollution and GHG emissions that crosses beyond institutional boundaries, and *learning from lessons* by the intensive monitoring scheme and the flexible decision-making framework.

5) Addressing the Four Challenges

This work demonstrates that a key strategy for the wastewater industry to adapt to the increasingly demanding regulatory and economic climate is to encourage application of technological advances by more flexible and holistic permitting policy. The proposed integrated operational/control-based permitting approaches have shown a potential to address the four challenges mentioned in Chapter 1 as follows.

- The environmental water quality can be improved by minimising the total impact of all wastewater discharges from an UWS to the environment.
- The GHG emissions, though not directly simulated and measured in the model, can be inferred by the cost entailed in the operation of the treatment works. Results in Chapters 4-6 suggest that carbon reduction can be achieved together with improvement of environmental water quality by better system operation though trade-off exists.
- The regulation on intermittent wastewater overflows is bolstered through enhanced operation of the sewer network by coordinating with that of the WWTP, so that the overall impact to the environment and cost is reduced.
- The RTC-based permitting, in particular, enables the WWSs to be responsive to internal/external changes to achieve desired system performance under an uncertain and rapidly changing environment.

The performance-based form of permitting, assisted by model simulation and multi-criteria analysis, can be applied to other types of technologies (e.g. resource recovery and recycling, innovative wastewater treatment technologies)

if further gains are sought and more radical change to the existing system is acceptable.

8.3 Recommendations for Future Work

A number of potential topics can be conducted following the work of this study, such as more detailed and full account of GHG emissions, innovative permitting on multiple pollutants, exploration of other types of RTC of the UWS, catchment-based permitting, more comprehensive uncertainty analysis, and field trial of the proposed ideas. Details are provided as follows.

1) Full Account of GHG Emissions in Wastewater Discharge Permitting

Due to the amount of work needed to extend the existing model to account for full GHG emissions, only indirect GHG emissions yielded from energy consumption for system operation are considered in this work. However, as revealed by a multi-objective optimisation of aeration control of a WWTP by Sweetapple et al. (2014a), partial emissions cannot represent the full amount of the GHG emissions and a control strategy that entails least operational cost may emit the largest amount of total carbon emissions. Hence, a full assessment of GHG emissions is necessary to understand the trade-off between GHG emissions, environmental water quality and operational cost, and to derive cost-effective permitting solutions accordingly.

2) Innovative Permitting on Multiple Pollutants

This study was conducted by using single pollutant total ammonia and did not consider the interactions with other pollutants. However, previous studies (Schütze et al., 2002) have demonstrated the conflict between the treatment efficiencies of different pollutants. A preliminary test by this work also showed the trade-off between total ammonia concentration and the level of COD and TSS in the effluent. Thus the application of the proposed permitting approaches to multiple pollutants is not a simple sum of permitting on single pollutants, but needs to consider and balance the intricate relationships between the pollutants.

3) Exploration of Other Types of RTC

Two tiered real-time aeration control was used for the investigation of RTC-based permitting in Chapter 5. It was found that the investigated form of RTC cannot achieve much cost savings without increasing the pollutant load discharged to the environment. To examine the cost-efficiency of RTC strategies, which is a major driver for the adoption of the technology, other control types should be investigated, such as dosage control for coagulation, sedimentation or addition of external carbon sources and flow control.

4) Catchment-Based Permitting

The opportunity for catchment-based permitting was discussed in Chapter 6 for the risk-based cost-effective permitting. It does not require modelling of other pollution sources in the catchment, but by estimation of the environmental impact and cost associated with the control measures. However, detailed models can also be established for other pollution sources (e.g. agricultural lands, UWWs, industrial plants), and permitting of operational or RTC strategies of the studied UWW can be coordinated with potentially the operation/control/treatment technology of other pollution sources in the catchment.

5) Comprehensive Uncertainty Analysis of the Innovative Permitting Approaches

Different forms of uncertainty analysis were conducted in Chapters 4-6 to test the reliability of the proposed methods, such as scenario analysis that uses another input data set and sensitivity analysis. As it is not the focus of the work, only primary analysis was made. Yet to get a deeper understanding of the uncertainty and reliability of the innovative permitting approaches, more comprehensive uncertainty analysis should be performed.

6) Pilot Scale and/or Full Scale Experiments

A field trial is the next logical step to test the idea and provide more confident information on cost and benefits. This would require the engagement and buy-in of the water sector.

Appendix A An Example Permit for Effluent Discharge and Storm Tank Overflow of a WWTP in England and Wales

1. Management

1.1 General management

1.1.1 The operator shall manage and operate the activities:

(a) in accordance with a written management system that identifies and minimises risks of pollution, including those arising from operations, maintenance, accidents, incidents, non-conformances and those drawn to the attention of the operator as a result of complaints; and

(b) using sufficient competent persons and resources.

1.1.2 Records demonstrating compliance with condition 1.1.1 shall be maintained.

1.1.3 Any person having duties that are or may be affected by the matters set out in this permit shall have convenient access to a copy of it kept at or near the place where those duties are carried out.

2. Operations

2.1 Permitted activities

2.1.1 The operator is only authorised to carry out the activities specified in Schedule 1 Table A.1 (the “activities”).

2.2 The site

2.2.1 The activities shall not extend beyond the site, being the land shown edged in green and the discharge(s) shall be made at the point(s) marked on the site plan at schedule 7 to this permit and as listed in Table A.3 (discharge points).

2.3 Operating techniques

2.3.1 For the activity A1 referenced in schedule 1, Table A.1 the operator shall comply with the relevant requirements of the Urban Waste Water Treatment (England and Wales) Regulations 1994.

2.3.2 For the discharge(s) specified in Table A.4:

(a) The discharge shall only occur when and only for as long as the flow passed forward is equal to or greater than the overflow setting indicated due to rainfall and/or snow melt.

(b) The off-line and/or storm tank storage capacity indicated must be fully utilised before a discharge occurs. It shall only fill when the flow passed forward is equal to or greater than the overflow setting indicated due to rainfall and/or snow melt and shall be emptied and its contents returned to the continuation sewer as soon as practicable.

(c) The discharge shall not be comminuted or macerated.

(d) The discharge shall have passed through screens as specified and shall not contain a significant quantity of solid matter with a particle size greater than any indicated. All screenings shall be removed from the discharge.

(e) Where a mechanically raked screen is installed a telemetry alarm system shall be installed and maintained so as to give the operator immediate notification of a failure of the screen raking mechanism, unless otherwise agreed in writing by the Environment Agency. The operator shall take all appropriate measures to return the screen raking mechanism to normal operation as soon as reasonably practicable after receipt for notification of the failure.

3. Emissions and monitoring

3.1 Emissions to water

3.1.2 The limits given in schedule 3 in Table A.2 shall not be exceeded.

3.1.2 For the emission limits in schedule 3 in Table A.2 to which this condition applies, if (a) unusual weather conditions were adversely affecting the operation of the sewage treatment works and (b) the operator has used appropriate measures to mitigate that adverse effect, no results of any sample of the discharge taken during that time shall be used in deciding whether or not the emission limit has been complied with.

3.1.3 For the emission limits in schedule 3 in Table A.2 to which this condition applies, if (a) abnormal operating conditions were adversely affecting the operation of the sewage treatment works and (b) the operator has used appropriate measures to mitigate that adverse effect, no result of any sample of

the discharge taken during that time shall be taken into account in deciding whether or not the emission limit has been complied with.

3.1.4 (a) If the measured Dry Weather Flow exceeds the permitted Dry Weather Flow limit then the operator shall, as soon as is practicable, investigate the reasons for the exceedance. The operator shall report the reasons for the exceedance to the Environment Agency and steps that it proposes to take to restore compliance. An exceedance of the Dry Weather Flow limit shall not be recorded as a failure if the operator takes appropriate steps to restore compliance;

(b) If the measured Dry Weather Flow exceeds the permitted Dry Weather Flow limit because of unusual rainfall during the 12-month period, then it will not be recorded as a failure of the Dry Weather Flow limit. For the purposes of this condition, unusual rainfall shall mean rainfall that causes significantly higher sewage flows during the three-month period that normally records the lowest flows;

(c) The permitted Dry Weather Flow limit is set at the operator's planned annual 80% exceeded flow;

(d) For compliance with this permit, the measured Dry Weather Flow is that total daily volume that is exceeded by 90% of the recorded measured total daily volume values in any period of 12 months; and

(2) For unusual rainfall to be considered, the operator shall notify the Environment Agency and provide supporting evidence as part of the normal specified data returns.

3.1.5 The limits in schedule 3 Table A.2 to which this condition applies may be exceeded where: in any series of samples of the discharge taken at regular but randomised intervals in any period of twelve consecutive months as listed in column 1 of Table A.6, no more than the relevant number of samples, as listed in column 2 of Table A.6, exceed the applicable limit for that relevant parameter. For relevant parameters subject to schedule 3C the assessment is based on a fixed calendar year from 1 January to 31 December inclusive.

3.2 Emissions of substances not controlled by emission limits

3.2.1 For the activity A1 in schedule 1, Table A.1 the operator shall take appropriate measure to minimise so far as reasonably practicable the polluting effects of the emissions of substances not controlled by emission limits (excluding odour).

3.3 Monitoring

3.3.1 The operator shall, unless otherwise agreed in writing by the Environment Agency, undertake the monitoring specified in the following tables in schedule 3 to this permit:

(a) point source emissions specified in Table A.2 and A.5;

(b) inlet quality specified in Table A.2 and A.5.

3.3.2 The operator shall maintain records of all monitoring required by this permit including records of the taking and analysis of samples, instrumental measurements (periodic and continual), calibrations, examinations, tests and surveys and any assessment or evaluation make on the basis of such data.

3.3.3 Monitoring equipment, techniques, personnel and organisations employed for the emissions monitoring programme and the environmental or other monitoring specified in condition 3.3.1 shall have either MCERTS certification or MCERTS accreditation (as appropriate), where available, unless otherwise agreed in writing by the Environment Agency.

3.3.4 Permanent means of access shall be provided to enable sampling/monitoring to be carried out at the monitoring points specified in schedule 3 Table A.5 unless otherwise agreed in writing by the Environment Agency.

3.3.5 The monitoring programme for the parameters subject to schedule 3B shall be:

(a) pre-scheduled to cover a calendar year and the programme recorded before the start of a calendar year sample period; and

(b) spot samples collected at approximately equal intervals during the year, including samples from different days of the week and different times. Approximately 10% of samples should be outside the normal sampling window which is 9am – 3pm, Monday to Friday.

3.3.6 After becoming aware, or following a notification that a sample has not been taken on the schedule 3B Monitoring Programme pre-scheduled date, or is lost, or a result for that sample cannot be reported, the operator shall record the details and reschedule the sample.

3.3.7 The monitoring programme for the parameters subject to schedule 3C shall be pre-scheduled before each calendar year. Samples must be collected at approximately equal intervals during the year from different days of the week and approximately 10% of samples should be taken at weekends.

4. Information

4.1 Records

4.1.1 All records required to be made by this permit shall:

- (a) be legible;
- (b) be made as soon as reasonably practicable;
- (c) if amended, be amended in such a way that original and any subsequent amendments remain legible, or are capable of retrieval; and
- (d) be retained, unless otherwise agreed in writing by the Environment Agency, for at least 6 years from the date when the records were made.

4.1.2 The operator shall keep on site all records, plans and the management system required to be maintained by this permit, unless otherwise agreed in writing by the Environment Agency.

4.2 Reporting

4.2.1 The operator shall send all reports and notifications required by the permit to the Environment Agency using the contact details supplied in writing by the Environment Agency.

4.2.2 Within 28 days of the end of the reporting period the operator shall, unless otherwise agreed in writing by the Environment Agency, submit reports of the monitoring and assessment carried out in accordance with the conditions of this permit, as follows:

- (a) in respect of the parameters and monitoring points specified in schedule 4;

(b) giving the information from such results and assessments as may be required by the forms specified in the table.

4.3 Notifications

4.3.1 The Environment Agency shall be notified without delay following the detection of:

(a) any malfunction, breakdown or failure of equipment or techniques, accident, or emission of a substance not controlled by an emission limit which has caused, is causing or may cause significant pollution;

(b) the breach of a limit specified in schedule 3 Table A.2 (including individual exceedances of limits which are covered by condition 3.1.5); or

(c) any significant adverse environmental and health effects.

4.3.2 Any information provided under condition 4.3.1 shall be confirmed by sending the information listed in schedule 5 to this permit within the time period specified in that schedule.

4.3.3 Where the Environment Agency has requested in writing that it shall be notified when the operator is to undertake monitoring and/or spot sampling, the operator shall inform the Environment Agency when the relevant monitoring and/or the spot sampling is to take place. The operator shall provide this information to the Environment Agency at least 14 days before the date the monitoring is to be undertaken.

4.3.4 The Environment Agency shall be notified within 14 days of the occurrence of the following matters, except where such disclosure is prohibited by Stock Exchange rules:

Where the operator is a registered company:

(a) any change in the operator's trade name, registered name or registered office address; and

(b) any steps taken with a view to the operator going into administration, entering into a company voluntary arrangement or being wound up.

Where the operator is an incorporate body other than a registered company:

(a) any change in the operator's name or address; and

(b) any steps taken with a view to the dissolution of the operator.

4.3.5 For the activity A1 referenced in Schedule 1, Table A.1, where the operator proposes to make a change in the nature of the activity by increasing the concentration of, or the addition of, or allowing the introduction of, a pollutant to the activity to an extent that the activity may be liable to cause pollution and the change is not permitted by emission limits specified within schedule 3 Table A.2 or the subject of an application for approval under the EP Regulations or this permit:

(a) the Environment Agency shall be notified at least 14 days before the increase or addition or allowing the introduction; and

(b) the notification shall contain a description of the proposed change in operation.

4.4 Interpretation

4.4.1 In this permit the expressions listed in schedule 6 shall have the meaning given in that schedule.

4.4.2 In this permit references to reports and notifications mean written reports and notifications, except where reference is made to notification being made “without delay”, in which case it may be provided by telephone.

Schedule 1 – Operations

Table A.1 Activities

Activity reference	Description of activity	Limits of specified activity
A1	Discharge of final effluent via Outlet 1	N/A
A2	Discharge of settled storm sewage via Outlet 2	N/A

Schedule 2 – Waste types, raw materials and fuels

Wastes are not accepted as part of the permitted activities and there are no restrictions on raw materials or fuels under this schedule.

Schedule 3 – Emissions and monitoring

Table A.2 Point source emissions to water (other than sewer) – emission limits and monitoring requirements

Parameter	Limit (including unit)	Reference period	Limit of effective range	Monitoring frequency	Compliance statistic
Dry weather flow	1800 m ³ /d	Total daily volume	N/A	Continuous	(Condition 3.1.4 applies)
15-minute instantaneous or averaged flow	No limit set. Record as L/s	15 minutes	N/A	Continuous	N/A
ATU-BOD as O ₂	15 mg/L	Instantaneous (spot sample)	N/A	As specified in schedule 3B	Look up table (Conditions 3.1.2 and 3.1.5 apply)
ATU-BOD as O ₂	50 mg/L	Instantaneous (spot sample)	N/A	As specified in schedule 3B	Maximum (Conditions 3.1.2 applies)

Table A.3 Discharge points

Effluent name	Discharge point	Discharge point NGR	Receiving water/Environment
Final effluent	Outlet 1	ST XXXXX XXXXX	River X
Settled storm sewage	Outlet 2	ST XXXXX XXXXX	Tributary of the River X

Table A.4 Storm sewage discharge settings

Emission	Description of discharge	Overflow setting L/s	Maximum size of solid matter	Screen aperture size	Minimum screen capacity flow L/s	Storm tank/storage capacity m ³
Settled storm sewage via Outlet 2	Settled storm sewage	70	No greater than 6 mm in more than 1 dimension	6 mm x 6mm	N/A	452 off-line

Table A.5 Monitoring points

Effluent(s) and discharge point(s)	Monitoring type	Monitoring point NGR	Monitoring point reference
Final effluent via Outlet 1	Effluent sampling	ST XXXXX XXXXX	M1
Final effluent via Outlet 1 (UWWTD)	Effluent sampling (UWWTD)	ST XXXXX XXXXX	M1
Settled storm sewage via Outlet 2	Effluent sampling	ST XXXXX XXXXX	M2
Final effluent via Outlet 1	Flow sampling	ST XXXXX XXXXX	M3

Schedule 3A – Look up table

Table A.6 Look-up table for compliance analysis of 95%ile permit limits

Series of samples taken in any year	Maximum permitted number of samples which fail to conform
4 – 7	1
8 – 16	2
17 - 28	3
29 - 40	4
41 - 53	5
54 – 67	6
68 – 81	7
82 – 95	8
96 – 110	9
111 – 125	10
126 – 140	11
141 – 155	12
156 – 171	13
172 – 187	14
188 – 203	15
204 – 219	16
220 – 235	17

Series of samples taken in any year	Maximum permitted number of samples which fail to conform
236 – 251	18
252 – 268	19
269 – 284	20
285 – 300	21
301 – 307	22
318 – 334	23
335 – 350	24
351 - 365	25

Schedule 3B – Opra tier 3 sampling frequency

Parameter	'Normal frequency' of samples per year	Reduced sampling frequency after 12 consecutive months of numeric permit compliance, samples per year or pro rata over the remainder of a year	On numeric permit failure return to normal frequency as soon as reasonably practicable, samples per 12 months	Out of hours samples
Sanitary	24	12	24	For 24 samples 2 out of hours samples per annum
Non sanitary	12	12	12	For 12 samples 1 out of hours sample per annum

Schedule 3C – Urban Waste Water Treatment Directive sampling frequency

Population equivalent	Samples per year	Reduced sampling frequency after a year of compliance with the UWWTD numeric limits, samples per year	On UWWTD numeric limit failure return to the higher frequency in the year that follows, samples per year
2,000 to 9,999	12	4	12
10,000 to 49,999	12	N/A	N/A
50,000 or over	24	N/A	N/A

Schedule 4 – Reporting

Parameters, for which reports shall be made, in accordance with conditions of this permit, are listed below.

Parameter	Monitoring point reference	Reporting period	Period begins
Dry Weather Flow	M3	Annually	1 January
UWWTD – BOD and COD	M1	Monthly	1st of month
Operator Self Monitoring – BOD, ammonia, suspended solids	M1	Quarterly	1st of month
Operator Self Monitoring – BOD, ammonia, suspended solids	M1	Annually	1 January

Schedule 5 – Notification

[This schedule outlines the information that the operator must provide.]

Schedule 6 – Interpretation

‘Accident’ means an accident that may result in pollution.

... ..

Schedule 7 – Site plan

[Description of the location of the discharge points]

[Figure showing the boundary of the site for the activities and the location of the discharge points]

Appendix B Emission-Based and Environmental Quality-Based Standards for Urban Wastewater Discharges in England and Wales

Table B.1 The UWWTD requirements for discharges from WWTPs under secondary treatment processes

Parameters	Concentration	Minimum percentage of reduction
Biochemical oxygen demand (BOD ₅ at 20 °C) without nitrification	25 mg/L O ₂	70 - 90
Chemical oxygen demand (COD)	125 mg/L O ₂	75
Total suspended solids ¹	35 mg/L (For high mountain regions over 1500 m above sea level, the limit is 35 mg/L for areas with more than 10,000 p.e. and 60 mg/L for 2000 – 10,000 p.e.)	90 (For high mountain regions over 1500 m above sea level, the limit is 90 for areas with more than 10,000 p.e. and 70 for 2000 – 10,000 p.e.)

Note: ¹This requirement is optional and is not adopted in England and Wales.

Table B.2 Additional requirements by UWWTD for discharges from WWTPs under more stringent treatment processes

Parameters	Concentration	Minimum percentage of reduction
Total phosphorus (TP)	2 mg/L P (10,000-100,000 p.e.) 1mg/L P (more than 100,000 p.e.)	80
Total nitrogen (TN)	15 mg/L N (10,000-100,000 p.e.) 10 mg/L N (more than 100,000 p.e.)	70 - 80

Table B.3 The 90 and 99 percentile limits for BOD₅ in England and Wales

	90 percentile (mg/L)	99 percentile (mg/L)
WFD high status for type 1, 2, 4 and 6 and salmonid	3	7
WFD good status or types 1, 2, 4 and 6 and salmonid and high status for types 3, 5 and 7	4	9
WFD good status for types 3, 5 and 7	5	11
WFD moderate status for types 1, 2, 4 and 6 and salmonid	6	14
WFD moderate status for types 3, 5 and 7	6.5	14
WFD poor status for types 1, 2, 4 and 6 and salmonid	7.5	16
WFD poor status for types 3, 5 and 7	9	19

Table B.4 The 90 and 99 percentile limits for total ammonia and unionised ammonia in England and Wales

	Total ammonia (NH ₃ -N mg/L)		Unionised ammonia (NH ₃ -N mg/L)
	90 percentile	99 percentile	99 percentile
WFD high status for type 1, 2, 4 and 6	0.2	0.5	0.04
WFD good status or types 1, 2, 4 and 6 and high status for types 3, 5 and 7	0.3	0.7	0.04
WFD good status for types 3, 5 and 7	0.6	1.5	0.04
WFD moderate status for types 1, 2, 4 and 6	0.75	1.8	0.04
WFD moderate status for types 3, 5 and 7 and WFD poor status for types 1, 2, 4 and 6	1.1	2.6	0.04
WFD poor status for types 3, 5 and 7	2.5	6.0	No value

Table B.5 Fundamental intermittent standards for un-ionised ammonia concentration/duration thresholds not to be breached more frequently than shown

a) Ecosystem suitable for sustainable salmonid fishery

Return period	Un-ionised ammonia concentrations (NH ₃ -N mg/L)		
	1 hour	6 hours	24 hours
1 month	0.065	0.025	0.018
3 months	0.095	0.035	0.025
1year	0.105	0.040	0.030

b) Ecosystem suitable for sustainable cyprinid fishery

Return period	Un-ionised ammonia concentrations (NH ₃ -N mg/L)		
	1 hour	6 hours	24 hours
1 month	0.150	0.075	0.030
3 months	0.225	0.125	0.050
1year	0.250	0.150	0.065

c) Marginal cyprinid fishery ecosystem

Return period	Un-ionised ammonia concentrations (NH ₃ -N mg/L)		
	1 hour	6 hours	24 hours
1 month	0.175	0.100	0.050
3 months	0.250	0.150	0.080
1year	0.300	0.200	0.140

Table B.6 Fundamental intermittent standards for dissolved oxygen concentration/duration thresholds not to be breached more frequently than shown

a) Ecosystem suitable for sustainable salmonid fishery

Return period	Dissolved oxygen concentrations (mg/L)		
	1 hour	6 hours	24 hours
1 month	5	5.5	6
3 months	4.5	5	5.5
1year	4	4.5	5

b) Ecosystem suitable for sustainable cyprinid fishery

Return period	Dissolved oxygen concentrations (mg/L)		
	1 hour	6 hours	24 hours
1 month	4	5	5.5
3 months	3.5	4.5	5
1year	3	4	4.5

c) Marginal cyprinid fishery ecosystem

Return period	Dissolved oxygen concentrations (mg/L)		
	1 hour	6 hours	24 hours
1 month	3	3.5	4
3 months	2.5	3	3.5
1year	2	2.5	3

Appendix C Effluent Water Quality Standards of Wastewater Discharges in the United States

Table C.1 Secondary treatment standards in the United States

Parameters	30-day average	7-day average
BOD ₅	30 mg/L O ₂ (or 25 mg/L CBOD ₅)	45 mg/L O ₂ (or 40 mg/L CBOD ₅)
TSS	30 mg/L	45 mg/L
Percentage of BOD ₅ and TSS removal (concentration)	No less than 85%	
pH	Within the limits of 6.0 – 9.0 ¹	

Note: ¹Unless the WWTP demonstrates that: 1) inorganic chemicals are not added to the waste stream as part of the treatment process; and 2) contributions from industrial sources do not cause the pH of the effluent to be less than 6.0 or greater than 9.0.

Table C.2 Equivalent to secondary treatment standards

Parameters	30-day average	7-day average
BOD ₅	45 mg/L O ₂ (or 40 mg/L CBOD ₅)	65 mg/L O ₂ (or 60 mg/L CBOD ₅)
TSS	45 mg/L	65 mg/L
Percentage of BOD ₅ and TSS removal (concentration)	No less than 65%	
pH	Within the limits of 6.0 – 9.0 ¹	

Note: ¹Same requirements as in Table C.1.

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