Reservoir Released Waves as a Transient Riverine Pollution Mitigation Tool

An examination of the feasibility of using waves of water released from a reservoir as a response to pollution spills.

Simon DeSmet

Submitted in accordance with the requirements for the degree of
Doctor of Philosophy

The University of Leeds

School of Geography

December 2014

The candidate confirms that the work submitted is his/her own and that appropriate credit has been given where reference has been made to the work of others.

Acknowledgements

I have depended on God and faith to carry me though the preparation of this Thesis.

A number of individuals have been of great assistance over the past few years. Firstly my two primary supervisors Dr Paul Kay and Dr Jonathan Carrivick have been immensely patient, taught me to write properly, vetted my ideas and have provided great wisdom and experience.

Technical assistance with the Fluent software was provided by Dr Andy Sleigh and Dr Carl Gikeson. Access to the SORBY flume facility and input on ideas for the flume study were given by Dr Daniel Parsons and Dr Gareth Keevil. For help in designing the field study and collecting the data I would like to thank David Ashley, Rachel Gasior, Dr Pengfei Li, Dr Ben Gillespie and numerous individuals a the Environment Agency. Additionally I owe a thanks to Rachel Homer for assistance with statistics.

Yorkshire Water Services have provide funding, site access, and industrial experience that has been invaluable. In particular I owe thanks to Mark Tinsdeall, James Kitson, David Townend, Joanne Baxter, Simon Wolmersley, and Jenny Banks.

Finally Jacqui Manton, Jacqui Brown, Dr John Gibbons, Dr David Romero, Dr Balazs Kulik, Dr Gemma Dooling and Dr Carol White have all supported me in innumerable ways.

[7] *All the rivers run into the sea; yet the sea is not full; unto the place from whence the rivers come, thither they return again. Ecclesiastes 1:7*

Abstract

Pollution spills are a serious threat to rivers causing fish kills, and lasting damage to the biological systems of rivers. At present financial penalties are used to deter such incidents but no mitigation system exists. A multiple method approach involving a hydrological and water chemistry based field study, a flume tank experiment, a computer fluid dynamics model and a 1D flow model was under taken to examine the feasibility of using a wave of water released from a reservoir to dilute pollution spills.

Nine waves were released from reservoirs on three different catchments. Water quality was measured downstream of sewage treatment works and the progress of the wave was tracked down the river with gauges. Additionally in one experiment a slug of rhodamine dye was released into the river ahead of a wave. To understand the impact of a wave on mixing processes within the water column, in particular longitudinal dispersion, a series of flume tank and computer fluid dynamics experiments were ran. In both experiment sets a wave was released from one end of a tank and a slug of either rhodamine dye, or tracked particles was released mid tank, the interaction between the wave and the dye was then captured as footage and analysed.

Across the three sets of experiments waves were found to move significantly faster than the baseflow with mean velocities ranging between $0.86ms^{-1}$ $1.63ms^{-1}$. In the Holme River a dye slug would be caught within 3 hours and 46 minutes. Catch up times and response times were both demonstrated with a dye test and estimated with a 1D model providing management focused results previously unreported in the literature. Dilution of water quality parameters including NH_4 and conductivity was recorded during wave passage at the sewage treat works outflows in the majority of experiments. Peak dilutions of 59% for NH_4 and 58% for conductivity were recorded. An increase in longitudinal dispersion with wave magnitude was observed in the computer fluid dynamics model but unclear within the flume tank.

Table of Contents

Chapter 1. Introduction 1
1.1 Thesis Structure 1
1.2 Terminology 2
1.3 Key Questions 3
1.4 Aims 6
Chapter 2. Literature Review 7
2.1 The Importance of Water Quality 7
2.2 The Nature of Pollution 8
2.3 The Focus of this Study: Pollution 12
2.4 The Conceptual Solution 18
2.5 Conclusion 22
Chapter 3. Reservoir Release Experiments: The Field Study Approach 23
3.1 Introduction 23
3.2 Literature 23
3.3 Aims 26
3.4 Methodology 27
3.5 Results 46
3.6 Discussion 74
3.7 Conclusions 82
Chapter 4. The Flume Study Approach 83
4.1 Introduction 83
4.2 Literature and Background 83
4.3 Aims and Objectives 87
4.4 Methodology 89
4.5 Results 100
4.6 Discussion 119
4.7 Conclusion 126
Chapter 5. Computer Fluid Dynamics Approach 128
5.1 Introduction 128
5.2 Literature 128
5.3 Aims 132
5.4 Methodology 133
5.5 Results 146
5.6 Discussion 157

5.7 Conclusion 162

Chapter 6. Discussion and Application 164

6.1 Key Question 1; How quickly can a wave of water released from a reservoir catch a slug of polluted water? 164

6.2 Key Question 2; How much dilution can be achieved? 166

6.3 Key Question 3, What Mixing Processes Occur When a Wave Catches Polluted Water? 169

6.4 The 1D model 171

6.5 Key Lessons of the Work 175

6.6 Challenges to using reservoirs as a mitigation tool 181

6.7 Conclusion and Evaluation 185

List of Tables

Table 3-1 Rivers, and reservoirs used either in this study, or within the literature. Release is the maximum release magnitude recorded for the reservoir in question. Distance to measurement is the distance between the reservoir and the farthest downstream water quality measurement site used in each study. 28

Table 3-2. The reservoir release program employed in the current study. Program refers to the release design and purpose. An explanation of each is given below. 35

Table 3-3. Water quality parameters on the YSI sonde device and the instrumentation used to measure them in the current study.................. 41

Table 3-4. The travel times and velocities for the wave in each experiment first between the reservoir and the STW, and then between the STW and the Queens Bridge site at the bottom of the primary Holme Catchment. All velocities were calculated with equation 1, given in the methodology. 49

Table 3-5. Velocity, travel time and distance of wave propagation for the two secondary field sites... 50

Table 3-6 Froude numbers for the experiment waves on the Holme and Ryburn catchment as calculated at the STW gauging sites.................... 52

Table 4-1 A summary of the density of the pollution substitutes and what they were intended to represent. 92

Table 4-2 Each experiment set consisted of taking one variable and running experiments over a number of iterations of it. 92

Table 4-3 Each wave treatment with its Reynolds number as calculated from wave height and velocity recorded in the results section. 97

Table 4-4 details the change in dye concentration and volume as measured in pixels in figure 4.10. 104

Table 4-5 percentage change in oil pixel count through time from the bathymetric diagrams in figure 4.13. 110

Table 4-6 details the mean (μχ), velocity (V) of the baseflow, the UDVP output during wave passage, the wave velocity as calculated from the footage, and Froude numbers derived from the numbers. 115

Table 4-7 Summary kaolinite progress down tank with different wave treatments... 116

Table 5-1 the two parameters that were varied and their respective iterations that were tested. 142

Table 5-2. Descriptive statistics for the particle distribution down the tank. Only the spread over the longitudinal down tank axis is detailed here. 148

Table 5-3. Summary statistics for particle distributions for the different wave magnitude scenarios 15 seconds after wave release. 153

Table 5-4. Percentage of particles that that have moved beyond 100m at 15 seconds after release for the differing wave durations. 156

Table 5-5. Comparison between 5m model and flume tank results. Data is drawn from the 12.5l base scenario shown in figure 20 of chapter 4 for the flume. The CFDM data is identical to that shown in figure 1. 157

Table 6-1 Flow frequencies for each scenario and the corresponding discharges at the three gauged points down the river. All flows are derived from the 2 year data set described in chapter 3. 173

Table 6-2 Mean velocities between the STW outflow and the confluence for each flow scenario. 174

Table 6-3 The components for equation 6.3 for NH_4 data from the River Holme experiments and two hypothetical scenarios. 179

Table 6-4 The components for equation 6.3 for conductivity data from the River Holme experiments and two hypothetical scenarios. 180

List of Figures

Figure 1-1. In response to pollution point P, water is released from the reservoir, through the stilling basin and into the river course, generating an artificial high flow event. In frame A the wave is released, it progresses down river in B, and catches the polluted water in C. 3

Figure 1-2. The Questions Aims Methods (QAM) flow chart, mapping the thesis 5

Figure 3-1 : A map of Yorkshire Region and UK locating the three study catchments. The catchments are colour coded; yellow for the Holme, red for the Don, and Green for the Ryburn. Rivers are shown in blue and urban areas in grey. 27

Figure 3-2. A map of the primary Holme Catchment. Arrows have been used to locate the reservoir used for release experiments, the STW site at which dilution of water quality was measured, and the Queens Bridge site at the bottom of the system used for stage and dye experiment measurements. Each of these sites is referred to extensively in the text. 29

Figure 3-3. Photo of a weir on the River Holme. Numerous weirs such as this act to oxygenate the water as it flows downstream and maintain high river stage. 30

Figure 3-4. The stilling basin at Digley Reservoir during a release. Water is discharged at high pressure out of the pipe (right hand side of image) with considerable force, again oxygenating the water as seen in the later results sections. 31

Figure 3-5. A view down the River Holme downstream of the STW outflow. A riffle pool system can be seen in this image with the rough area of water in the centre of the image being the riffle. This image has been included to give a visual indication of the scale of the river. 31

Figure 3-6. This line was constructed from interpolated Ordinance Survey contour data and consequently has some artificial bumps. The locations of the 3 data collection sites are indicated with arrows. 32

Figure 3-7. A map of the Don catchment above Blackburn Meadows STW. Underbank Reservoir and both of the STW sites studied are noted on the map. 33

Figure 3-8. A map of the Ryburn catchment with Ryburn Reservoir and Ripponden STW labelled. 34

Figure 3-9. Primary Holme site rating curve for flow downstream of the STW outflow. 37

Figure 3-10. Secondary Ryburn site rating curve for flow downstream of the STW outflow. 37

Figure 3-11. Gauge locations for the three field sites. 38

Figure 3-12. The three sites of importance are labelled on the map; the reservoir release at Digley Reservoir, the dye injection atthe STW, and the water sampling site at Queens Bridge in Huddersfield. 39

Figure 3-13. Locations of the sondes employed to measure water quality during release experiments at the three field sites. 40

Figure 3-14. The black rectangles represent conductivity probes at three heights in the water column 42

Figure 3-15. Discharge hydrographs for the release experiments carried out in the Holme primary catchment. The locations of the gauges can be seen in figure 11. 47

Figure 3-16. Discharge hydrographs from the two experiments on the River Don by gauge site. 48

Figure 3-17. Discharge hydrographs from the experiment on the River Ryburn.. 49

Figure 3-18. A box plot of flow velocities recorded at four morphologically defined sites within the river. A box plot of the wave front velocities as detailed in table 4 has been added for comparison. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers... 50

Figure 3-19. Rhodamine WT is shown with the dark purple line and flow with the blue. In both graphs the flow is Environment Agency supplied data. Panel (a) shows data collected on 09/05/13 during a release experiment, and panel (b) shows the control experiment on 11/10/13. As described in the method this data was collected at the Queens Bridge site 14.7km from the reservoir and 6.5km from the STW. 51

Figure 3-20. Conductivity chemographs at the reservoir and STW sampling sites. The blue represents discharge, black conductivity concentration through time. 54

Figure 3-21. Conductivity chemographs at the River Don sites. The blue represents discharge, black conductivity concentration through time.. 55

Figure 3-22. Conductivity data at River Ryburn sites. The blue represents discharge, black conductivity concentration through time.. 55

Figure 3-23. NH_4 time series graphs for the Holme primary site. Flow is described by the blue line.. 57

Figure 3-24. NH_4 response on the Don, blue line is discharge, black NH_4. 58

Figure 3-25. NH_4 response on the Ryburn, blue line is discharge, black NH_4.. 58

Figure 3-26. DO time series data from the primary River Holme site, blue line is discharge, black DO concentration. 60

Figure 3-27. DO time series data from the secondary River Don site, blue line is discharge, black DO concentration. 61

Figure 3-28. DO time series data from the secondary River Ryburn site........ 61

Figure 3-29. Time series data for pH and flow during wave passage at the primary River Holme site. The blue line depicts discharge and the black pH level.. 62

Figure 3-30. Time series data for pH and flow during wave passage at the secondary Don site. The blue line depicts discharge and the black pH level. 63

Figure 3-31. Time series data for pH and flow during wave passage at the secondary Ryburn site. The blue line depicts discharge and the black pH level. 63

Figure 3-32. Turbidity time series graphs for the primary River Holme site during wave passage. The blue line represents discharge, the black turbidity. 64

Figure 3-33. Turbidity time series graphs for the secondary Don site during wave passage. The blue line represents discharge, the black turbidity. 65

Figure 3-34. Turbidity time series graphs for the secondary Ryburn site during wave passage. The blue line represents discharge, the black turbidity. 65

Figure 3-35. Temperature time series graphs for the primary Holme Field site. The blue line represents discharge, the black temperature........... 66

Figure 3-36. Temperature time series graphs for the Don site. The blue line represents discharge, the black temperature. 67

Figure 3-37. Temperature time series graphs for the Ryburn site. The blue line represents discharge, the black temperature. 67

Figure 3-38. A week of data from the Sonde for 6 water quality parameters <200m below the STW. Discharge is represented by a blue line, water quality parameters the black. 71

Figure 3-39. 24 month rainfall (Ludhill farm EA tipping bucket gauge) in grey, discharge at Queens Bridge EA gauge (Huddersfield) in blue. The red lines locate the reservoir release experiments. Locations of these gauges can be seen in figure 10.. 72

Figure 3-40. A graphical display of conductivity data from three depths in the water column labelled top middle and bottom. The values presented here have been zeroed and shown as a decline rather than actual in river conductivity values. This has been done to communicate the relative declines in conductivity at different heights in the water column, rather than simply reiterate conductivity trends already described in this results section. The time frames displayed start less than <10 minutes before the arrival of the wave front and continue until an appreciable drop has been shown. 73

Figure 4-1 a photograph of the flume tank.. 89

Figure 4-2 A diagram of the flume tank showing the basic components. The tank is divided into sections A, B and C for descriptive purposes. The position of the inflow hose is indicated, as are the two Perspex lips and the gate. 90

Figure 4-3 The flume tank setup for the bulk tests. Dyed water is indicated in pink, held in section A behind the gate. Note that no flow is released from the hose above the tank. 93

Figure 4-4 The flume tank setup for the flowing bulk tests showing the dyed wave water in section A behind the gate, the input of flowing water from the hose, and the position of the camera 2m down the tank. 94

Figure 4-5 The flume tank setup for the drop tests with both a stationary camera and a running camera. The dye is injected with pipette shown above the camera and clear water is held in section A behind the gate. 95

Figure 4-6 a colour map of three different dye concentrations within tank as seen in the results section. 97

Figure 4-7 Results from the standard ambient bulk test. The dotted lines represent the water surface at various points in time defined in the key. The progressively lighter pink coloured areas represent the area of tank occupied by dyed water at a given point in time. Labelled in seconds since the appearance of the wave on frame. The right hand panel is a still image for comparison. 100

Figure 4-8 Results of the ambient depth experiments. Results from the three water depths tested are shown in descending panels with a bathymetric graphic and the original footage side by side. Dotted lines and gradations in colour have been used to show the position of the wave and dye mass through time. 101

Figure 4-9 Results of the bulk test in flowing water. Two panels are given, on the left a still image at 0.2 seconds after the wave arrives on shot and a bathymetric diagram through time on the right. The position of the wave front (dotted line) and the dye mass are shown at 0.2, 0.4, 0.6 and 1.0 seconds after wave arrival. 102

Figure 4-10 Camera tracking a wave and a dye slug down the tank Distance intervals of 1m are marked at the bottom of each frame and time steps are noted in the top left hand corner. Footage stills are shown on the right and bathymetric diagrams on the right. 103

Figure 4-11 Tests with 1ml of dye allowed to disperse over varied time periods prior to wave release. The top frame covers the 4 seconds of dispersion, the second frame 6 seconds, the third 10 and the fourth the no wave scenario. Each bathymetric diagram shows the dye motion from 0 seconds after the arrival of wave front, until 1 second at intervals of 0.2 intervals. 105

Figure 4-12 Results from the constant dye injection. The Bathymetric diagrams on the right show the progression of the dye as it is moved by the wave. The time after wave arrival on footage frame is noted in the top left corner of each panel. Time frames were chosen to illustrate the progression of the dye and the time it takes for the dye distribution to return to pre-wave arrival state. 107

Figure 4-13 Results of 30ml of oil interacting with a normal 12.5l wave. The oil, a mixture of olive oil and rhodamine dye, appears dark red in the footage and is represented on the Bathymetric diagram on the left. The time after wave arrival is indicated in the top left corner.. 108

Figure 4-14 Panels through time of the camera tracking the dye and wave interaction. The oil shows up as a dark red in the video footage, and is represented in that colour on bathymetric diagram. Some dye moves from the oil into solution and appears pink. Distances down the tank are shown at 1m intervals along the base of each panel, time after wave arrival on frame are shown in the top left corner. 109

Figure 4-15 Results of 30ml of kalinite interacting with a normal 12.5l wave. The kaolinite has a pale cream colour both in the footage shown on the right and the bathymetric diagram on the right. 110

Figure 4-16 Results of 30ml of jelly interacting with a normal 12.5l wave. In the footage frame on the right the jelly can be seen in dark red at the base of the tank. Progression through time of the jelly, in shades of red and pink, and the wave front, dotted lines, are show in the bathymetric diagram on the left. The time of the footage frame is shown in the top centre left corner. 111

Figure 4-17 The effect of three wave magnitude treatments upon 1ml of dye. The dye was allowed to disperse 4 seconds before wave release. All times on the graphic, top left in the footage frame, are given from the point of wave arrival. The bathymetric diagrams detail the dye and wave progression over three time steps. 112

Figure 4-18 The effect of the two peaked wave on 1ml of dye. The dye was allowed 4 second s of dispersion time prior to wave arrival. The time step for each panel is given in the top left corner................. 113

Figure 4-19Time series lines for each wave treatment of dye concentration through wave passage. 0 seconds in the time line represents the arrival of the wave front on camera frame. 114

Figure 4-20 Time series of dye concentration through time for the no wave scenario. 114

Figure 4-21 Bar graphs of velocity in ms^{-1} against water height at the time of the wave peak arrival. The 1st and 2nd peak are the consecutive peaks of the 2 peak wave. 117

Figure 4-22 A conceptual diagram dividing the normal wave scenario into its supercritical wave peak, sub critical base flow and turbulent wake. The wave is flowing from right to left. 125

Figure 5-1. 5m Model design. The particle input surface shown in grey is located 1m down the tank. The input surface is shown in green and output in red. The blue area indicates the volume filled with water. The dark blue arrow and the dark blue surface indicate the baseflow inlet. 139

Figure 5-2. Timeline of the model running time. Time is counted forward and backward from the start of the wave release as this is how the data is presented in the results. The blue second shows the time in which the wave input was raised and the grey the particle input. 140

Figure 5-3. Reach Model design. The dashed lines represent the limits of the 100m section of the tank represented in the graphical results. The statistical results cover the 10m following this too. The blue shows the approximate volume occupied by water. The green surface and arrow show the inlet, the grey the particle input (35m from the inlet) and the red, the outlet. 141

Figure 5-4. Discharge through time at the inlet of the Reach Model. 142

Figure 5-5. The timeline for the Reach Model. Time is centred at the point of wave release since this is how the results are presented. The blue seconds show the input of the wave and the grey the particles. 144

Figure 5-6. The progression of the wave (blue area) and the tracked particles (black points) down the tank at 0.5 second intervals over the first 3 seconds after the waves release. 146

Figure 5-7. The base scenario results from the Reach Model. The blue shows $10m^3s^{-1}$ of wave water which is released from left to right down the tank. The pollution particles are represented in black. 147

Figure 5-8. A side view of the velocity results from the base scenario in the Reach Model. White represents higher velocity magnitude and black lower. Both water and air are represented. 148

Figure 5-9. Particles are represented by the black points. Unlike figure 7 this figure is stretched on the y axis to show the position of individual particles in finer detail. 149

Figure 5-10. The plan view of the tank with the cross tank axis being stretched across the y axis 2.5m being the centre line and the x axis being the down tank distance. Particles are represented by black points and two points in time are depicted. 150

Figure 5-11. Particle distribution boxplots down the centre line of the tank (y axis) 10 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 4 scenarios with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers. 151

Figure 5-12. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 1 scenario with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers. 152

Figure 5-13. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 4 scenarios with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers. 154

Figure 5-14. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers. 155

Figure 5-15. shows two boxplots on the Y axis representing the ditribution of particles down the tank at 15 seconds after wave release. The left hand boxplot shows the 3 second duration wave with a 10m^{3}s^{-1} magnitude and the right hand shows the 2 second duration 15m^{3}s^{-1} magnitude. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers. 156

Figure 6-1 the trend in velocity and travel time of a pollution slug moving between the STW input and the confluence with the River Colne 6.5km down river. 172

Figure 6-2 velocity by model cross section for the three flow scenarios tested, with each cross section labelled by distance downstream of the STW. 173

Figure 6-3 The 1km grid Map shows the three scenarios from the 1D model reported earlier in this chapter. A 1ms^{-1} wave was released to catch the pollution slugs. The green line represents the distance travelled by the pollutant traveling at 0.189ms^{-1} before being caught, the orange the 0.253ms^{-1} and red 0.96ms^{-1}. 176

Chapter 1. Introduction

The object of this PhD thesis is to examine the feasibility of using water released from reservoirs to mitigate in river pollution. Poor water quality is a global problem exerting pressures on the environment and its ability to support both human life and ecosystems (Vörösmarty *et al.* 2010). Short term transient pollution incidents can lead to significant widespread impacts upon the aquatic environment such as that of the aluminium plant sludge release which occurred on the Marcal River in Hungary in 2010 (Klebercz *et al.* 2012; Nagy *et al.* 2013). Consequences of these incidents include acute effects such as fish kills (La and Cooke 2011). Whilst there is a wealth of research on managing pollution pathways, incident prevention studies focussing on in river mitigation of unpredictable short term incidents are less common. Diluting polluted water is an approach to dealing with such incidents that could be implemented. In the field of experimental studies, a few papers such as Barillier *et al.* (1993) and Chung *et al.* (2008) have examined managed dilution but not with replication of results or in reference to specified pollution point source. This thesis will examine this concept over the following 6 chapters.

To understand the effect of a release wave on water quality, field investigation, flume experiments and computer modelling have been employed. The current chapter has been written to give the reader sign posts for the thesis as a whole, to show and explain its structure and intent.

1.1 Thesis Structure

This thesis is divided into 6 chapters;

Chapter 1; An introduction that outlines the structure and logic of the thesis. Brief descriptions of the key questions and aims of the thesis are presented.

Chapter 2; General Literature Review places this study within the wider context of water quality studies and expounds the need for an examination of a dilution based mitigation system in light of the other approaches to manage river pollution.

Chapter 3; The Field Study Approach first reviews other publications that examine the use of waves of water to induce alterations in water quality. Second data is presented from a series of field experiments that demonstrates the effect of a released wave of water upon water quality. This includes both the dilution of the outflow from sewage treatment works (STW) and a dye based simulation of a pollution incident.

Chapter 4; The Flume Study Approach starts with an explanation of wave theories and a review of flume tank studies dealing with wave flows. The results from a series of flume experiments recorded in images and diagrams are then presented. Each experiment examined the impact of a wave on a simulated pollutant such as a dye.

Chapter 5; The Computer Fluid Dynamics Model (CFDM) includes a review of CFDM studies on waves and wave pollution interactions, and general approaches to hydraulic modelling. Results from a CFDM are then presented showing the impact of a simulated wave upon the distribution of a set of tracked particles.

Chapter 6: The General Discussion discusses the key questions laid down in this chapter in turn before detailing a 1D model, presenting the key lessons of the thesis and then outlining some operational limitations for water managers. Finally the work is evaluated and its contribution to the field of published literature is considered.

1.2Terminology

Terms and acronyms are defined in individual chapters but it is useful to clarify a few that are used repeatedly here.

Sewage Treatment Works (STW) is used to describe an sewerage processing plant. Terms such as Waste Water Treatment Plant (WWTP), or Sewage Treatment Plant (STP) are sometimes used in the literature. This study will use STW.

The term baseflow is often used in the literature to describe flow derived from subsurface flow as opposed to the surface runoff usually associated with storm flows (Nathan and McMahon 1990). In this thesis a term (baseflow) was required to describe the flow of water prior to the arrival of the wave flow. The key distinction being between water sourced from the impoundment that has been released and the water already within the water course from other sources.

1.3Key Questions

This thesis is concerned with whether it is feasible to mitigate a transient pollution incident by releasing a wave of water from a reservoir.

If a transient pollution incident, that is a pollution incident with a duration of hours and days as opposed to weeks months or seasons, is detected within a river, the in-river concentration of the pollutant substances involved could be diluted. Water stored in an impoundment could be released into the river upstream of the polluted waters. This wave of water would then move down the river increasing localised flow and inducing dilution. Figure 1-1 below gives a graphical representation of this;

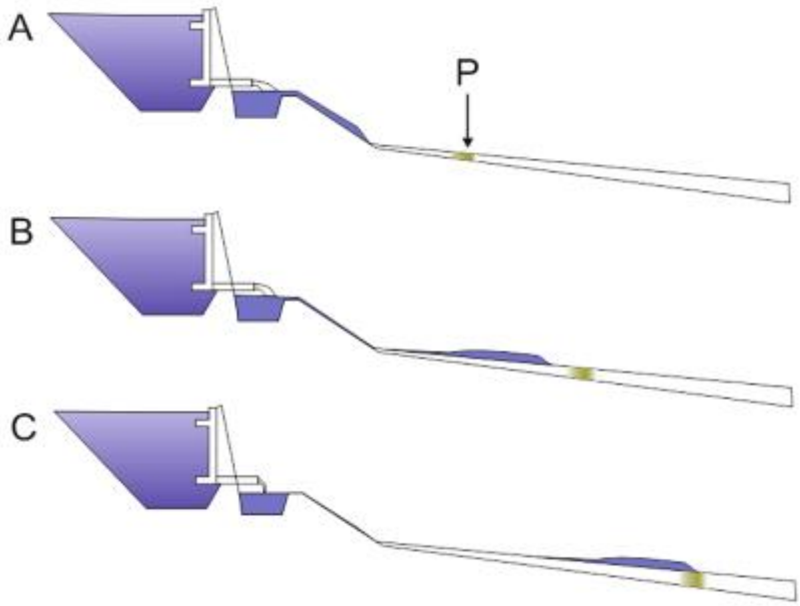


Figure 1-1. In response to pollution point P, water is released from the reservoir, through the stilling basin and into the river course, generating an artificial high flow event. In frame A the wave is released, it progresses down river in B, and catches the polluted water in C.

Three key questions need to be answered in assessing the practical feasibility of this approach to managing water pollution. They are;

Q1 How quickly can a wave of water released from a reservoir catch a volume of polluted water?

This is primarily a question of hydraulics. It is has been shown that waves in rivers travel faster than baseflow (Glover and Johnson 1974; Gilvear 1989). Given a long enough reach of river a wave would be expected to catch up with a pollution slug. The question is one of timing and relative velocities of both the wave and the polluted water. A dye test was carried out and reported in chapter 2. Furthermore wave travel and river baseflow flow velocities were measured. Experiments in both the flume tank (chapter 4) and CFDM (chapter 5) also examined the velocity of the wave against a simulated pollutant in the proceeding water. A 1D model is detailed in chapter 6 that considers catchment topography, flow resistance and discharge to produce baseflow estimates on a catchment scale. The results of this model and recorded wave speeds are then used to inform management decisions.

Q2 How much dilution can be achieved?

This is the next logical question once the wave has reached the pollution. Dilution has been defined as “The process of making weaker or less concentrated.” (American-Heritage 2003). A concentration is often given as a ratio between two substances. Increasing the quantity of the higher concentration substance will increase dilution of the lower if they are evenly distributed, or mixed, in the volume being considered. All three chapters seek to quantify changes in the concentration of pollution during the passage of a wave

Q3 What mixing processes occurs when a wave catches polluted water?

It is not certain that even mixing will occur across the volume of water holding the pollution. Waves exert a circular motion upon fluids in deep water (Masselink and Hughes 2003). Whilst in rivers, where the wave length greatly exceeds the depth, this is not replicated and turbulence and chaotic particle movements are produced on the vertical and horizontal plane (Whitham 2011). It is necessary to examine what affect this has on dilution.

The flume experiments of chapter 3 and the CFDM of chapter 4 were both designed with answering this question in mind. An experiment with conductivity probes at multiple heights in the water column is also reported in chapter 2 with the field experiments.

These Questions, and the Aims, Methodological Approaches and chapters that deal with them are graphically presented in the flow chart on the following page.

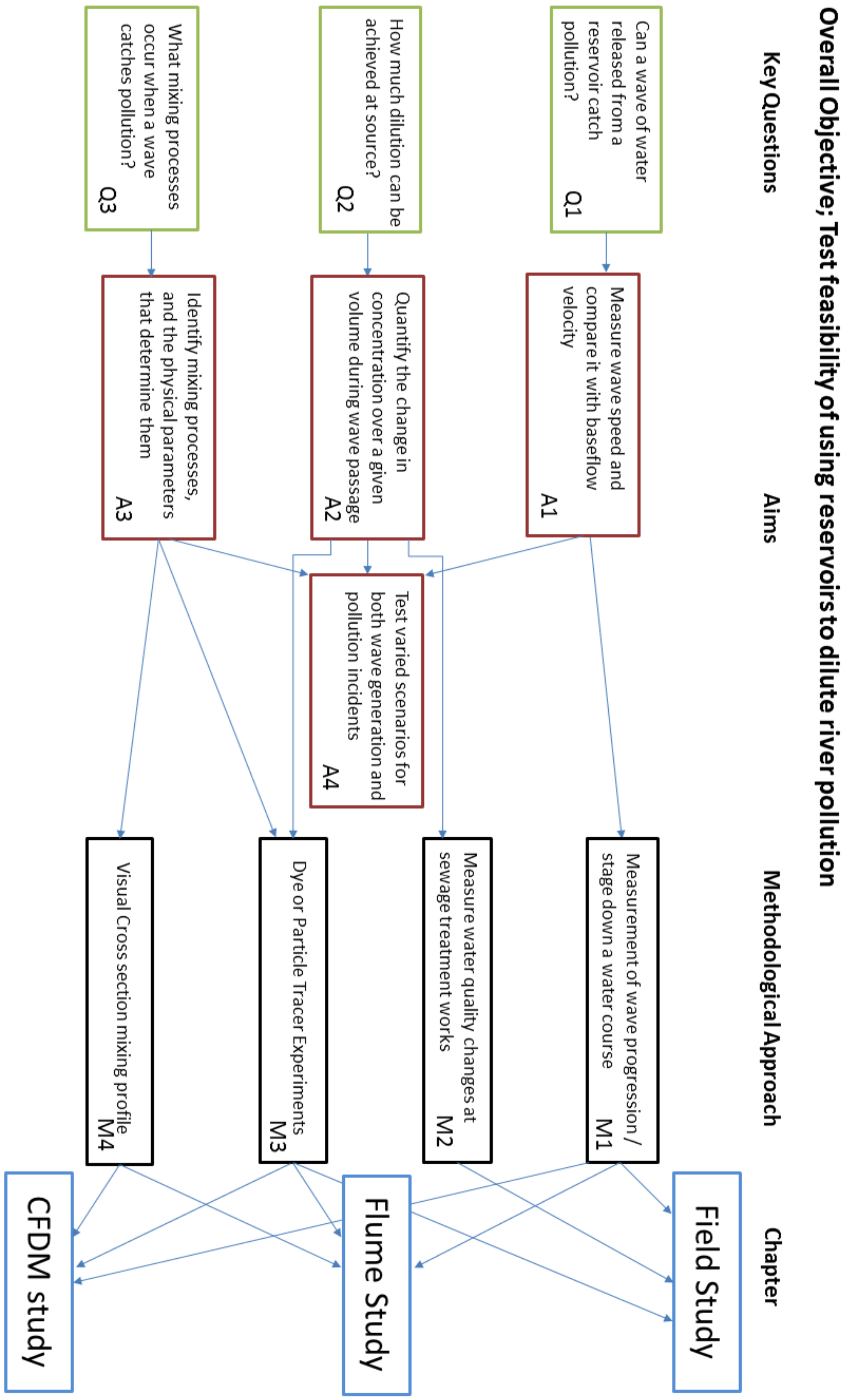


Figure 1-2. The Questions Aims Methods (QAM) flow chart, mapping the thesis

1.4Aims

A1; measuring wave velocities and comparing them with baseflow velocities that occur prior to wave passage will produce a quantifiable answer to question Q1. If the relative velocities and start locations of both the wave and pollutant are known then the time it will take the wave to catch the pollutant can be calculated. Whether the time period between wave release, or even incident detection, and eventual dilution is too long to be considered effective is a legislative assessment and ultimate beyond the authority of this thesis. A discussion on this subject is however given in chapter 6.

A2; quantifying the change in concentration of a given substance will provide an in-situ measure of dilution. In the field study this involved measuring concentrations of various water quality parameters below the out flow of a STW. In both the flume and CFDM chapters this involved measuring longitudinal dispersion. In each case this gives a quantified measure of the effect of the wave on solutes.

A3; Mixing processes are three dimensional and difficult to study with single point measurements. To answer Q3 it is necessary consider both lateral and longitudinal dispersion (Rutherford 1994). The two previous aims were to provide numeration of a process. A3 is open, it is a case of determining what bias exists both vertically and horizontally within the water column for movements of fluid during wave passage. All three chapters tackle this aim. The flume study and CFDM chapter provide visual data pertaining to both cross channel and vertical water column mixing during wave passage.

A4; The duration, magnitude, and profile of the wave released all have the potential to affect chemographs and the velocity of the wave. For managers it is important to use released water efficiently. Therefore each experiment considers variations in the form of the wave both to observe changes in results in general terms, but also to establish if one wave form is more effective than another. In the transient nature of the field environment water quality, weather, and flow conditions will all vary seasonally. Therefore to account for changes in conditions experiments were repeated in different seasons over a two year period in order to test varied scenarios.

Chapter 2. Literature Review

This literature review is intended to guide the reader from the basic premise of water quality being a worthy subject of study through to the rationale for the topic of this study; the dilution of polluted waters as a mitigation strategy. More in-depth studies of literature relevant to specific questions associated with the investigative approaches used to study this subject are dealt with in each of the three methodological approach chapters as outlined in the introduction chapter. Here, a general narrative is given that surveys a broad spectrum of literature with the aim of demonstrating the need for this study and the implications of taking a dilution based approach to pollution mitigation.

2.1 The Importance of Water Quality

Life on earth is utterly dependant on its access to high quality water. Terrestrial ecosystems and human society at all levels require a plentiful supply of potable and 'clean' water. The crucibles of the Neolithic agricultural revolution; the Nile delta, the Yangtze, the Indus, the Tigris and Euphrates and the basins of Mesoamerica, were all based on a plentiful water supply (Hillel 1991; Delli Priscoli 2000), the world's most diverse terrestrial ecosystem, tropical rain forests, again is in many respects a water driven system dependant on high water quality (Haines *et al.* 1983; Adam 1994). Historically, major advances in human health coincided with the understanding of the link between water quality and disease (Paneth *et al.* 1998).

This thesis is concerned with water quality, particularly the water quality of fresh water bodies in the landscape of human habitation. The importance of good quality water to human kind and the biosphere cannot be understated. Neither can the threat to global water quality posed by human development. Vorosmarty *et al.,* (2010) in their paper on global threats to water security describe the present situation for both human and biodiversity as pandemic. The model based analysis presented in the paper identifies pollution as one of the two major threats to global water security. The 2011 United Nations Human Development Report, titled Sustainability and Equity (Klugman *et al.* 2011) considered water pollution as a threat to global equity as much as sustainability. Prior to this, the 2006 UN development report on water scarcity, notes that it is not natural resource limitations but rather human management decisions including the generation of poor water quality that are responsible for poor access to potable water over much of the world (Watkins *et al.* 2006). Poor water quality impacts food production and health with the poor being particularly targeted by the effects and dis empowered to deal with them. For this reason both UN reports see water quality as a challenge for the Millennium Development Goals with target 7.c specifically addressing this (UN 2000). Whilst the effects of water pollution told in the life expectancy of the population may be more acute in the global south there are costs to be paid in developed nations too. Vörösmarty *et al.* (2010) identify many developed areas of the world as having a lower threat from pollution, despite high pollution stressors because of the investment and management practices implemented to mitigate the problem. A question arises to how sustainable and effective this current investment and management are at tackling pollution.

2.2 The Nature of Pollution

In any study on pollution a clear definition is required. Pollution has been variously defined, in 1982 the United Nations published a definition aimed at the marine environment that has subsequently formed the basis of many other definitions;

"The introduction by man, directly or indirectly, of substances or energy into the marine environment (including estuaries) resulting in such deleterious effects as harm to living resources, hazards to human health, hindrance to marine activities, including fishing, impairment of quality for use of sea water and reduction of amenities."

(UN 1982)

Here, pollution is defined as being introduced by mankind, and having a harmful effect on the environment. It is defined by effect rather than necessarily the intrinsic properties of the substance or energy. Critically some substances or energies may have an effect at certain concentrations, but not at other lower concentrations. Equally, detrimental effects may be prevalent in one environment but non-existent in another. This thesis will use the term pollution to describe substances in most circumstances, but also water temperature changes, that have the potential to have a negative impact on the terrestrial fresh water environment or water quality in such a manner as to affect either human utility or ecological systems.

2.2.1 The Water Framework Directive

In 2000 the EU introduced the Water Framework Directive, an article of European wide legislation dealing specifically with water quality degradation. With the original due date for targets set in the directive being 2015, this is presently the most significant item of water legislation in Europe and must be considered as the wider context of any study into water quality issues. The Directive uses an almost identical definition of pollution to the one given above in statement 33 of Article 2(EC 2000/60/EC). However throughout its text the chemical aspects of pollution are very often tied to the control of substances though either Environmental Quality Standards (EQS), or other more qualitative and comparative assessments leading to what is termed Good Status.

Good status, a term applied to both ecological and chemical facets of water bodies, can be rather nebulous (Borja and Elliott 2007) with a combination of statistical procedures and the authors judgement used to define it (Kelly *et al.* 2008) . Set concentrations for substances with an EQS are used but so are broad classifications based on the subjective point of view of those doing the assessment. This principle carries over in to UK water quality control legislation and administration. The Environment Agency (EA), the governmental regulatory body responsible for water bodies in the UK, uses a system known as the Common Incident Classification System, or CICS to determine the severity of a pollution incident and thus the response and often financial penalty (EA 2007). The CICS also uses qualitative categories to classify pollution in a similar manner to the WFD. There is a simple reason for this approach. Under the definition given above, any substance or energy can be pollution. The list of EQS substances held on the EA webpage (Wilkinson 2011) is currently 1016 entries long. It is more realistic to use a qualitative approach to describe pollution in some circumstances. Therefore, whilst the definition of pollution can be simple describing pollution in water bodies is a more complex matter. This study will, in later chapters, consider specific volumes or concentrations of substances and energies when describing water pollution, as quantitative science is easier to measure and report in a rigorous manner, however the qualitative aspects cannot be neglected.

This thesis is concerned with the mitigation of pollution in riverine water bodies and therefore the WFD and its view of water pollution is important. The WFD takes a catchment wide holistic view of water quality. If a mitigation system for pollution incidents is to work with the WFD, it needs to account for this catchment wide view. The provision of environmental flows, or in other terminology, sufficient water for the function of the river, is considered by the WFD (2000/60/EC), and is listed in the catchment management plans for various river systems (EA 2009a). Furthermore authors such as Acreman and Ferguson (2010) discuss in detail the potential to manipulate flows within rivers to achieve good status. Management practices such as this will be discussed in greater detail later in this thesis.

2.2.2 The Sources of Water Pollution

Now that pollution has been defined, and that the key legislative landscape has been outlined the sources and occurrence of water pollution must be addressed before the attempts to manage it can be examined.

Within the terminology of the academic literature and water resource management pollution sources are typically described as either point source or diffuse source. Point source pollution is that which can be traced to one or more specified points of input into the water system, typically of industrial origin (Stewart *et al.* 2002; Monge-Corella *et al.* 2008). An example of this might be a combined sewer overflow (Estèbe *et al.*

1998). Diffuse source, sometimes referred to as nonpoint source, has been defined simply as everything else (Duda 1993), a more technical description is given by some authors as; pollution that is driven by mass water movement through the surface, or subsurface of the land (Olness 1995; Subra and Waters 1996). The division between point source and diffuse can be vague, a good example of this, is the nature of unregulated sewage discharge in developing nation cities (Duda 1993) which can create both multiple point sources but also a wide area leaching of pollution through the soil and in run off.

Water pollution sources are often divided into two groups; agricultural and urban. Agricultural pollution is the term typically used to describe surface runoff from agricultural fields (Pretty *et al.* 2000), the leaching from slurry stores (Neal *et al.* 2008), and the general chemical overspill and produce from farming business (Skinner *et al.* 1997). Whilst often diffuse (Howden *et al.* 2009), it can be strongly characterised by point sources (Neal *et al.* 2008). This can include chemical groups such as pesticides (Carson 1962; Levitan *et al.* 1995), nutrients such as nitrates, and phosphorus (Ahiablame *et al.* 2011; Bosch *et al.* 2014), sediment (Steegen *et al.* 2000), veterinary medicines (Kay *et al.* 2005a; Kay *et al.* 2005b) and bacterium (Kay and Stoner 1988). In upland catchments undergoing more intensive land use acidification, raised dissolved organic carbon, the mobilization of metals (Mitchell 1991) and sulphate build up (Bottrell *et al.* 2010) can occur.

Urban sources include sewage outflows (Madore *et al.* 1987; Purdom *et al.* 1994), industrial effluent (Nedeau *et al.* 2003), domestic waste, or landfill leachate (Harwood 2014) and road runoff (Laxen and Harrison 1977). In developed nations this is more often than not point source, even for urban runoff outside of floods due to the extensive employment of drainage systems although diffuse urban sources are given coverage in the literature (Mitchell 2005; Ellis and Mitchell 2006; Kim *et al.* 2007). The content of these pollution inputs to the river can be highly varied between catchments. For instance the output of a paper mill is characterized by substances such as nonylphenol polyethoxy carboxylate and its metabolites (Field and Reed 1996), whereas heavy industry and road runoff is associated more with soluble, or particulate metal concentrations, in particular Cr, Pb, Cd, Ni, and Fe (Göbel *et al.* 2007; Etim and Onianwa 2013). The legislative approach to controlling these pollutant inputs also varies between governing bodies. For instance a consent limit, that is the highest concentration of a given substance that a released effluent is allowed to reach, of $2mgl^{-1}$ is reported by Bowes *et al.* (2010) for a series of UK rivers. In China, water quality is split across 5 categories with the lowest category, water considered to have no practical use, having a total nitrogen concentration of $>2mgl^{-1}$(Huang *et al.* 2010).

Pollutants are often quantified in two forms, concentrations and loads. A concentration is a ratio of the pollutant substance to a given unit of fluid volume of the river or water body. The load is a total volume or weight of the pollutant substance, usually given over a unit time (Taebi and Droste 2004b). Loads are often used in papers concerned with a holistic system analysis of pollutant flows (van der Weijden and Middelburg 1989) and modelling (Adamus and Bergman 1995). In water quality studies and legislation concentration is given greater attention. Annex 10 of the WFD is a list titled priority substances that deals in concentrations (EC 2000/60/EC) as is the focus of the Directive on Environmental Quality Standards (Environmental Quality Standards Directive 2008). Regulation of licensed or recommended pollution inputs is often given in concentrations. For instance Liang *et al.* (2014) put forward recommendations for nitrogen and phosphorus threshold targets to be set at 2.4mg Nl^{-1} and 0.2mg P L^{-1} respectively for urban river systems in the Yangtz Delta. These values are justified primarily on the limiting factors for algae growth. This contemporary example is a good illustration of how concentrations of pollutants guide water quality management. At low concentrations many pollutants have either limited chronic toxicity or negative impact on the aquatic environment (Påhlsson 1989). For some substances the concentrations involved can be at the nano-gram level however (Adams *et al.* 2006).Therefore lowering concentrations is a key aim of water quality management.

2.2.3 The Drivers of Water Quality Management in the UK

In the UK the WFD, Bathing Water Act, the occurrence of blue green algae blooms, eutrophication, and the desire to avoid fish kills are the key drivers water quality regulation (DEFRA 2012). These drivers are not distinct, but each is worth discussing individually. To achieve good chemical quality most catchments have set standard for substances that are considered to be overly abundant within the given catchments management plan. For example Appendix B for the Humber River Basin District Plan lists improvement of DO levels and other substance concentrations for a number of water bodies (EA 2009b). The European Bathing Water Directive (Council Directive concerning the quality of bathing water (Bathing Water Directive) 1975; EU 2006) places concentration limits on bacterium of various forms including Escherichia coli, or faecal coliforms, with high quality listed at 100 colonies per 100ml for both. Eutrophication has had a rather fluid definition (Nixon 2009) but is often a term used to describe an increase in biological material within a water body often caused by an increase in available nutrients (Correll 1998; Schindler *et al.* 2008). This process creates two main issues, firstly it has an adverse effect on ecosystem balance, and secondly, an increase in biological activity and respiration increases biological oxygen demand (BOD). Blue green algae is the colloquial term for various species of Cyanobateria. The toxicity and hazardous effects of such algae are well documented by Falconer (1999), in brief a bloom of the algae can pose a significant acute threat to humans, livestock and wildlife. A high concentration of available nutrients in warm waters can encourage the growth of blue-green algae colonies (Kratz and Myers 1955) and so, as with eutrophication this threat is principally a question of nutrient concentrations. Fish being at or close to the top of aquatic trophic web and a good indicator of water quality (Karr 1981), and being the primary interest of fishing groups, one of the more vocal stakeholders around water bodies, are generally treated as an alarm bell for water quality issues (Magnien 2001). Blue- green algae, eutrophication (Anderson *et al.* 2002) and a host of other deteriorations in water quality including acidification of water bodies (Hesthagen 1987), very high suspended sediment levels (Bilotta and Brazier 2008), and drops in DO (Ellis 1991) can lead to a fish kill. It is important to note that pollution is often considered in the effects that are noticed and reported. The EA CICS system (EA 2007) in its categorisation of pollution considers effects such as visual changes, in addition to toxicity. Whilst duration is a factor considered in determining the severity of an incident it is not quantified. When a pollution incident moves from being minor to significant in terms of duration is vague.

Each of these water quality pressures is considered under the umbrella of the WFD, but more than that they are what water quality managers in practice consider when making decisions (Reynolds 2010). It has been demonstrated in the field of ecology that there is a gulf between what is published in scientific journals and the approach of site managers (Rogers 1998). Similar discussions have been had in papers focused on the aquatic environment (Day *et al.* 2002; Hillman and Brierley 2005). The success of any scientific solution to a problem ultimately does not rest just on the quality of the science but also the ability of managers to implement it.

2.3 The Focus of this Study: Pollution

Any study concerned with water quality must zero in on a subset of pollution substances or forms. Metals (Shiller and Boyle 1985), low concentration actute toxicity substances (Brown 1968; Blackburn and Waldock 1995), veterinary substances, drugs and pesticides (Ternes 1998; Revitt *et al.* 2002; Calamari *et al.* 2003; Murata *et al.* 2011), bacterium (Ongerth and Stibbs 1987; J. and Yu 1995; Crowther *et al.* 2002), and other compounds (Malaj *et al.* 2014) are specialist fields of study, requiring considerable specialism and resources. This study is concerned with the broader problem of water quality, albeit over short term incidents. Therefore the pollutants often associated with fish kills and short term urban pollution incidents have been focused on; DO, nutrients, temperature, suspended solids, conductivity and pH (Taebi and Droste 2004b).

In the UK context pollution from sewerage sources has long been considered the most common and damaging of pollution sources (Ellis 1991), with short term and CSO derived incidents (Mulliss *et al.* 1996). Ellis (1991) and Ellis and Mitchell (2006) both provide tabulated summaries of pollutant levels and ranges of various types from a selection of data sets, although the latter publication deals primarily with diffuse sources these data summaries do illustrate the highly variable nature of urban pollution concentrations and the significance of sewerage as a source, a reality that is widely reported in the literature (Robson *et al.* 2006). In 2010, the Environment Agency recorded 67 serious pollution incidents resulting from sewerage to water bodies for the UK with only agricultural sources recording higher with 71 (EA 2011).

2.3.1. The Duration of Pollution Incidents

Studies on specific shorter term pollution incidents, those taking hours or days rather than weeks, or months, are less common in the literature as noted in a number of papers (Harremoës 1982; Seager and Maltby 1989; Marsalek *et al.* 1999). Whilst studies with timescales of months, seasons or a year are common place (Correll 1998; Neal *et al.* 2005; Varol *et al.* 2012; Warner *et al.* 2013). Ellis and Hvitved-Jacobsen (1996) divide the effects of pollution incidents, from urban sources such as CSOs, into two broad categories, the acute effects, which have a duration of hours through to days, and the cumulative effect which pose a longer term problem. Oxygen depletion, bacterial pollution and some specific nutrients and toxic pollutants are classed as having accumulative effect, others such as suspended solids, and the wider spectrum of nutrients are considered to be more cumulative effect problems. The first flush effect associated with CSO and other urban drainage is well reported in the literature and is a transient short duration pollution phenomenon (Taebi and Droste 2004a; Li *et al.* 2007; Miskewitz and Uchrin 2013). There is debate over the precise definition of the first flush (Saget *et al.* 1996; Bach *et al.* 2010), however in brief it is often described as an initial spike in pollution concentrations following a precipitation event as accumulated pollutant loads in drainage systems and other temporary storage is mobilized and enters the river. These papers deal principally with runoff induced pollution, there is however a dearth of papers that consider spills in rivers, over turned trucks on roadways or other forms of precipitation independent pollution incidents. Oil spills in freshwater bodies have been reported on (Kemerer *et al.* 1985; Yapa and Tao Shen 1994), as have spills from agricultural stores (Mallin *et al.* 1997), additionally Malle (1994) provide a chronology of spills from numerous sources on the river Rhine. It is not surprising that there are fewer detailed studies on transient pollution incidents, since foreknowledge of the incident is often required to plan and collect samples, making such studies harder to perform. Spills of most urban pollutants however could be considered comparable to what occurs during a first flush with a lower volume of water involved. The pollutants themselves whether from the roadway, industry or sewer will be the same. In UK within recent years a number of spills and failures of treatment works have occurred within the Yorkshire region. In 2010 a fire at a factory in Huddersfield (BBC 2010) resulted in pollutants entering the river. Additionally lorries using the M62 motorway have been recorded by the Environment Agency as spilling their fluid loads into drains that then accessed the rivers (Reynolds pers comms 2010).

This thesis will focus on short term transient pollution incidents measuring in the duration of days and hours as there is a lower volume of literature dealing with this subject in detail. Secondly the mitigation being proposed in this study is more suited to shorter term pollution incidents as will be discussed later in this chapter.

2.3.2 Approaches to Managing Pollution

Prevention is always better than cure, and source control where possible is preferable. In the context of urban sewage sources this can involve the abandonment of combined sewer systems, but this is often cost prohibitive (Balmforth 1990). Another approach is temporary storage, this can be utilised both by combined systems, and separated sewerage drainage systems that have the potential to fail or overflow. Storage tanks are limited by physical constraints, they can be costly to locate within the urban environment, are subject to blockages in the network, and can always overspill in extreme events (Fu *et al.* 2010). Some authors have suggested reduction of overflow events does not necessarily improve river water quality (Lau *et al.* 2002). For diffuse sources sustainable urban drainage systems (SUDS) can be employed to remove various pollutants from run off and slow the movement of water down, thus limiting in river concentrations during high precipitation events (Mitchell 2005) which in turn reduces loading on CSOs and down system point sources. Such systems however only deal with precipitation induced events, blockages in drainage, pump failures and overloading of STWs can only been managed by storage, reduction, or in river. Furthermore SUDS are not required under UK law. Reduction of waste and pollution generation moves this discussion into the realms of social engineering and sustainability (Cabezas *et al.* 1999). Whilst technological solutions have been proposed to reduce pollution loads (Tipton *et al.* 2000) these are beyond the scope of this study. Furthermore, whilst reduction is an important approach in water quality management it has a limited role in managing short term high magnitude events resulting from a failure in the treatment plants or accidental industrial spills. All of these methods of urban pollution control are under pressure from climate change induced increases in extreme precipitation events (Astaraie-Imani *et al.* 2012).

If the system fails, be it the bunding at a chemical plant during an accident (Struthers and Illidge ; Cassie and Seale 2003; Atherton *et al.* 2008), the capacity of the sewerage system, or a mechanical failure within that system (Rouleau *et al.* 1997), or spill on a roadway or other environment (Mattson *et al.* 1977; Leveille *et al.* 1995; Sanders *et al.* 2002), pollution can circumvent the management system and enter the water body. Once within the river the pollutant must either be removed or the damage they can impose must be mitigated.

2.3.3 Removal of Pollution

Literature dealing with the removal of a pollutant from waste water within a controlled or otherwise contained environment are reasonably common, and deals with nutrients (Scholes *et al.* 1999; Jing *et al.* 2001; Wu *et al.* 2007), boron (Okay *et al.* 1985), or metals (Scholes *et al.* 1999), principally utilising constructed wetlands, an engineering solution that can be employed within the river but is more often used prior to river entry (Baker 1992). Attempts to remove in river pollutants with short term action are scant in the literature. In theory it would be possible to skim petrochemicals from the surface of the water (Damberger 1973), or dredge contaminated sediments, or even drain off contaminated water if the resources were available. The lack of reporting of these approaches would suggest that they are either not economically feasible, or they themselves can worsen the environmental damage. Dredging in particular can have negative consequences both in the disposal of the sediments (Rhoads *et al.* 1978) and in the action of dredging mobilising polluted sediments (Eggleton and Thomas 2004; Guevara-Riba *et al.* 2004). Flocculation followed by skimming of surface pollutants is a process employed in STW (Grutsch and Mallatt 1971; Kriipsalu *et al.* 2008) but due to the additives required would not be suitable in a river in most circumstances. The immobilization of metals has been achieved with salt additives in soils (Huamain *et al.* 1999; Seaman *et al.* 2001). Experiments mixing salt water with metal contaminated water to induce coagulation have been carried out (Eckert and Sholkovitz 1976) and similar results are reported from the estuarine environment (Alipour *et al.* 2012). Examples of adding additives to rivers to counter metal pollution issues are scarce however, Yantasee *et al.* (2007) do report the use of iron oxide nanoparticles as a sorbent for toxic solute metals in a riverine environment.

In the oceanic environment dispersants have been used to mitigate oil spills. In such a vast area surface skimming is often limited. Dispersants do not remove oil from the water body, rather they distribute it through the water column there by achieving increased localised dilution, lowering the concentration and reducing damage to the marine environment (Lessard and DeMarco 2000; Kujawinski *et al.* 2011). Dispersants have been used, and even modified for use in the fresh water environment, but with more limited results (George-Ares *et al.* 2001). Even if dispersants are a good counter measure for oil spills, such a method will only work on substances that do not mix with water and therefore is not applicable to any solute pollution.

Aquatic microorganism and biofilms of various species have been shown to adsorb metals such as Cd and As in laboratory experiments covering several weeks (Duong *et al.* 2010; Pokrovsky *et al.* 2010; Guezennec *et al.* 2012). In more controlled environments such as STW tanks biofilms have been used to treat a wider range of pollutants including organic compounds, nutrients and various metals (Edwards and Kjellerup 2013). Efforts have been made to apply this bio technology approach to polluted river reaches (Arini *et al.* 2012; Sheng *et al.* 2013; Wei and Cao 2013) by seeding biofilms into the river, or by constructing 'eco-tanks' along the rivers course (Xiao *et al.* 2012). These methods have been shown to reduce concentrations of specific contaminants, but they are constrained both by requiring a relatively long application time, weeks in the papers cited, and the need to either divert water to a contained tank, or release alien microorganisms into the riverine environment. Sheng *et al.* (2013) report the use of other technological additives including photosynthetic bacteria, floating microphyte beds, coal cinder and converter slag filled bags, as sorbents, chiefly for NH_4, and demonstrate a long term drop in concentrations from around 20-25mgl^{-1} NH_4-N to closer to 5mgl^{-1} over the course of 4 months. The use of chemical sorbents to reduce nutrient concentrations is widely reported in the literature (Li *et al.* 2013; Rout *et al.* 2014), and summarised by Loganathan *et al.* (2013). Whilst this method can be effective, the time scales demonstrated can be too long for short term pollution incidents discussed above. Furthermore additives can be costly and are not without environmental threats of their own tending toward their use in STW rather than rivers.

Sheng *et al.* (2013) also construct flow control structures such as weirs in order to oxygenate the water. The end product of nutrient pollution is to reduce DO in the water harming respiring organisms. Enforced aeration of a water body can be achieved through constructed water falls, or weirs (Cha *et al.* 2014), the installation of subsurface water lifting aerator systems (Huang *et al.* 2013) or by pumping oxygenated waters into regions of anoxic water (Stigebrandt *et al.* 2014). All these solutions have produced results in either an increase in DO levels, or the oxidising of pollutants. This is a hard engineering solution, and requires capital investment to both construct and maintain, and additionally causes a disruption to the flow of the river.

The pollution control methods discussed above are often suitable for managing rivers with a pollution history and a long term management timescale. However, few of these methods are suitable for reacting to a short term unpredicted transient pollution incident. The holistic aim of this thesis is to propose another method and investigate its feasibility. The method proposed is the in situ dilution of pollution through the controlled raising of the flow of water in a river by releasing water from a reservoir. The following paragraphs of this literature review will focus in on this concept and look at its strengths and weaknesses.

2.3.4 Dilution as a Pollution Management Tool

Using dilution as a pollution management strategy has a long and rather tarnished history. In the first half of the last century many pollutants were released into either large water bodies or the air on the basis that the dilution achieved would be so great that a laissez-faire attitude could be maintained (Andersen 1994). This approach has had a lasting environmental legacy with rivers, seas and the atmosphere often still bearing evidence of both the industrial past as well as the present (Sanders *et al.* 1995; Farmer *et al.* 1996). It is not unusual for modern papers to state out right that dilution is not the solution to pollution, and that source control and separation are the correct approach (Wilsenach *et al.* 2003). There is truth in this, source control and separation of pollutants are effective strategies in reducing damage to water quality, and a dilution based strategy will still compromise, if even in a more limited way, the quality of the water body involved. However, as has been described, when dealing with short term transient, unpredictable pollution sources source control strategies such as suds, storage tanks and engineered strategies can fall short. Dilution may not be a popularist approach, and certainly society should not return to the laissez-faire approach of former eras, but with pollution incidents in rivers being a reality, must be considered amongst the arsenal of responses.

Dilution effects have long been a determinant factor in air pollution control and study, down to the design of chimneys (Slawson and Csanady 1967), and exhaust structures (Al-Atresh *et al.* 2012) through to the study of air pollution of an urban area as whole (Mayer 1999). In the management of water pollution control too, dilution is still a core paradigm. When a regulatory body in a given government licenses an emission of a pollutant into a water body, the dilution factor that the pollution will achieve is an essential consideration in the agreement with the polluter (Whitehouse *et al.* 1996). In the UK this is often referred to as a consent (Manyumba *et al.* 2009), in the US an Environmental Quality Standard (D'Arcy and Frost 2001). Dilution is important because it determines in rivers concentrations which in turn affect toxicity levels as has already been discussed in this review.

On discussing pollution spills in the Rhine Malle (1994) notes that some spills can take considerable time to achieve significant dilution as the river is slow flowing and has a high width to depth ratio limiting mixing of the plume. A higher flow would mitigate this. The purpose of this thesis is to examine the feasibility of raising the flow in a river in response to pollution incident in order to induce increased dilution. Manipulating river flow to influence water quality is of itself not an original idea to this thesis with papers such as Malatre and Gosse (1995) have examined this concept on rivers such as the Seine. However as detailed later in chapter 3, many of the papers that do examine this concept are limited in both the depth and breadth of their study. Whilst not always backed by a scientific basis, raising the flow in a river by releasing water from a reservoir has been used to mitigate pollution incidents in the UK on certain occasions (Reynolds pers comms 2010). Papers such as Malatre and Gosse (1995) and others are reviewed in chapter 3, which details the field experiment results and sets them against this literature background.

Dilution equations are based on conservation of mass equations. In essence, conservation of mass assumes that the mass of a substance is conservative and must be maintained between two given points on a flow path. Okunish *et al.* (1992) use the following equation to describe a constant injection for salt dilution gauging of a stream;

$$C_f(Q + Q_t) = C_t Q_t + C_b Q$$

Equation 2.1

Where C_f is the tracer after an infinite time period post injection measured in mgl^{-1}, C_t is the concentration of the tracer in mgl^{-1}, or pollutant for this study, Q_t is the discharge of the pollution injection in m^3s^{-1}, C_b is the background concentration of that pollutant already within the river (mgl^{-1}) and Q (m^3s^{-1}) is the river discharge at the injection point.

This equation, as stated, assumes that mass is not lost, that the pollutant is conservative, and that mixing through the water column and cross channel is complete. Of these assumptions the mixing of the pollutant in the water column will be considered in this thesis.

For the purposes of this study the object is to increase the value of Q in order to reduce the relative importance of C_t in balancing the other side of the equation.

2.4The Conceptual Solution

On detection of an in-river transient pollution incident a response would be to release a wave of water from a reservoir upstream. This wave would then progress down the stream, catch up with the polluted water, and raise the volume of water available for dilution locally. Furthermore the higher turbulence of the flow would induce a higher rate of mixing (Rutherford 1994).

This approach has a number of benefits. Firstly, should the reservoir infrastructure exist, it requires low technological investment, secondly it has a response and effect time of hours rather than weeks, thirdly the secondary effects of increasing flow in a river are well understood within the literature (Turowski *et al.* 2013), and fourthly, when widely available, water is inexpensive.

The scientific questions that arise from this proposal were outlined in the introduction chapter and are investigated in chapters 3, 4 and 5. The three key questions are;

- Can a wave of water released from a reservoir catch up with polluted water?
- How much dilution can be achieved for a given volume of water released?
- What mixing processes occur and how are they influenced by the wave form?

These questions underpin the QAM diagram presented in chapter 1 the Introduction. Each of these questions and the hypothesis, aims and methods that follow from them were described in the previous chapter. This will not be repeated here as it is dealt with extensively in other chapters of this study. Rather the challenges and potential limitations of such a system should it be found to be effective and then implemented will be discussed instead.

2.4.1 Theoretical Limitations

To increase the flow in the river water in storage is required. An impoundment such as a reservoir is a common way to achieve this. The earliest records of reservoirs are detailed in Herodotus's the Histories (Herodotus 1987), who describes impoundment structures on the River Nile writing circa 400 BC of constructions built around 2900 BC. In industrialised nations of the global north many reservoirs were constructed principally for flow control. In the UK and many other countries the mills in and industry of the 19th century required a steady state flow within the rivers to achieve optimal production, consequently reservoirs were designed to store water when there was a surplus and release it during drought periods. Such reservoirs and more modern constructions can regulate flow releasing water as necessary. This allows for real time alterations to the flow in the downstream river. Whilst this water might be available three problems arise both water shortages and extreme water surpluses, and the variable quality of that water. There are many major river systems that do not have reservoirs or impoundments capable of releases in sufficient number. In the Yorkshire region of the UK areas such as the Upper Wharfe have viewer large reservoirs compared with rivers such as the Don. A system based on water releases will be specific to catchments with reservoirs.

Vörösmarty *et al.* (2010) estimate that 80% of the world's population is under duress from water scarcity in one form or another. Whilst the authors do account water pollution as a stressor that underpins this statistic, and the solution proposed here is to mitigate water pollution, it cannot be ignored that expendable water is not always available. In areas of the globe where there is a volumetric shortage of water supply a system based on dilution will not be practical. Even in areas with seasonal abundance such as the UK there is a management issue. The most famous drought of recent times in the UK occurred in the summer of 1995 in the Yorkshire region (Mead 2010). As a consequence of water shortage events such as these many water managers see a drought as the scenario that must be avoided at all cost. Therefore spending water for the dilution of pollution is a risky proposition if the same water is needed later in the year when stocks fall low. The value of water is in flux with supply and demand

and it is therefore difficult to make a decision on whether or not to release water objectively. Whilst demand forecasts are often well accounted for by models (Renzetti 2002), supply can be very varied (Ehrendorfer and Murphy 1988; Stewart *et al.* 1992).

Academic literature concerned with the value of water is rather sparse. This is highlighted by two questions reported in Brown *et al.* (2010)'s list of priority research questions as determined by the UK water industry and policy makers. Question 30 asks what the total economic value of clean water supply in the UK is, and question 31 asks what the value of freshwater ecosystem services are? Whilst these questions are not the same as the question above they highlight the sketchy understanding of the value of water and its condition.

One method currently used to assign a numerical value to the importance of water sources is the reservoir management system. Numerous systems are in use, and they are summarised by (Yeh 1985). Whilst the statistical methods employed do vary, the majority of systems either through forecasting, or probability distributions based on long term data sets estimate future supply. The current head level of each reservoir is known. Together these two values can be used with an estimate of demand to assign a value to water. Once water has a value, the risk of spending it for pollution mitigation over water supply or compensation flows can be calculated.

If water can be released is the water quality of the source reservoir high enough not to induce a pollution incident of itself? It cannot be assumed that water stored within a reservoir is not polluted itself as there are many studies detailing the pollution of reservoirs (Loska *et al.* 1997; Cai and Hu 2006; Davis and Koop 2006; Kay *et al.* 2012). This pollution can result from agricultural runoff (Kay *et al.* 2012), peat land and upland management practices, and soil erosion (Rothwell *et al.* 2005; Shotbolt *et al.* 2006; Holden *et al.* 2012). These inputs can provide the aquatic environment with excess nutrients leading to algae blooms in warmer seasons (Falconer 1989), eutrophication, and metal heavy sediments. Reservoirs like other lotic systems often have seasonal mixing turnovers which influence water quality (Petts 1984).

Reservoirs can have a negative impact on water quality in a second manner. A rapid increase in flow and the turbulence associated with it has the ability to remobilise sediments from within the river (Barillier *et al.* 1993). The quality of sediments in urbanised areas, particularly those with an industrial history can be poor (Dawson and Macklin 1998). Old *et al.* (2003) describe the remobilisation of such sediments by precipitation induced high flow events. A reservoir induced increase in flow would have the same ability.

Both poor quality input water and sediments within the river have to be accounted for in study or implementation of this approach two pollution mitigation. Aim 4 from the QAM deals specifically with this challenge.

The reverse of having insufficient water to expend it diluting pollution is having too much water either in the impoundment or in the river. Some sources of pollution such as CSO are most commonly active during high flow conditions (Soonthornnonda and Christensen 2008), as storm flows are required to exceed the capacity of the drainage system. Introducing additional water into a water body has the potential to cause flooding. A system based on generating high flows will therefore be dependent on the river not being in flood or near bank full. Reservoirs in the UK, and in many other parts of the world, will over flow into the river if they are full. This is often referred to as over topping. Reservoir managers will often not carry out a release if a reservoir is overtopping as it might overload the stilling basin, or the river the reservoir flows into (Townend, D. 2010 pers comms).

These limitations make such a system conditional rather than universal. Water managers would need to view the river system at the catchment wide level to judiciously implement such a system.

2.4.2 Long Term Application

Whilst this method of mitigating pollution has primarily been viewed as a tool for short term pollution incidents, a question does arise as to whether this approach could be applied to longer term problems spanning weeks, or even months. Acreman and Ferguson (2010) discuss releasing of what are termed 'environmental flows', or longer term releases from impoundments to improve riverine ecosystems to meet WFD targets. Increasing the flow of a river over the long term to compensate for a diffuse, or persistent pollution problems is a possibility though questions over water supply would be exacerbated. Furthermore if pollution treatment is considered as a complete system, with chemical and energy inputs, and the air and carbon pollution outputs generated by those inputs a trade-off can be made. The air pollution outputs can be reduced with the intensity of the treatment processes. This would result in a lower quality effluent entering the river increasing the in pollution, which could then be mitigated by dilution (Kitson, 2010 pers comms). With this rational a water pollution output can be traded off with an air pollution output. In section 2.4 of this review it was noted that water pollution is more often concerned with concentrations rather than loads of pollutants. A dilution based pollution mitigation system will not reduce loads. If a dilution based system were to be employed as a longer term approach to pollution management then the question of concentrations vs loads is magnified. Concentrations can be maintained at a low level but loads within the river will increase.

Higher loads, even at low concentrations can have an adverse effect on the aquatic environment. Bioaccumulation, bi concentration (Gobas and Morrison 2000), and biomagnification (Gray 2002) are all processes where by pollutants build up within areas, or organisms over time to build up to a higher concentration of the pollutant. Bioconcentration is often reported for metals (Raskin *et al.* 1994), organic pollutants

(Barron 1990), which are of acute toxicity, there are also papers that report bioconcentration of nutrients however with Ruiz and Velasco (2010) reporting accumulations of N and P in a species of reed in an aquatic system with mean concentrations of 4.3mgl^{-1} N-NH_4, concentrations that fall above those listed for Ammonium under both the protection of fisheries (Council Directive on the quality of fresh waters needing protection or improvement in order to support fish life (Freshwater Fish Directive) 1978), and water abstraction (Council Directive 75/440/EEC Surface Water Directive 1975) standards. There is a risk in permitting higher loads of pollutants into a river system that must be considered with long term scientific investigation should this route be taken.

2.5 Conclusion

It has been shown that whilst there are numerous approaches to managing the global issue of water pollution few deal with transient pollution events and their in situ mitigation. Whilst a dilution based system certainly cannot be considered a silver bullet solution, given the lack of alternatives for in-river management of transient pollution, it deserves serious investigation. The following three investigative chapters will narrow in on specific literature relating to this approach and present empirical evidence evaluating its feasibility.

Chapter 3. Reservoir Release Experiments: The Field Study Approach

3.1 Introduction

This chapter will seek to answer the questions laid out in the introduction and literature review chapters, 1 and 2, through the use of empirical field evidence. In a river system, can a wave of water released from a reservoir catch up with and then dilute polluted water? This chapter will examine this question, and the management and scientific questions that follow from it, using field evidence. The structure of this chapter is as follows; first a recap of these questions, the rational for the use of this investigative approach, some more detailed hypotheses are laid down and then the field sites are shown in maps and described. Methodology for all experiments are described and justified in the next section. Field data, its presentation, description, and analysis is split into three subsections; first experimental data concerning riverine fluid dynamics and wave progression velocities is dealt with, second water quality data relating to both the dilution of pollution and the wider water quality impacts of release waves is covered, and third, this data is set into context against two years of summarized background data. Finally all these results are discussed in reference to the aims and hypotheses laid down in this chapter and the wider literature, and then conclusions are drawn.

3.2 Literature

River water pollution is a real world problem and the ultimate test of any solution to a such a challenge is to test that solution under field conditions. The evidence presented in this chapter seeks to establish whether a wave of water released from a reservoir can firstly catch up with a slug of pollution moving down a river (key question Q1, and Aim A1 as described in chapter 1), and secondly whether the local increase in flow generated can dilute that pollution (key question Q2).

A wave in fluid, by the observable fact that it is transferring material forward, has a higher velocity in the direction of flow than the medium it is traveling through. Both mathematical (Lighthill and Whitham 1955; Beven 1979; Singh 1995), and field measurements (Gilvear 1989) attest to this. Gilvear (1989) outlines the limitations of using equations to estimate wave celerity velocities and progression. It is therefore necessary to investigate wave progression in the field to determine whether a wave released from a reservoir could feasibly catch a pollution slug in transit.

If a wave of water can catch a pollution slug, the question then is; what is the interaction between a wave and the water preceding it? How is water quality affected during wave passage? These questions have been approached via a number of routes in the literature. Papers such as Glover and Johnson (1974), Foulger and Petts (1984), Petts *et al.* (1985), Buttle (1994), Krein and De Sutter (2001), Kurtenbach *et al.* (2006), and Henson *et al.* (2007), all examine changes in water quality in a river reach not described as being polluted. Of these papers, only Glover and Johnson (1974) and Buttle (1994) are concerned with natural flood waves. Other authors including; Heidel (1966b), Gilvear and Petts (1985), Leeks and Newson (1989), and Bull (1997) focus on suspended solid concentrations (SSC) and turbidity concentrations during wave passage. Some common findings have been made which span across all these papers, first a clear response of water quality to wave passage, either in water quality fluctuation, such as a change in calcium (Petts *et al.,*1985), or as previously mentioned a rise in SSC. Second the inadequacy of mixing models to fully explain the changes in water quality that occur during wave passage, and thirdly the presence of a lag effect between wave front arrival and water quality response.

There are a limited number of publications that deal explicitly with a river that is considered either polluted, or of poor water quality, and the effect of a reservoir released wave upon that river; (Barillier *et al.* 1993; Malatre and Gosse 1995; Cánovas *et al.* 2008; Chung *et al.* 2008). All of these papers identify a dilution of some of the water quality parameters they measure and a change in water quality during wave passage although improvement in water quality is not unanimous. For instance Bariller *et al.* (1993) record a drop in NH_4 concentrations but also DO during wave passage, Chung *et al.,* (2008) reported a slight increase in organic phosphorus but also dilution in other nutrients such as NH_3.

Crucially all the papers that deal with water quality during the passage of a reservoir release wave have a number of short comings in common. Most examine only one release wave and whilst Krein and De Sutter (2001) deal with two, they are very different in form and alternative water quality parameters are reported for each experiment. Kurtenback *et al.* (2006) examine 5 identical releases, but each has a peak discharge of $1m^3s^{-1}$, and only results relevant to lag time responses are presented. Replication of results, and investigation of varying parameters such as wave form, changes in antecedent conditions, changes in wave magnitude, and changes in field site are not considered. If reservoir releases are to be used to dilute polluted water, data collected across a variety of conditions will be required to ascertain the practical plausibility of such a system. This chapter has therefore dealt with multiple releases at a single field site, others at secondary field sites, variations in wave magnitude, and seasonal and diurnal timing.

A second limitation across those papers that do deal with a river considered polluted is that they either address a diffuse pollution source (Canovas 2012), or a non-specific point source, both Malatre and Gosse (1995), and Bariller *et al.*(1993) deal with the

River Seine as it passes through urban areas generally. The study in this chapter will identify a specific pollution source in each catchment and attempt to deal with its dilution in isolation.

Other than the literature discussed here, there is a second driver for the research contained within this chapter. The results reported in chapters 4 and 5 from the flume and CFDM experiments provide evidence of a vertical stratification in the water column during wave passage. This result has been tested in the field environment in this chapter.

3.2.1 Definitions

A number of terms are used in this chapter that are either used in a specific manner or might be unfamiliar to the reader.

The term baseflow is commonly used by hydrologists (Kendall and McDonnell 1998) to describe the non-storm-related component of a hydrograph, produced by subsurface flow. In this chapter, and thesis in general, baseflow is used as a catchall term to describe flow that is not part of the release wave hydrograph. That is flow before or after wave passage, and flow on non-experiment days.

Release wave, or release experiment are the terms used to describe releasing water from a reservoir to produce an increased flow; terminology also used by some other authors (Gilvear 1989; Gido *et al.* 2000). Some papers have used terms such as 'environmental flow' (Bradford *et al.* 2011), artificial flood (Jakob *et al.* 2003; Muirhead *et al.* 2004; Henson *et al.* 2007), or peaking flows (Berland *et al.* 2004; Garcia *et al.* 2011) to describe similar experiments. Environmental flow has not been adopted for two reasons; firstly, the term is very vague and non-intuitive, environmental flow could mean anything. Secondly environmental flows are often used to describe longer term releases of water (Rolls *et al.* 2011). Artificial flood is a clear term, but the word flood can have negative connotations for members of the public, journalists, and people in industry of a non-scientific background. Since this thesis is meant to inform real world management a less loaded term was used. Peaking flows and hydropeaking are used to describe flows specifically associated with hydropower dams, and can often be a result of the dams operation rather than river management decisions (Maddock *et al.* 2013).

3.3 Aims

In chapter 1 the QAM was defined (see figure 1.2 chapter 1). This chapter will tackle all three questions, Q1, 2 and 3 introduced in chapter 1.

Aim A3.1 was to assess Key Question Q1

Key Question Q1: How quickly can a wave of water released from a reservoir catch a pollution slug?

This question will be answered through both water velocity and wave celerity measurements, and a rhodamine dye experiment.

Key Question Q2: How much dilution of a pollutant can be achieved?

This question was dealt with by measuring water quality parameters during wave transit downstream of a STW outflow.

Aim A3.2 of this chapter was therefore to quantify the change in concentration of a set of water quality parameters during wave passage.

Key Question Q3: What mixing processes occur when a wave catches polluted water?

This question has been dealt with in greater detail in chapters 4 and 5. It is however important to tie results from flume experiments and CFDM simulations to the field study.

Aim A3.3 of this chapter was to ascertain whether the vertical stratification occurs within the water column during wave passage.

Aim A4 in the QAM was to test varied scenarios for both wave generation and pollution incidents.

This overall thesis aim has a number of specific bearings on the experiments reported in this chapter. These are expressed in the following questions;

- What effect does wave magnitude have on dilution and wave progression?
- What effect does seasonal variation have on dilution and wave progression?
- What affect does diurnal change have on dilution?
- Can results be replicated across multiple experiments?
- Can results be replicated across multiple field sites?

Do release waves have adverse water quality impacts upon the river system?

Aim A3.4 of this chapter was to answer these questions and in doing so test varied scenarios.

3.4 Methodology

What follows is a detailed description of the field sites and the methods employed for each experiment conducted within them.

3.4.1 Field Study Sites

Three reservoirs and their downstream catchments were selected for field study in order to investigate the replicability of results between sites. Each of the sites has been generally referred to by its river name, but STW and reservoir names are also made reference too in the results. The first, and primary catchment, the River Holme, was selected to be instrumented and studied for a two year period with the intention of studying six reservoir releases. During each release the quantity of dilution achieved at an STW outflow, the wave travel times and quality of the reservoir outflow water were all measured. Secondly a larger scale catchment (The River Don) was chosen to examine a larger reservoir, greater length of river and bigger STW over two release events, and thirdly a smaller scale catchment (The River Ryburn) was studied for just one release event.

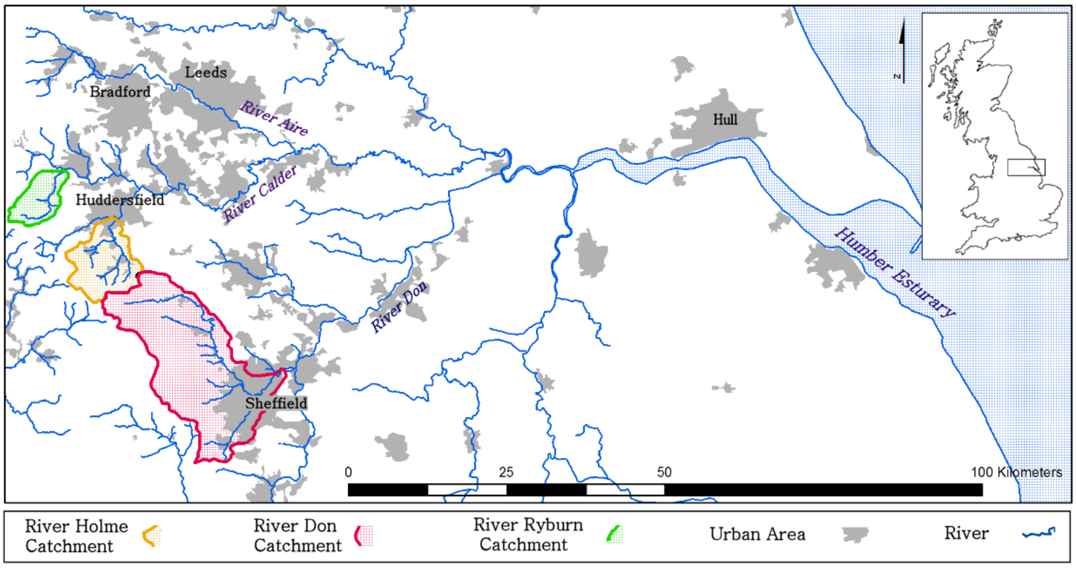


Figure 3-1 : A map of Yorkshire Region and UK locating the three study catchments. The catchments are colour coded; yellow for the Holme, red for the Don, and Green for the Ryburn. Rivers are shown in blue and urban areas in grey.

All three catchments outlined in figure 1 above are located within the Yorkshire region between 53.66°and 53.22° latitude. They lie on the eastern side of an upland area known as the Pennines and drain into the Humber river system. The highest points within the three delineated catchments was Moss Moor with a peak altitude of 463m AMSL (OS 2014). All three catchments experience an upland temperate climate (Kington 2010).

The local water company Yorkshire Water Services (YWS) operates approximately 130 reservoirs in the region, the majority of these being located within the Pennines. A working relationship was required with YWS to arrange reservoir releases outside of the normal operational schedule. 53 of YWS reservoirs were capable of releasing a peak flow > $1 m^3 s^{-1}$ and so were considered for study. After consultation with YWS, and site investigation of nine reservoirs, the final three were chosen. Sites investigation consisted of examining the channel to see if equipment could be safely deployed, and consultation was necessary to gain access to sites and the operating engineer.

The primary criteria considered for selection, other than accessibility, was reservoir discharge magnitude and the down river distance to the nearest STW outflow. In each of the three catchments a significant change in flow as a result of the wave release was needed to satisfy aims A4.1 – A4.4. The distance between the reservoir and STW was varied over the three catchments. To provide a scale comparison a large (>20km), medium (>5km) and a small (<5km) reach between reservoir and STW were required. Table 3-1 below details the size of the reservoirs and the distance to the point source to be diluted, as well as comparators from the relevant literature. The papers listed are those that described their field site in sufficient detail for a comparison to be made.

Table 3-1 Rivers, and reservoirs used either in this study, or within the literature. Release is the maximum release magnitude recorded for the reservoir in question. Distance to measurement is the distance between the reservoir and the farthest downstream water quality measurement site used in each study.

River	catchment size (km^2)	Reservoir	Release ($m^3 s^{-1}$)	capacity (m^3)	distance to measurement(km)	Reference
Holme	99	Digley	4.3	2867490	8.2	YWS pers comms
Ryburn	49	Ryburn	1.3	995370	3.1	YWS pers comms
Don	378	Underbank	9.7	3443200	26	YWS pers comms
Geum	3807	Daecheong Dam	200	790000000	78	Chung et al. 2008
Seine	78650	Seine Reservoir	26	205000000	64	Barillier et al. 1993. Malatre & Gosse. 1995
North Tyne	2933	Kielder Reservoir	16.6	200000000	15	Petts et al.,1985
Tryweryn	n/a	Llyn Celyn	3.94	71200000	5	Foulger & Petts., 1984
Mokelumne	1624	Cammanche Dam	57	532000000	54.4	Henson et al. 2007
Rio Tinto	793	Corumbel Dam	8.3	19000000	5	Canovas et al. 2012

The peak release discharges reported in this chapter are lower than the majority of those reported in the literature with only Canovas *et al*. (2012) having a comparable magnitude. The catchment sizes are also an order of magnitude smaller than many of those reported in the relevant papers. The distance down river between the reservoir and the primary water quality sampling points was of a similar scale to three of the six papers detailed in table 3-1 above.

3.4.1.1 The Primary Site, The River Holme

The River Holme runs 14.7 km from its source (SE112 068) upstream of Digley Reservoir to a confluence with the River Colne. Summary statistics for both the reservoir and catchment are recorded in table 3-1. The landscape is largely rural in the upland reaches with the urban centre of Huddersfield dominating the lower reaches. Digley reservoir, and the upper reaches of the river drain an area of moorland that give the water a browner colour and lower pH often associated with DOC (Worrall and Burt 2005). The lack of any heavy industry, or intense agriculture within the upper catchment limit the sources for potential pollution upstream of the STW site. The reservoir released water therefore has the potential to dilute pollution in the lower catchment.

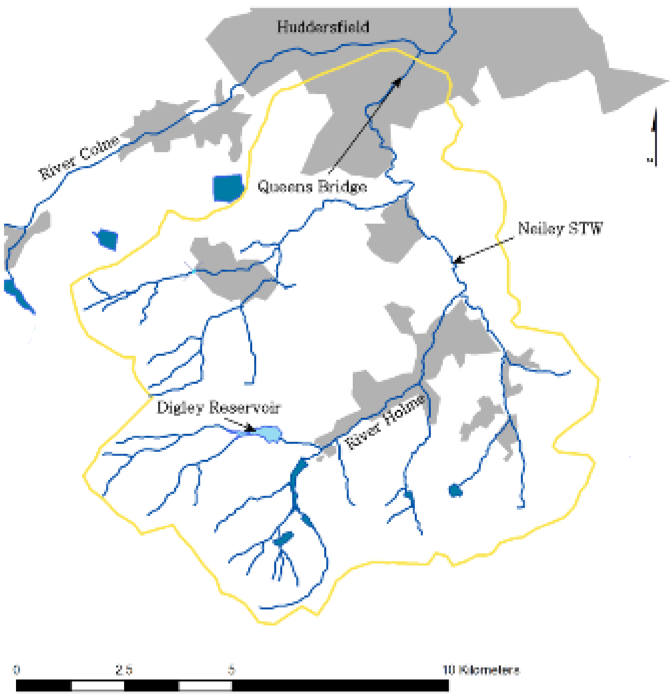


Figure 3-2. A map of the primary Holme Catchment. Arrows have been used to locate the reservoir used for release experiments, the STW site at which dilution of water quality was measured, and the Queens Bridge site at the bottom of the system used for stage and dye experiment measurements. Each of these sites is referred to extensively in the text.

In-river velocities, and consequently the velocities of both in river pollution and wave progression, are affected by substrate roughness, river topography (Engelund 1974) slope angle, and the presence of impoundments (Gilvear, 1989). The Holme river varies in width between 5m (within 50m of reservoir spillway at low flows) and 20 wide in some of the larger pools as measured on site visits. The upper ten km of the River Holme is dominated by pool riffle systems. As the river increases in size so do the scale of riffles and pools. Over much of the first kilometre below the reservoir the riffle pool systems span distances of less than 20 metres, with pool depths typically being under 0.5m. The width here ranges between 5 and 12 metres in baseflow conditions. Boulders over 1m in diameter are not uncommon and much of the bed sediment consists of course gravels varying in longitudinal diameter between tens and hundreds of millimetres. At 8.2 km downriver, the distance to the STW site, the riffle pool systems are less dense spanning lengths of 50m or more (Figure 3.5), and bed sediments consist of sands and clays between large boulders (typically <0.2m). Gravel bars occur on meanders and sediment within the river shifts with major flood events. Significant changes in the bed topography were noted after the July 2012 floods at the STW study site with significant deposition of fine sediments being generated near the banks. The other defining feature of the water course is manmade weirs. There are 18 flow control structures (figure 3-3 as an example) on the river Holme between its source and confluence. These occur at regular intervals and create an enhanced riffle pool system with water levels being increased and velocities slowed above the weirs and levels decreased and flows accelerated below them.

Figure 3-3. Photo of a weir on the River Holme. Numerous weirs such as this act to oxygenate the water as it flows downstream and maintain high river stage.

Figure 3-4. The stilling basin at Digley Reservoir during a release. Water is discharged at high pressure out of the pipe (right hand side of image) with considerable force, again oxygenating the water as seen in the later results sections.

Figure 3-5. A view down the River Holme downstream of the STW outflow. A riffle pool system can be seen in this image with the rough area of water in the centre of the image being the riffle. This image has been included to give a visual indication of the scale of the river.

The mean slope angle for the 14.7km reach of the river was 2.3º, topographical data such as this has an impact on potential energy and therefore potentially wave progression as discussed in Gilvear (1989).

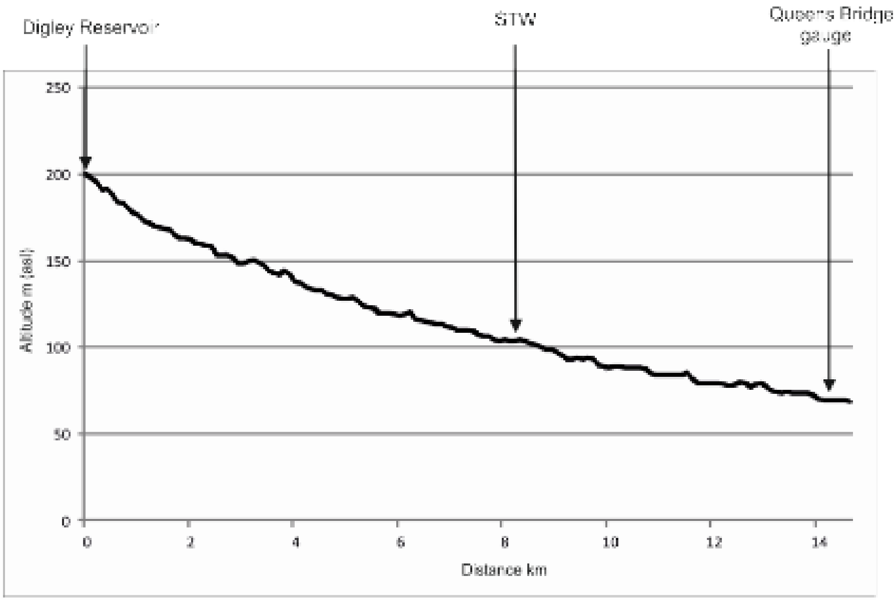


Figure 3-6. This line was constructed from interpolated Ordinance Survey contour data and consequently has some artificial bumps. The locations of the 3 data collection sites are indicated with arrows.

Slope angle decreases down catchment with a mean angle of 2.43 over the top 7.4km and 2.24 over the bottom 7.4km. It is possible that this change could influence wave celerity velocities over the two halves of the catchment.

3.4.1.2. Secondary Site, The River Don

As shown in table 3-1, the river Don with a catchment area of 378km^2 above Blackburn Meadows STW (Figure 3-7) is far larger than either the Holme primary catchment or the Ryburn secondary catchment. Underbank Reservoir (grid reference; SE253 991) is situated 26km up river from Blackburn Meadows STW in the foot hills of the Pennines. The reservoir drains an upland landscape containing heathlands, sheep grazing and mosaic of small woodlands. The reservoir acts as a source for the Upper Don tributary of the river Don. The Upper Don, and the upper reaches of the River Don are typical rivers of the region with a course cobble dominated substrate, a pool riffle system, frequent protruding Millstone Grit bedrock structures and dark dissolved organic carbon coloured waters. Like the Holme, this is conducive to slower flow velocities and a lower pH. As the river progresses towards Sheffield, it widens and deepens featuring regular culverts and hard engineering. Flow control impoundments occur at regular intervals along the river's course further slowing the flow and artificially deepening the water course. Two smaller scale STW release treated effluent into the river between Underbank and Blackburn Meadows. Stocksbridge STW is located 5.1km down river from Underbank Reservoir and processes sewage from the town of Stocksbridge and the surrounding villages. Wharncliff Side is a further 2.5km down river and receives sewage from a number of smaller villages. Both Blackburn Meadows and Stocksbridge STW were studied during the release experiments carried out in this catchment. Wharncliff Side was considered too small to warrant study given the limited personnel and field equipment available.

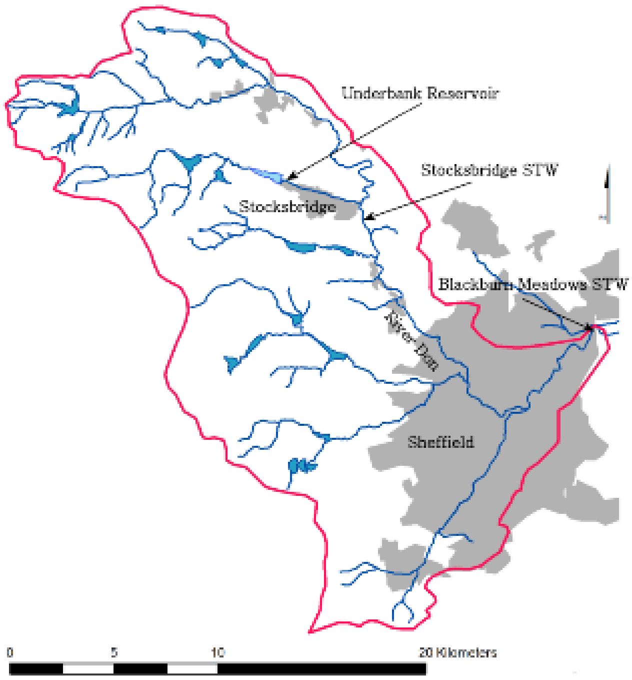


Figure 3-7. A map of the Don catchment above Blackburn Meadows STW. Underbank Reservoir and both of the STW sites studied are noted on the map.

As seen on figure 3-7 the urban area of Sheffield occupies the lower 10km of the rivers course. Blackburn Meadows STW serves the city as its primary STW and consequently has a significantly larger output than those studied in either of the other two study catchments.

3.4.1.3 Secondary Site, The River Ryburn

The Ryburn catchment at 49km^2 was the smallest studied. The peak release magnitude of 1.3m^{3}s^{-1} at the reservoir outflow, and the 3.1km reach between the reservoir and Ripponden STW are also both smaller than the other two catchments. In terms of peak release magnitude at 1.3m^{3}s^{-1} Ryburn reservoir has 31% of the capability of Digley Reservoir and 14% of that of Underbank Reservoir.

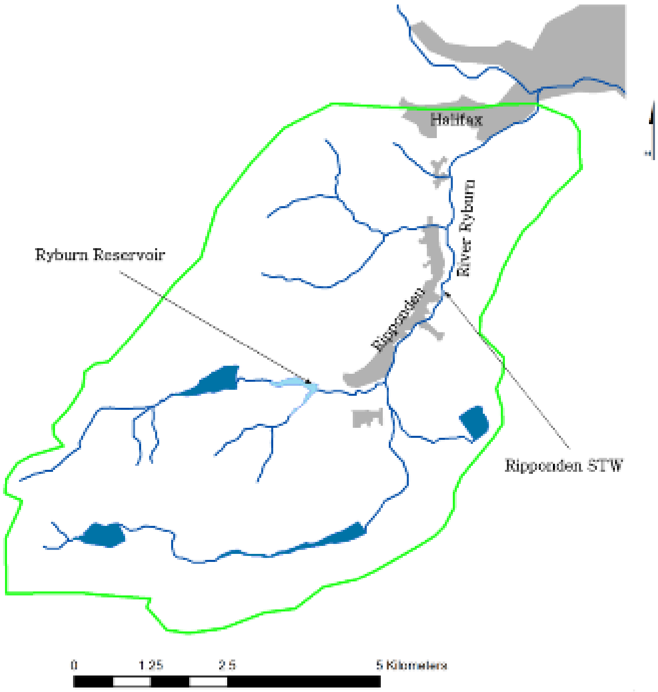


Figure 3-8. A map of the Ryburn catchment with Ryburn Reservoir and Ripponden STW labelled.

The River Ryburn is topographically similar to the upper reaches of both the Don and Holme catchment. A riffle and pool system, a gravel and boulder substrate and frequent artificial impoundments are all present.

3.4.2. Data Collection Methods

A general overview of the release experiments will now be given, then specific methods will be described in three subsections. The methods, as with the later results and discussion, have been ordered by aim as defined in the Aims section. The first section details methods that measure the velocity of both the reservoir release generated wave, and the baseflow as these are the factors that determine whether a wave will catch in transport pollutants and cover A3.1. The second section details the methods measuring water quality at both the STW outflow and reservoir inputs to the river system. Measuring water quality parameters at these locations during wave passage would quantify dilution, covering A3.2. A third section describes the methods that meet A3.3, addressing mixing processes within the water column. The questions listed under aim A3.4 are covered by the methods used to meet aims A3.1 and A3.2.

3.4.2.1 Release Experiments

A total of nine release experiments were carried out. Each experiment involved the release of water from a reservoir into the water course to generate a reservoir release wave. The dates, sites, and durations of these releases are described in table 3-2 below.

Table 3-2. The reservoir release program employed in the current study. Program refers to the release design and purpose. An explanation of each is given below.

Field site	Date	Duration (hours)	Program
Holme	26/10/11	4	Autumn flow
Holme	16/11/11	1.5	Experimental
Holme	14/03/12	4	Scour
Holme	29/05/12	2	Experimental
Holme	13/03/13	6	Scour
Holme	09/05/13	2	Experimental
Don	27/02/13	1.5	Scour
Don	05/06/13	2	Experimental
Ryburn	21/03/13	2.5	Scour

The Reservoirs Act (Reservoirs Act 1975) requires that all reservoir operators must test the operation of their drainage valves once every 12 months. YWS carries out two such releases at all its operated sites per year and refers to them as scour tests. These tests cycle the valves within the reservoir draw down pipes to assess their function and ability to discharge water from a reservoir in an emergency. These releases were pre scheduled by YWS but were suitable for data collection given their magnitude and flow characteristics. As each valve is opened and closed multiple flood peaks are generated producing a distinctive flood hydrograph. Of the other four reservoir releases, three were designed specifically for this study, and one was part of an autumn release program run by YWS. Unlike the scour valve tests, the autumn flow and the three experimental releases were all designed to have a single peak with a steep rising limb and slower falling limb similar to a natural flood event. The key difference between the three experimental releases and the autumn flow was duration, and the flashiness of the hydrographs. The three experimental releases were designed to be two hour flashy high magnitude hydrographs whereas autumn flow is much longer at 4 hours. The release experiments for the Holme site listed in table 3-2 can also be divided temporally by season. Two releases were carried out in late autumn – early winter, two in the early spring – late winter, and two in the late spring to early summer season. A summer winter comparison also dictated the timing of the two experiments on the Don, together these experiments deal with the question of seasonality laid down in A3.4. The two Holme Scour tests (14/03/12 and 13/03/13) and three experimental releases (16/11/11, 29/05/12, and 09/05/13) were intended to have similar hydrographs in order to replicate results. The two scours were also seasonally similar as were the two May releases. All releases at these reservoirs were generated by YWS technicians operating a manual valve. This combined with variations of pressure head within the reservoir lead to release hydrographs not being completely consistent, although still comparable (see figure 3.15 in the results section). This replication of experimental design deals with the question of replication of results in aim A3.4. Furthermore the experiment on 09/05/13 took place after sunset in order to examine the diurnal effect, again listed under A3.4.

3.4.2.2. Wave and Flow velocities

During the release experiments stage hydrographs were recorded at multiple sites as the release wave moved down river. The celerity velocity of these waves could then be calculated using the following formula;

$$v = \frac{d}{t}$$

equation 3.1

where v= velocity (wave celerity), d = distance, and t = time.

Timings input in to equation 1 drawn from hydrographs were based on the arrival of the wave peak rather than the initial incline in water as the peak was easier to define.

Distance, between the gauges was known (see gauge locations in figure 3-11) and stage was recorded at 15 minute intervals giving all time measurements for formula 1 at 15 minute resolution. On both the Holme and Don sites gauge data for one locale was supplied by the EA. 15 minute resolution was the highest resolution that the Environment Agency (EA) could provide for both gauges, therefore primary data collected for this chapter matched this interval. The gauged used to represent discharge at the Blackburn Meadows STW site was located 1km upstream of the STW site as this was the closest available.

At the Holme and Ryburn sites gaugings at weirs were used to construct rating curves, to convert stage to discharge, for these hydrographs in line with the methods presented in (Herschy 1998). An Ultrasound Doppler Current Profile (UDCP) system and electromagnetic flow meter systems were used to collect velocity area gaugings. The rating curves for two sites at these catchments are provided below in figures 3.9 and 3.10. The River Don during wave passage reached velocities of over $2ms^{-1}$ and therefore could not be gauged by hand safely. Hydrograph data for sites not either gauged by the EA on the Don is given as stage in the results.

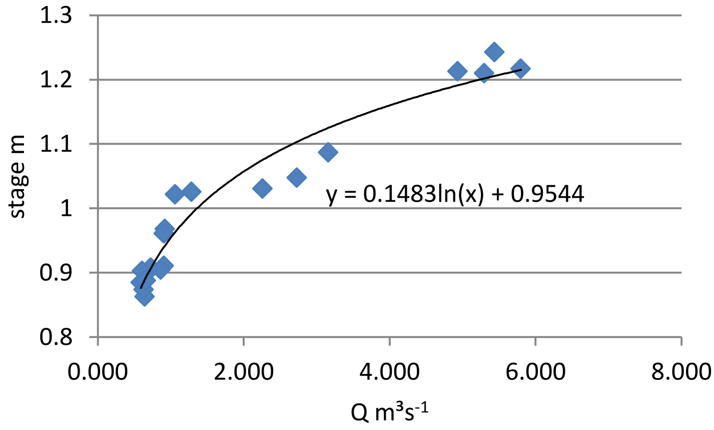


Figure 3-9. Primary Holme site rating curve for flow downstream of the STW outflow.

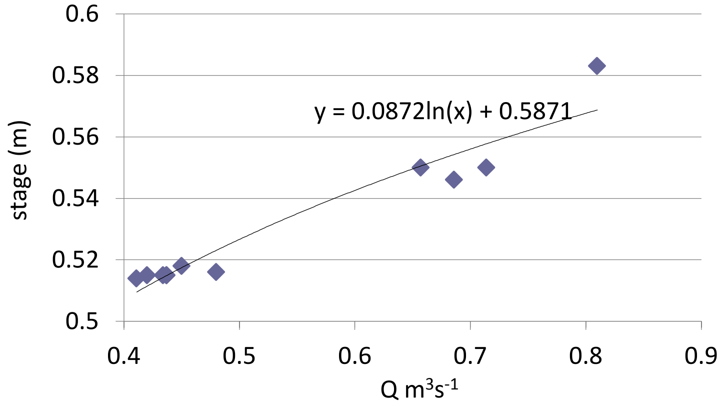


Figure 3-10. Secondary Ryburn site rating curve for flow downstream of the STW outflow.

In addition to measuring discharge to construct ratings curves, the UDCP and electromagnetic flow meters were used to measure velocities through the water column in baseflow conditions, that is flow conditions outside of those generated during release experiments, in four different flow environments within the River Holme. These environments included in a pool, in a riffle, 10m downstream of a weir, and in a straight engineered reach of the river. Each of these environments was selected to represent a variety of flow conditions within the river. If the wave celerity velocity was higher than these river section velocities it follows that the wave moves faster than the baseflow and would catch a slug of polluted water.

3.4.2.3. Dye Test

A dye tracer experiment was carried out during the release experiment that occurred on 09/05/13 and on a control non release date of 08/10/13. The primary purpose of this experiment was to compare the rate of wave progression down river with the rate of dye progression. This would provide a real world test for aim A4.1 of this chapter and a direct result to answer hypothesis H1 (Kilpatrick and Wilson 1982). The secondary purpose of this experiment was to compare the chemographs produced by the release experiment and the control experiment as this gives an indication of dilution potential of the wave. On both dates 2 litres of 40% Rhodamine WT dye were injected into the centre of the River Holme at the STW outflow 8.2km from Digley Reservoir. At Queens Bridge, 14.2km down river from Digley reservoir water samples were extracted from the river every 15 minutes. The input and sampling sites are shown on figure 3-12.

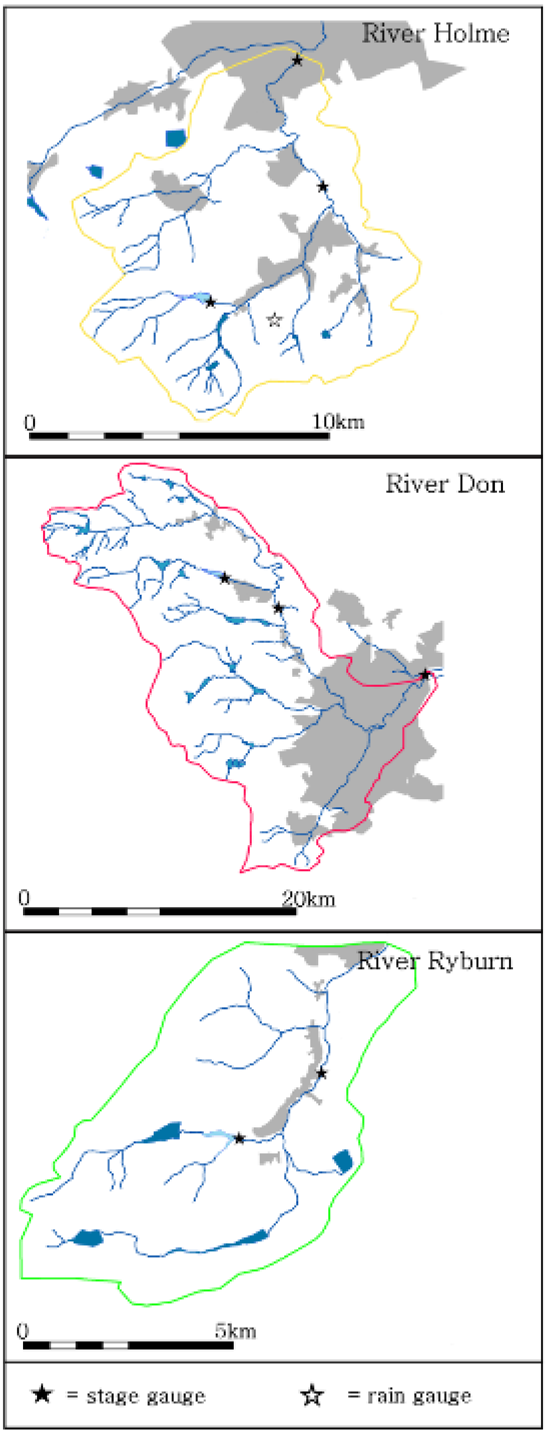


Figure 3-11. Gauge locations for the three field sites.

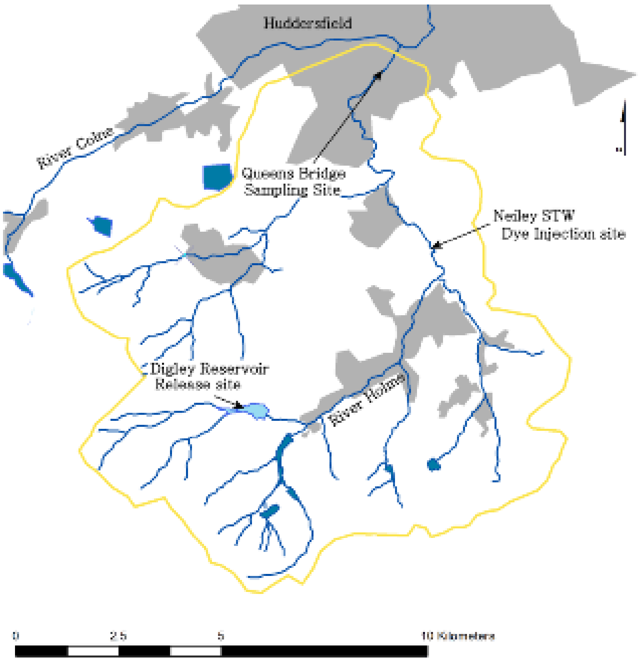


Figure 3-12. The three sites of importance are labelled on the map; the reservoir release at Digley Reservoir, the dye injection atthe STW, and the water sampling site at Queens Bridge in Huddersfield.

Due to the strong colour of the dye both the control and wave release experiments were carried out at night. On the 09/05/13 the valve at Digley reservoir was opened at 20.00 for two hours to release a wave. The two litres of Rhodamine WT dye (40% concentration) were injected using a Nalgene bottle in the centre of the water course at the STW outflow site at 21.00. Water sampling at Queens bridge started at 22.00 and continued at 15 minute intervals until 05.00 on the 10/05/13. On the control experiment date, 10/10/13, no wave was released from the reservoir. Dye was injected in the same fashion, but at 23.00. Sampling started at 00.00, and continued until 11.00 on the 11/10/13 with samples being collected from the centre of the river via a bucket from a bridge. Dye samples were measured with Tuner designs SCUFA fluorometer to ascertain Rhodamine WT concentration.

The one hour delay between releasing the wave from the reservoir and the injection of the dye was necessary to allow transit between the reservoir and the dye release site. A longer sampling period was employed in the control experiment as the dye slug took 9.5 hours to arrive at the sampling site. This was considerably longer than was expected.

3.4.2.4 Anecdotal Flow Observations

During each of the 2013 releases two anecdotal stage observations were recorded at each release event in response to findings of the flume and CFDM chapters. In these chapters waves simulations were found to have very fast rising limbs. Extra stage measurements were taken during the rising limb. These measurements were taken by hand with a ruler and not in a systematic fashion. Rather a stage measurement was taken prior to wave arrival and then during the rising limb if stage increased noticeably within a 15minute interval spot measurements were taken.

3.4.2.5 Water Quality Change

Water quality parameters were measured at the reservoir and STW inputs into the river system during reservoir release experiments. Measurements were taken at 15 minute intervals starting at least 30 minutes before wave arrival and continuing until at least 1 hour after baseflow conditions had returned. These two locations were presumed to be the dominant influence on water quality change in the system. The STW reduces water quality (data demonstrating this is provided in tables 3-10 to 3-12 in the results section), and the reservoir inputs water from a limnological system (Petts 1984) which can potentially be a source of pollution (Park and Curtis 1997; Tanik *et al.* 1999), , as well as a source of good quality water for dilution. Measuring water quality of the STW outflow provides a quantification of the impact of the STW on the system and the dilution achieved during wave passage. Measuring water quality at the reservoir validates that the reservoir is of good quality for dilution, provides a further control and insights into the adverse effects of a reservoir release contributing to aim A3.4.

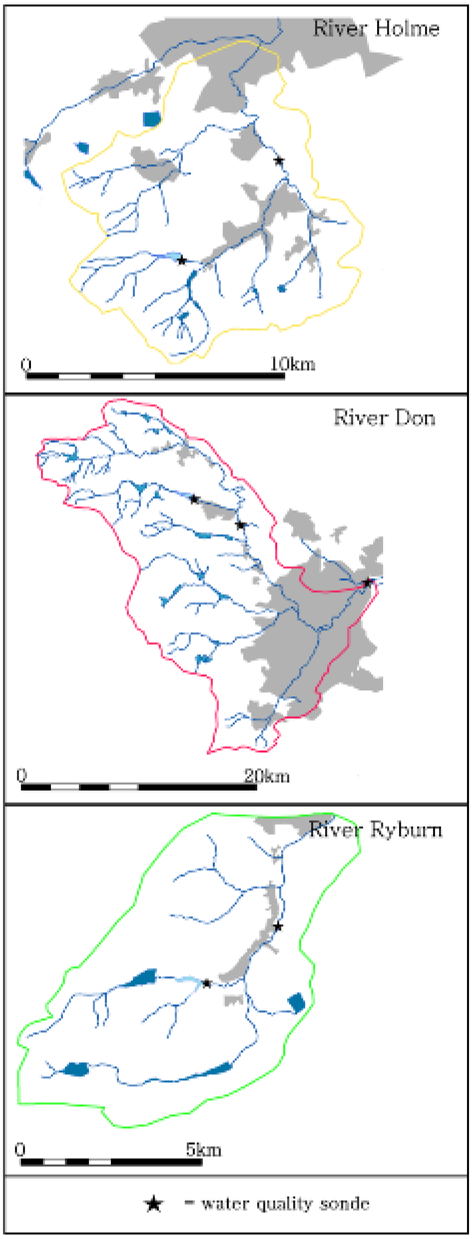


Figure 3-13. Locations of the sondes employed to measure water quality during release experiments at the three field sites.

In the primary Holme catchment water quality data was also collected for a 24 month period between September 2011 and 2013 at 15 minute intervals at both the STW outflow and the reservoir stilling basin to provide a control to compare the release experiments too. Changes in water quality during wave passage would demonstrate the quantity of dilution that could be achieved for a given release of water meeting aim A3.2.

As described in the literature review in chapter 2, STW outflows are often associated with increases in electrical conductivity, nutrients, specifically NH_4 (Crumpton and Hersh 1987) and a decline in DO (Morrison *et al.* 2001). In addition to these primary pollution indicators pH, temperature and turbidity can also be affected (Martinelli *et al.* 1999). Each of these water quality parameters was measured with an in river YSI Sonde device.

Several water quality parameters were omitted from the study. Phosphorus, and its compounds are a key pollutant associated with treatment works (Taebi and Droste, 2004b), as are microbes (Bell *et al.* 1980; Ouattara *et al.* 2014). Other nitrate compounds or measures such as total nitrogen are also considered in the literature (Capella *et al.* 2014; Guo *et al.* 2014). Both nitrogen and phosphorus compounds can be either in solution or sediment bound making sampling and measurement more complex. Microbes similarly require more complicated methods than insitue use of probes. Given the number of experiments undertaken in this study, time spent on preparation and processing of samples had to be kept to a minimum. NH_4 and conductivity were sufficient examples of pollution for quantification of dilution.

Table 3-3. Water quality parameters on the YSI sonde device and the instrumentation used to measure them in the current study.

Water Quality Parameter	Instrument method
conductivity	electrical conductivity
NH_4	Ag/AgCl electrode
DO	luminescence, rox optical
Temperature	thermistor, metallic oxide
pH	electrode, hydrogen Ion
Turbidity	optical nephelometric LED

With the exception of NH_4 and conductivity all of the instrumentation methods on table 3-3 above are optical. Consequently the sonde system had to be cleaned in river prior to release. This involved flushing the sonde with river water to remove any solids or biofilm build up. The sondes were calibrated by the Environment Agency and replaced on a monthly basis, and immediately prior to releases at the secondary sites. Turbidity results have not been converted to SSC concentrations. It is possible to collect field samples and calibrate the turbidity measurement to an SSC figure. However Gilvear and Petts (1985) demonstrate the varied relationship between turbidity and SSC during a release event, therefore converting turbidity to SSC must be considered questionable.

Each sonde was suspended in the water column from above and took measurements at 15 minute intervals. Either a tree over hanging the water or a horizontal pole were used to suspend the sonde at least 1m from the bank. On each site at the reservoir, a sonde was deployed within the stilling basin, that is the pool area that the reservoir outflow pipe flows into prior to the water running into the river course. At each STW site the sonde was deployed within 200m downstream of the STW outflow to allow for the outflow to mix with the river course waters.

3.4.2.6 Mixing in the Water Column

Vertical mixing of reservoir release water in the water column was recorded by measuring conductivity at three depths during wave passage. STW outflows typically raise conductivity levels, whereas reservoir water usually has a low conductivity level (see tables 3-10 to 3-12 for supporting field data). By measuring the differential in conductivity between the three depths in the water column incomplete mixing of the reservoir and riverine waters could be identified. This data would deal with aim A3.3 of this chapter.

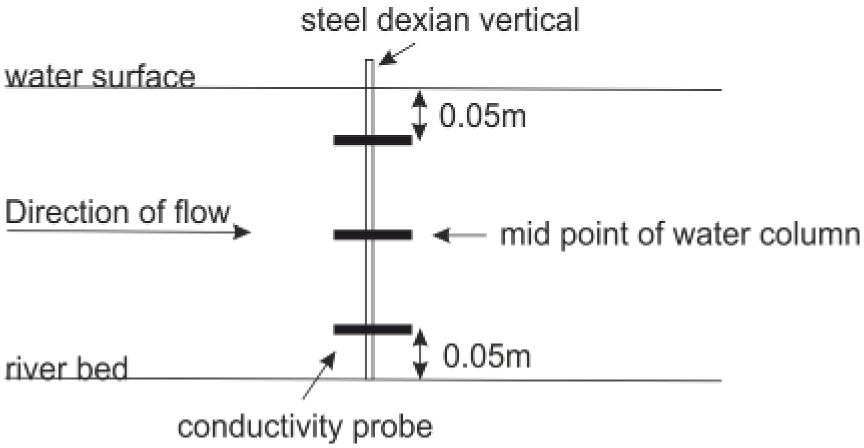


Figure 3-14. The black rectangles represent conductivity probes at three heights in the water column

Three INW CT2x conductivity probes with a 0.1µscm resolution were deployed into the river in a vertical profile. The ‘top’ probe was deployed 0.05m from the surface of the water column, the ‘middle’ was deployed at 50% point in the water column, and the ‘bottom probe’ was deployed at 0.05m above the river bed. These three heights were used to provide stratigraphy that accounted for the edge effects of the water surface and the river bed, and to replicate the results demonstrated in chapters 4 and 5. At these heights each probe was attached to a metal dexian pole, which had been driven into the river bed near the mid channel point. Both the Holme and Don rivers have a shallow sediment substrate underlain by a millstone grit bed rock. The probes were always deployed more than 1m from the river banks so as to avoid the extremes of lateral mixing, and in water with a depth of over 0.4m, however the exact position of the probes varied between events. Given that the sampling location at each field site was always >50 from the STW outflow, downstream variation of <10m in position would not be noticeable. It is possible that the magnitude of vertical stratification could be affected if the probes were within 2m of the bank rather than at the centre of the water column. The purpose of this experiment was to establish a pattern rather than produce perfectly comparable results between experiments, therefore slight variations in position were considered unimportant. The probes were deployed in both the Holme and Don catchments at the Neiley STW (figure 3-6) and Stocksbridge STW (figure 3-7) water quality measurement sites respectively. This data set was only collected during six experiments, four on the Holme and two on the Don due to equipment availability.

3.4.2.7 Dilution Calculations

It is useful for water managers to be able to consider dilution in terms of units of water, or as a consideration of effective use of water. Releasing a wave, as shown later in the results, produces a chemograph curve with concentration decline and then recovery of a pollutant such as NH_4. It is difficult to equate an exact magnitude of release or total volume of water released to a specific level of dilution because the wave produced and the chemograph are curves that vary through time. Giving a straight dilution per unit water spent would be an overgeneralisation and misleading even if it is desirable. Instead a number of measures of dilution are given;

Percentage dilution; for the River Holme and Ryburn sites control data from 3 days prior to and 3 days post the release experiment was recorded. It can be seen in figure 3-38 in the results that concentration of both NH_4 and conductivity were not constant, as the inputs vary throughout the day. Consequently for each experiment a week of control data was used to estimate, by taking the means of the 6 days of data, the concentration of a pollutant on the experiment day in the absence of the wave. The percentage reduction in concentration as the result of the wave could then be calculated by comparing the mean result from the control days and the actual concentration under the influence of the wave. Because only experiment day data was available for both Don experiments a percentage dilution was not given. An argument could be made for taking the concentration prior wave arrival and assuming it remains constant to then calculate dilution. This however would be questionable. It is well known that output from STW is not constant both from the literature (Jordan *et al.* 2007), and the data presented in figure 3-38.

Regression between flow and contaminant concentration (conductivity and NH_4); A regression between the hydrograph and the chemograph if significant having a high R^2 value (>80) could be used to establish a relationship. Third order polynomial curves were fitted to the results for each of the Holme experiments. If similar equations were derived for these lines a predictive model could be established.

Dilution duration against wave duration; It is useful for a water manager to be able to relate their actions to results in simple terms. If the valves at the reservoir are opened close to maximum output for a given time how long does the dilution effect last? This measure compares the duration of the release wave at the reservoir input end of the system to the duration of dilution. Rather than take an arbitrary level of dilution, the duration of the chemograph as a whole is considered, from the initial drop in concentration to recovery to stable state.

Lag time; The time between peak dilution and peak flow is indicative of the kinematic nature of the wave. If a lag between wave arrival and change in water quality occurs it suggests that the wave front has outpaced the reservoir water as described by Glover and Johnson (1974).

Each of these calculations is detailed in tables 3-6, 3-7, 3-8 and 3-9 within the result section. Where appropriate some of these results are also described within the text.

3.4.2.8 Limitations

A major pollution incident was not diluted. One experiment reported dilution of 1.1mgl^{-1} NH_4 which exceeds the EQS level, but in general high pollution was not witnessed in any of the experiments. The conclusions drawn in this chapter are therefore limited to low pollution situations, and only predictions can be made concerning dilution of higher concentration pollution.

The sampling regime was limited to two water quality collection points. The method has dealt with the input at the reservoir and an output result downstream of the STW outflow. How the water quality varies between the reservoir and the STW, the relative contribution of each tributary, the role of any industrial or unregistered inputs are all unknowns. On the River Don for instance there are known to a number of steel plants and industrial facilities with discharge consents.

A 15 minute sampling resolution was low for this type of study, Kurtenbach *et al.* (2006) sampled conductivity at 6 minute intervals, and Gilvear and Petts (1985) sampled SSC at 4 minute intervals through the rising limb of their study. Whilst it was desirable to have a high resolution sampling regime, studies such as Barillier *et al.* (1993) sampled less frequently at 2 hour intervals over a release lasting more than 2 days. A 15 minute sampling interval had three effects: First, any rapid fluctuations in water quality or stage would not have been detected because SSC is known to fluctuate during the rising limb of a hydrograph (Petts 1984). Second, any lag time between the arrival of the wave front and dilution that occurred over a smaller time interval that 15 minutes will not have been detected. Third, peak flow and peak dilution during any experiment may be under represented if they occurred between samples.

The 15 minute sampling resolution was necessary to make a 2 year sampling regime practical.

As only two water quality sondes were available rather than the >5 needed to cover each tributary and input into the river, and as high resolution outflow data from the STW was not available dilution could not be calculated from the system inputs. Therefore samples from the 3 days either side of the experiment day were used to estimate how much dilution was achieved downstream of the STW. These 6 samples whilst giving a good range for comparison are not necessarily an accurate representation of the water quality conditions in the river on the experiment day in the absence of a wave. As shown in figure 3.38 in the results section water quality parameters such as NH_4 and conductivity can vary in concentration significantly from day to day and as such the comparative samples must be viewed as an estimate.

Flow velocity measurements were taken in order to characterise the flow speeds through different geomorphological features of the channel, these are presented in figure 3.18. Whilst intended to represent likely flow conditions within the River Holme as a comparator to the wave velocities, these four locations cannot by any objective analysis considered to be representative of the river at the catchment scale or all the different flow conditions that the river might experience in a given year.

Anecdotal results are reported within this chapter. They are described as anecdotal as they were not collected in a systematic and scientific manner. The majority of the methods presented within this chapter were based around electronic equipment, the >15 minute supplementary stage measurements were late additions to the field method designed to take advantage of events seen in the field. As anecdotal observations they have limited value and simply indicate when very rapid increases in stage occurred.

Key results such as wave velocities are generalised by distance between gauges rather than reach specific. Ideally gauges would have been introduced every kilometre, as this was not possible a lower resolution result is given. A water manager using such wave velocities would need to be aware of this limitation.

3.5 Results

Results are presented here in the same order as the methods were given, first dealing with wave propagation and water velocities and second results collected for water quality and pollution dilution. Finally results from the vertical mixing experiment are reported.

3.5.1 Wave and Flow Velocities

This section deals with the question of how quickly a wave of water released from a reservoir can catch up with an area of polluted water. The data using in this analysis were recorded from the primary Holme field site unless otherwise stated. This data relates to aim A3.1 of this chapter. To avoid unnecessary replication of graphs the flow results from the secondary sites are described when hydrographs are first displayed in the conductivity section of the results rather than here.

The discharge at Digley Reservoir peaked at $3.1m^3s^{-1}$ or greater in each experiment with the highest peak of $4.1m^3s^{-1}$ being achieved during the 29/05/13 release. The hydrographs generated by the reservoir take broadly three forms; The releases on 26/10/11 and 29/05/12 have a sharp rising limb, 1 hour, and 30 minutes respectively and then a long declining limb, 3 hours 30 minutes and 2 hours 30 minutes respectively. Both events were designed to be closer to a natural flood in hydrograph form. The experiments on 16/11/11 and 09/05/13 were both sharp, flashy pulses of increased flow, and are the most comparable in terms of replicating individual release waves. On the 16/11/11 the release was only 1 hour 30 minutes and on 09/05/13 2 hours, both have a 15 minute rising limb, a plateau and then a 1 hour falling limb. Both 14/03/12 and 13/03/13 experiments were reservoir scour tests, so valves were opened and closed to check their operation, hence there are multiple flashy flow peaks in each event. Again the scour test hydrographs are comparable, although not identical. Discharge prior to wave arrival ranged between $0.6m^3s^{-1}$ on 26/10/11 and $1.1m^3s^{-1}$ on 13/03/13, demonstrating a variation in antecedent conditions as mentioned under aim A3.4.

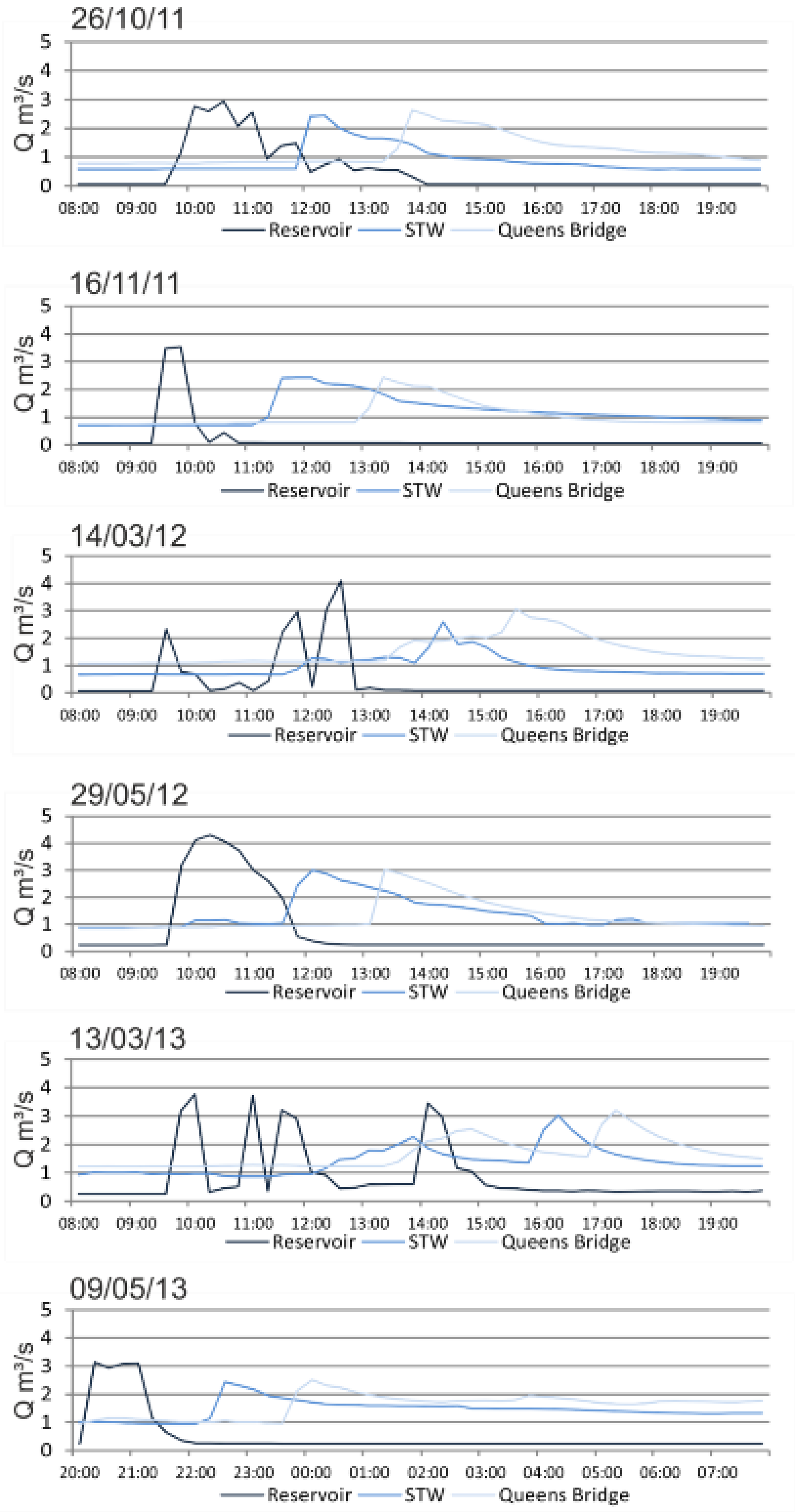


Figure 3-15. Discharge hydrographs for the release experiments carried out in the Holme primary catchment. The locations of the gauges can be seen in figure 11.

Despite the reservoir producing longer duration hydrographs on the 26/10/11 and 29/05/12 than on the 16/11/11 or 09/05/13, the resulting hydrograph down river at the STW and Queens Bridge sites are not dissimilar in both profile and wave length. The wave lengths are 5 hours 30 minutes, 4 hours, 4 hours and 6 hours going chronologically forward through the experiments at the Queens Bridge site. The peak discharges range between 3.02 and 2.4 m^3s^{-1} at the Queens Bridge site. The two multipeak scour release experiments produce two peak waves further downstream. The second wave peak is of higher magnitude in both events despite a greater volume of water being released earlier in the 13/03/13 experiment.

As described in the method, gaugings for the Don site could only be obtained at the Blackburn Meadows site, so flow data at both Underbank and Stocksbridge is given as stage. On the 27/02/13 discharge at Blackburn Meadows increases from 2.22m^3s^{-1} at 15.45 gently to 4.16m^3s^{-1} by 17.15 this then recedes to 2.46 by 20.15. Up river similar hydrographs are seen at Underbank and Stocksbridge with a 45minute delay between peak flows at the two sites. At Underbank stage rises from 10.45 to a peak of 0.45m at 11.45 to then return to pre-release conditions by 12.30, at Stocksbridge this translates to a rise from 0.21m at 11.45 to a peak of 0.54m at 12.30. The wave that reaches Blackburn Meadows on 05/06/13 has nearly twice the peak magnitude of the first at a peak discharge of 7.92m^3s^{-1} at 14.45 having risen from 1.48m^3s^{-1} over the previous hour. The receding limb then lasts another 7 hours. Again stage graphs at the two up river sites are comparable with peak stages of 0.68m and 0.65m at the Underbank and Stocksbridge respectively. Both sites feature a plateau in flow lasting between 1 hour 30 minutes and 1 hour 15minutes.

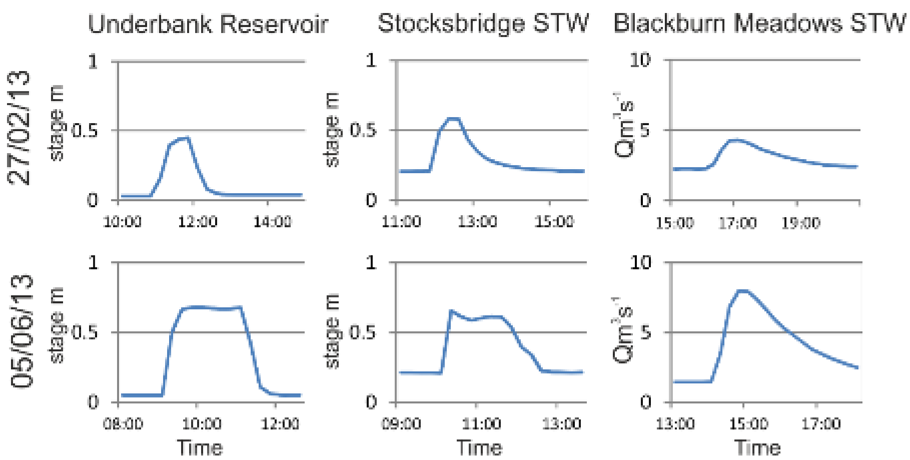


Figure 3-16. Discharge hydrographs from the two experiments on the River Don by gauge site.

Discharge during the Ryburn release rose from 042.m^{3}s^{-1} to 1.28m^{3}s^{-1} upon wave arrival. Discharge then declines over the next 30 minutes to 0.53m^{3}s^{-1} to be followed by a limited second peak at 0.75m^{3}s^{-1} and an eventual decline back to prevent levels after two more hours.

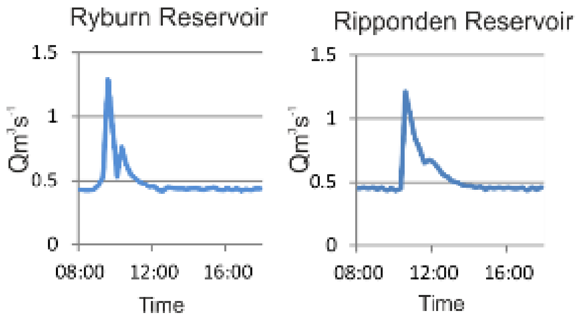


Figure 3-17. Discharge hydrographs from the experiment on the River Ryburn.

Table 3-4. The travel times and velocities for the wave in each experiment first between the reservoir and the STW, and then between the STW and the Queens Bridge site at the bottom of the primary Holme Catchment. All velocities were calculated with equation 1, given in the methodology.

Release date	26/10/2011	16/11/2011	14/03/2012	29/05/2013	13/03/2013	09/05/2013
travel time from reservoir to STW hh:mm	01:45	02:00	01:45	01:45	02:15	01:45
wave celerity velocity from reservoir to STW ms^{-1}	1.30	1.14	1.30	1.30	1.01	1.30
travel time from STW to Queens Bridge, hh:mm	01:30	01:30	01:15	01:15	01:00	02:00
wave celerity velocity from STW to Queens Bridge ms^{-1}	1.20	1.20	1.44	1.44	1.81	0.90
travel time from reservoir to Queens Bridge, hh:mm	03:15	03:30	03:00	03:00	03:15	03:45
wave celerity velocity from reservoir to Queens Bridge ms^{-1}	1.26	1.17	1.36	1.36	1.26	1.09

Waves released from the Reservoir took a mean 3 hours and 30 minutes to travel the 14.7km to the Queens Bridge. Wave velocities within the catchment exceed 1ms^{-1} in all but two cases, the reach between the reservoir and STW on 13/03/2013 and the lower reach between the STW and Queens Bridge on 09/05/2013. Four release events show an acceleration of the wave velocity between the upper and lower reaches contrary to any expectations based upon channel slope (figure 3-6). Over the complete 14.7km reach of the River Holme the fastest wave was that released on 14/03/2012 at 3 hours, In contrast the slowest waves took 3 hours and 45 minutes.

Wave velocity data for the secondary sites is reported in table 3-5 below. Over a much greater distance of 25km, waves released from Underbank reservoir achieved a similar velocity (1.26ms^{-1}) to those studied on the primary site. The velocity achieved on 05/06/13 on the Don is substantially faster, though, at 1.63ms^{-1} taking 1 hour and 15 minutes less time to traverse the 25km river reach. Of all the waves studied, the wave released on the Ryburn on 21/03/2013 is the slowest at 0.86ms^{-1} taking 1 hour to cover 3.1km.

Table 3-5. Velocity, travel time and distance of wave propagation for the two secondary field sites.

Field Site	River Don	River Don	River Ryburn
release date	27/02/2013	05/06/2013	21/03/2013
travel time from reservoir to STW	05:30	04:15	01:00
Wave velocity from reservoir to STW ms-1	1.26	1.63	0.86
distance from reservoir to STW gauge km	25	25	3.1

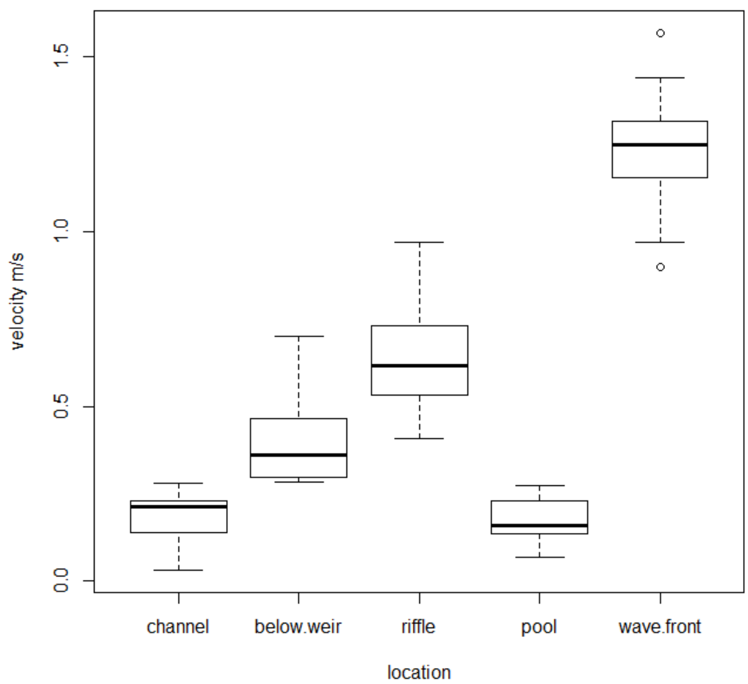
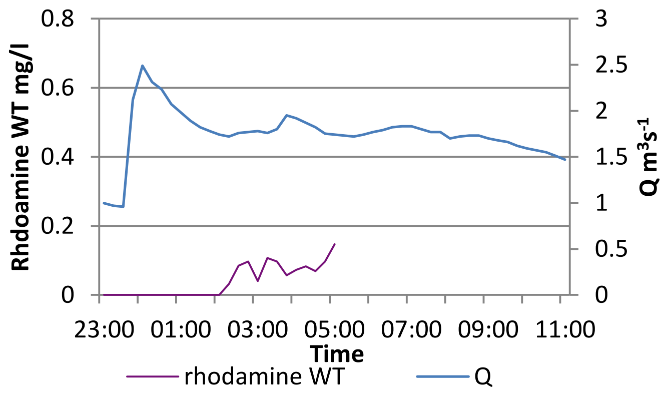


Figure 3-18. A box plot of flow velocities recorded at four morphologically defined sites within the river. A box plot of the wave front velocities as detailed in table 4 has been added for comparison. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

Outside of natural flood conditions velocities in the river do not exceed 0.8ms^{-1} at any given location. The mean flow velocity for the whole in river data set is 0.36ms^{-1} with the locations that are most representative of the largest reaches of the river, the channel, and the pool having means of 0.18ms^{-1} and 0.17ms^{-1} respectively. This data provides an empirical basis for meeting aim A3.1 of this chapter and aim A1 of this overall thesis.

In the initial experiment (figure 3-19) it was underestimated how long the dye would take to arrive, so this data set is truncated at 5:00. This limits the value of the data in terms of discussing relative dilution levels. The data is still highly relevant to the discussion on flow velocities and progression times required by aim A3.1.

(a)



(b)

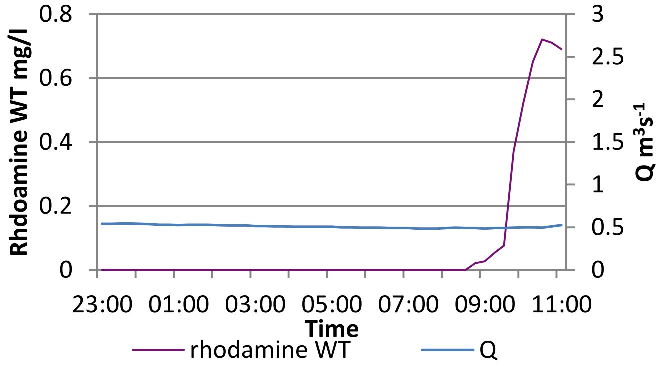


Figure 3-19. Rhodamine WT is shown with the dark purple line and flow with the blue. In both graphs the flow is Environment Agency supplied data. Panel (a) shows data collected on 09/05/13 during a release experiment, and panel (b) shows the control experiment on 11/10/13. As described in the method this data was collected at the Queens Bridge site 14.7km from the reservoir and 6.5km from the STW.

On 09/05/13 (a) the wave front, having been released from the reservoir at 20.00 covers the 14.7km in 3 hours 15 minutes to arrive at 23.15. The dye, by comparison, is released at 21.00 and is first detected at 02.00 taking 5 hours to cover 6.5km (having been released from the STW rather than the reservoir). This gives the wave a mean celerity of 4.5kmh^{-1}, or 1.25ms^{-1}, and the dye slug a mean velocity of 1.3kmh^{-1} or 0.36ms^{-1}. Consequently both the dye and the wave would cover 9.1km in 2 hours 15 minutes and converge before reaching the confluence with the Colne. During the control the dye takes 9 hours 45 minutes to cover 6.5km or 0.185ms^{-1}. Peak concentrations of 0.147mgl^{-1} and 0.72mgl^{-1} were recorded respectively during the experiment and control. Discharge during the control night, a mean of 0.51m^{3}s^{-1} has an exceedance probability of 99.42%. As a comparator, a discharge of 1m^{3}s^{-1} at the Queens bridge site has a 89.81% exceedance probability. The conditions present in the control experiment can be considered low flow. Both these flow duration figures probabilities are based upon daily averages data provided by the EA for the Queens Bridge site between 01/10/2011 and 01/11/2013.

3.5.1.2 Anecdotal Results

The anecdotal results presented here are of minimal relevance to the discussion in this chapter but are necessary for the argument in chapter 6 and comparing the waves generated in this chapter with those seen in the flume and the CFDM of later chapters.

The only wave to generate an extremely rapid rise in stage occurred on 05/06/13 on the Don. Spot stage measurements taken in this event identified a rise in stage of >30cm between 10.15 and 10.20.

3.5.1.3 Froude Numbers

The waves produced in the flume of chapter 4 were found to be supercritical in nature. To determine how similar the waves in the flume were to those measured in the field experiments Froude numbers were calculated.

A flow of water can be described as supercritical if its Froude number is >1. The Froude number being a dimensionless term that is defined by the ratio between the gravitational forces and flow velocity.

$$\mathrm{Fr} = \frac{U}{\sqrt{g\frac{A}{B}}}.$$

Equation 3.3

Where U is velocity in ms^{-1}, g is the gravitational acceleration in ms^{-1}, A is the cross sectional area and B is the free surface width, both in m. Froude numbers were calculated from wave celerity with factors A and B being estimated from a cross sectional profile and data recorded during river gauging. Froude numbers were not calculated for the Don catchment as factors A and B were not known.

Table 3-6 Froude numbers for the experiment waves on the Holme and Ryburn catchment as calculated at the STW gauging sites.

Release date	26/10/2011	16/11/2011	14/03/2012	29/05/2013	13/03/2013	09/05/2013	21/03/2013
River			Holme				Ryburn
Froude Number	0.481	0.446	0.513	0.508	0.468	0.416	0.361

All of the Froude numbers calculated are below 1, therefore subcritical. Whilst Froude numbers were not calculated for the River Don releases if the wave celerity of 1.63 is taken as U, then the ratio between A and B would need to be less than 0.27 to generate a supercritical flow. Such a ratio would only occur is a very broad shallow area of water which would be uncommon on the Don. It can therefore be inferred that except in areas of the river where the flow takes the form of a shallow rapids supercritical waves are unlikely to occur within the experiments conducted within this chapter.

3.5.2 Water Quality Change

This section relates to aim A3.2 of this chapter, aim A2 of the thesis as a whole. The following graphs and tables detail the dilution achieved during wave passage at the STW outflow sites at each of the three study sites. Conductivity and NH_4 are given particular attention as these are the primary water quality parameters associated with STW derived pollution. DO could also be considered as important, however an appraisal of the results that follow will show that the impact of wave arrival on DO was minimal.

3.5.2.1 Conductivity

Reservoir release waves have a discernible and often immediate effect on conductivity concentrations. In each of the reservoir releases at the primary Holme site in figure 3-20 conductivity declines during wave passage. This dilution is further detailed in table 3-7. Each of the non-scour test, single peak releases generates a similar hydrograph at the STW and similar dilution curve despite the variations in duration and magnitude delivered from the reservoir. The shape of the conductivity chemograph is often related to the shape of the hydrograph, with the two spring scour tests, 14/03/12 and 13/03/13 on the Holme being good examples of this as the peaks in conductivity dilution have minimal lag time from peak flow. This is reflected in the correlations between flow and conductivity given in table 3-7. Conductivity levels at Digley Reservoir are consistently very low, a mean of 74µscm, and are often not impacted by the release with 13/03/13 and 09/05/13 showing minor declines of 132 and 10µscm respectively for a period of less than an hour.

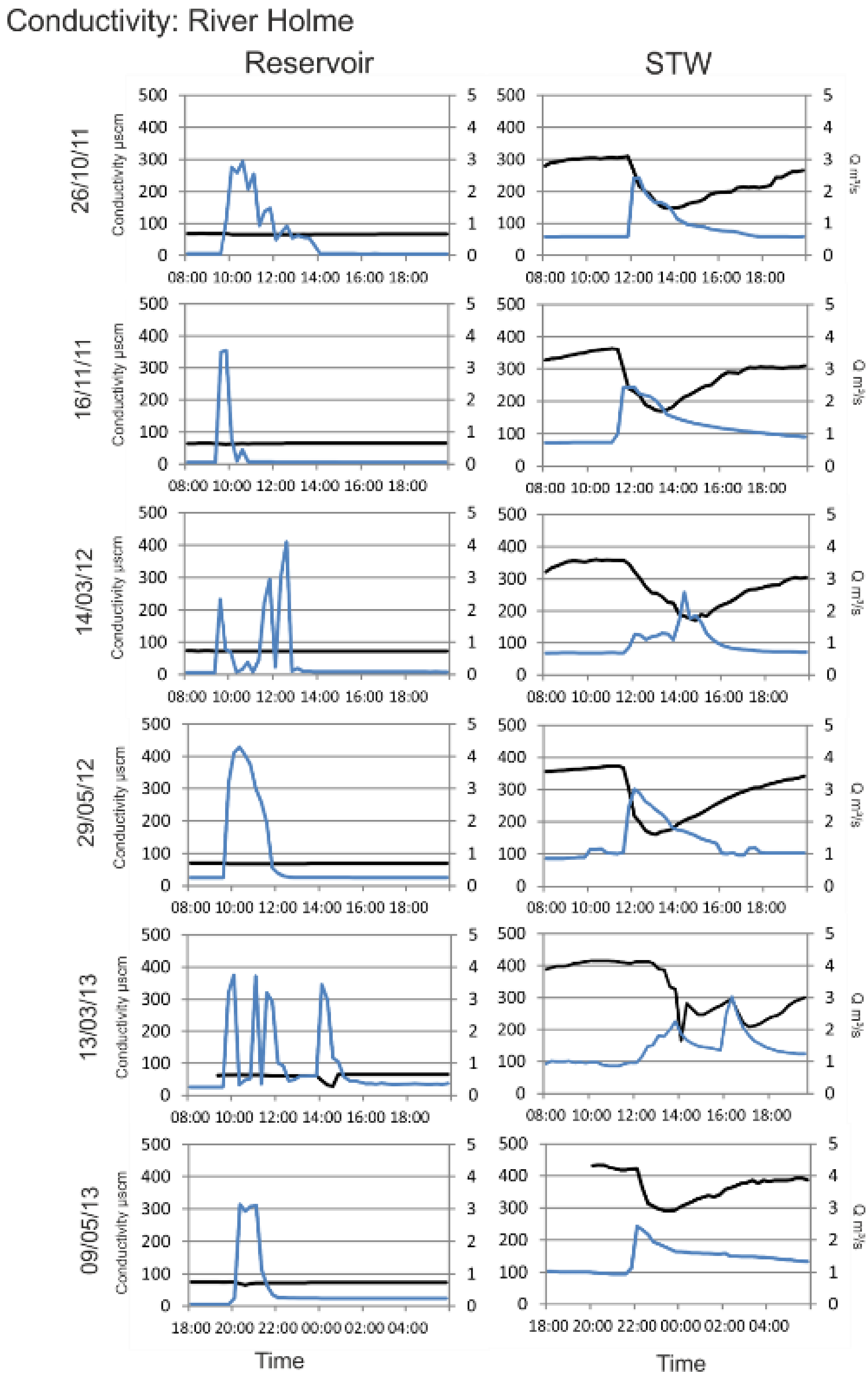


Figure 3-20. Conductivity chemographs at the reservoir and STW sampling sites. The blue represents discharge, black conductivity concentration through time.

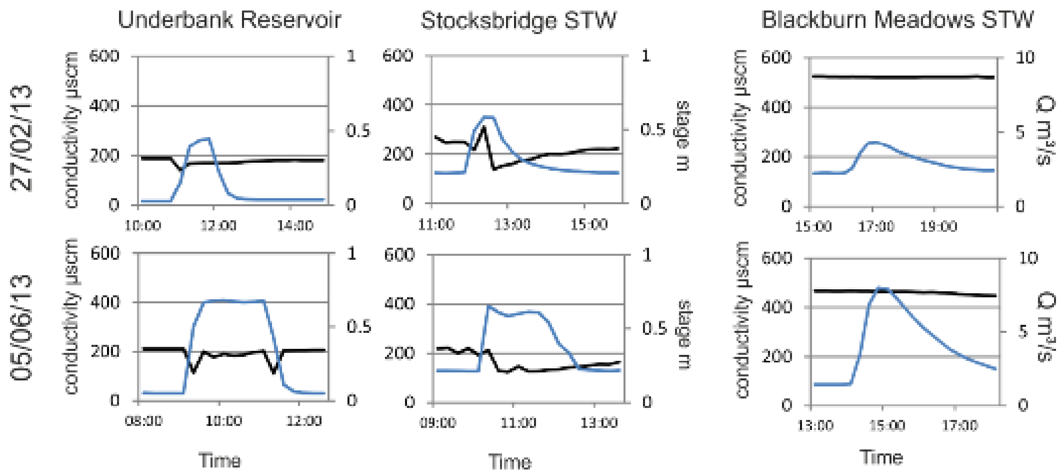


Figure 3-21. Conductivity chemographs at the River Don sites. The blue represents discharge, black conductivity concentration through time.

On the Don no appreciable decline in conductivity during wave passage at the downstream Blackburn Meadows STW was observed. Furthermore response of conductivity to flow increases is near instantaneous on the Holme (figure 3.17) but at Stocksbridge STW there is a 30 minute delay. Conductivity levels at Underbank Reservoir do fluctuate during release with declines of between 47 and 31µscm.

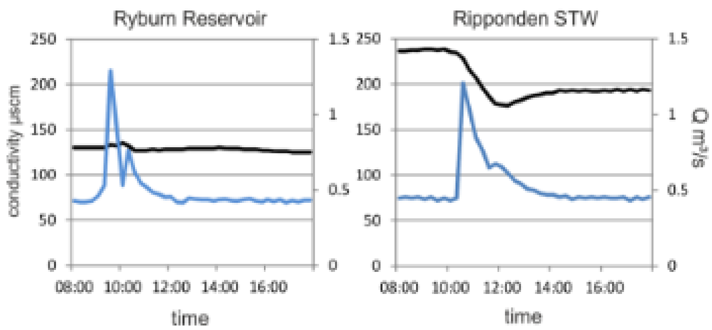


Figure 3-22. Conductivity data at River Ryburn sites. The blue represents discharge, black conductivity concentration through time.

Table 3-7. Summary of dilution achieved during wave passage at the STW on the primary Holme site.

experiment date	26/10/11	16/11/11	14/03/12	29/05/13	13/03/13	09/05/13
peak dilution µscm	148	169	170	160	164	291
percentage dilution	54%	54%	51%	58%	58%	30%
release / dilution duration	04:15/08:30	01:00/09:00	03:30/08:30	02:15/11:00	06:30/08:15	02:00/07:00
Regression R^2 between conductivity and flow, polynomial	0.78	0.93	0.63	0.76	0.65	0.93
lag time between peak flow and peak dilution in minutes	105	30	30	45	0 / 30	15

From 26/10/11 through to 13/03/13 dilution percentages are very consistent with a low of 51% and a high of 58% showing a degree of replication of results. The poor performance of the 09/05/13 experiment is highlighted here with only 30% dilution being recorded. The polynomial regression lines produced R^2 values ranging between 63 and 93%. A lag time between peak flow and peak dilution is apparent but it is variable between experiments.

Table 3-8. summaries dilution achieved during wave passage at the secondary sites. Some of the data sets for the river Don have an N/A, this is due to there being insufficient data from other days in the week to calculate a percentage decline.

Site	Ryburn	Don Stocksbridge	Don Stocksbridge	Don Blackburn Meadows	Don Blackburn Meadows
experiment date	21/03/13	27/02/13	05/06/13	27/02/13	05/06/13
peak dilution concentration µscm	176	138	124	521	463
release/ dilution duration	02:30/04:00	01:15/03:00	02:30/03:45+	00:00	00:00
Regression between conductivity and flow, polynomial	0.70	0.42	0.50	0.36	0.56
lag time between peak flow and peak dilution in minutes	60	15	0	n/a	n/a*

A figure of 71% peak dilution was calculated for the Ryburn catchment. This is a slight exaggeration of the reality. The 22 and 23rd of March were dominated by a significant pollution incident that brought conductivity levels up to a high point of 1187µscm and consequently inflated the apparent dilution achieved on the 21st as dilution is calculated by comparing the peak dilution value to those of the three days either side of the event.

3.5.2.2 NH_4

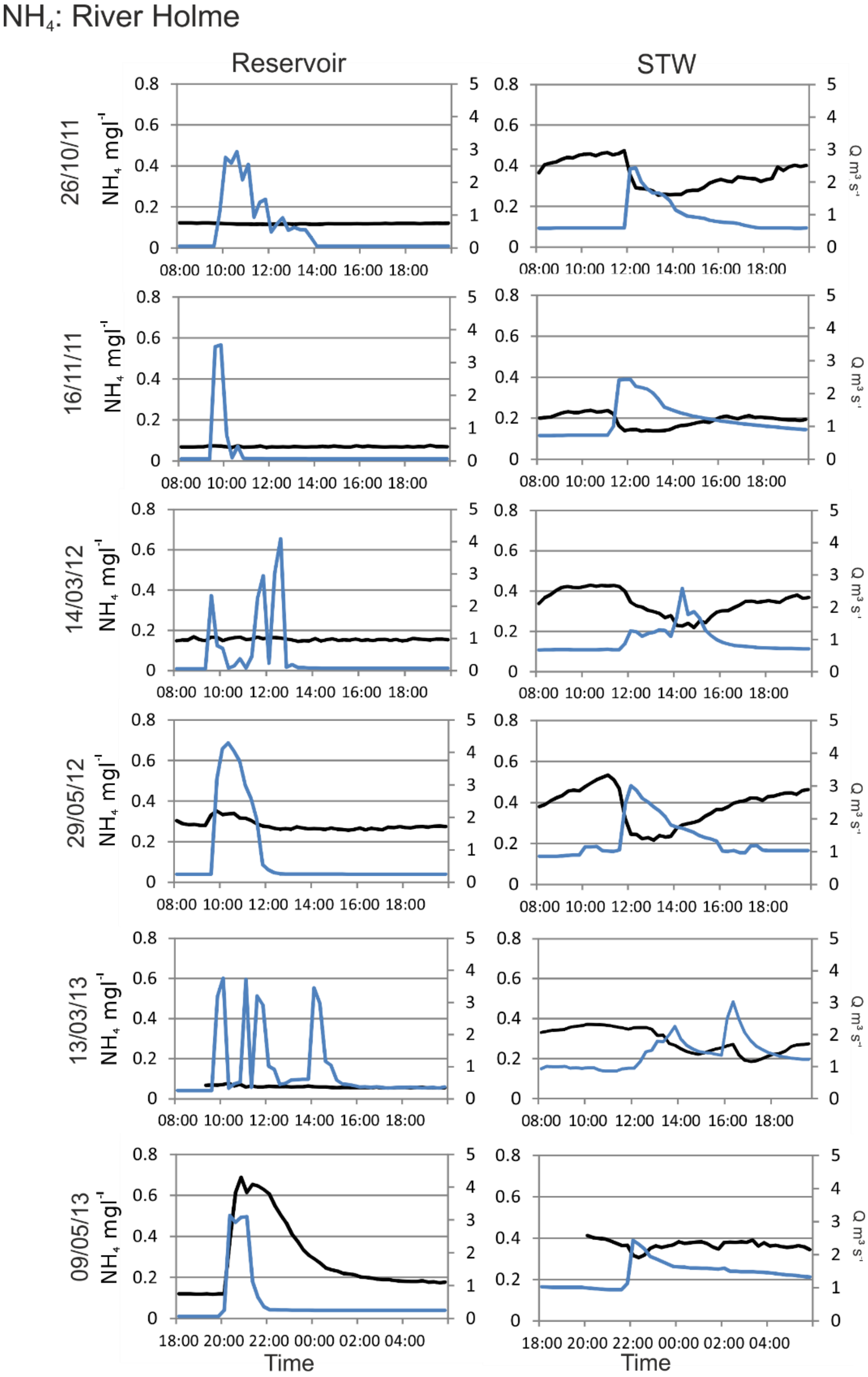


Figure 3-23. NH_4 time series graphs for the Holme primary site. Flow is described by the blue line.

In a similar manner to conductivity trends, NH_4 levels respond to the increase in flow rapidly with a variable level of dilution (Figure 3-23). NH_4 levels in the hours before wave arrival are varied with a low of 0.2mgl^{-1} and a high of 0.56 mgl^{-1}. During wave passage this is diluted down to between 0.138mgl^{-1} and 0.3mgl^{-1}. The first five experiments all show a clear relationship between flow and NH_4 dilution although the experiment on 09/05/13 is an exception with a large spike in NH_4 at the reservoir during the release. This spike from 0.12mgl^{-1} to a peak of 0.61mgl^{-1} is much larger than the spike generated during the other summer release on 29/05/12 which increases from 0.28mgl^{-1} to a peak of 0.34mgl^{-1}. The spike on 09/05/13 is followed by the poorest dilution performance recorded at the STW.

On the Don site (figure 3-24) there is considerable variation in result between the two experiments. At Stocksbridge on 27/02/13 the greatest decline in NH_4 is recorded from the whole data set, as NH_4 drops from 1.1mgl^{-1} to 0.45mgl^{-1}. During the same experiment however no response in NH_4 is recorded at Blackburn Meadows STW. During 05/06/13 a limited level of dilution is achieved at Stocksbridge but of greater interest NH_4 does decline from 0.3mgl^{-1} to 0.12mgl^{-1} at Blackburn Meadows. The lag time between the rise in discharge and the decline in NH_4 at Blackburn Meadows is exaggerated as the gauging site is located 1km upstream from the sonde deployment site, as noted in the method. Dilution is apparent at the STW site on the with concentrations falling from 0.29mgl^{-1} to a low of 0.19mgl^{-1}.

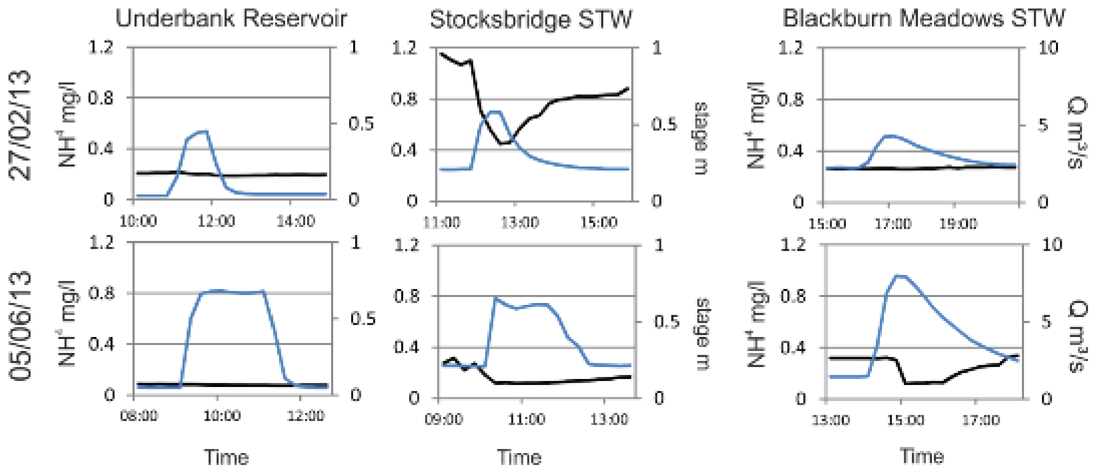


Figure 3-24. NH_4 response on the Don, blue line is discharge, black NH_4.

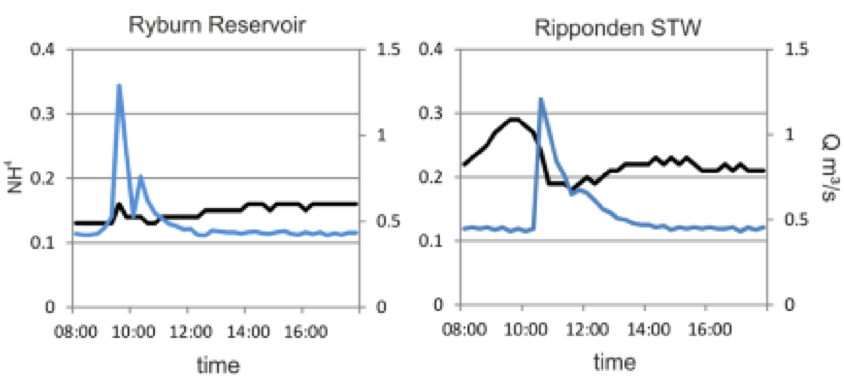


Figure 3-25. NH_4 response on the Ryburn, blue line is discharge, black NH_4.

Table 3-9. A summary of the level of dilution achieved during wave passage of NH_4 on the River Holme catchment.

experiment date	26/10/11	16/11/11	14/03/12	29/05/13	13/03/13	09/05/13
peak dilution NH_4 concentration	0.255	0.136	0.219	0.216	0.189	0.305
percentage dilution*	44%	45%	47%	59%	41%	33%
release /dilution duration	04:15/07:30	01:00/04:15	03:30/07:00	02:15/07:30	06:30/06:00	02:00/01:00
Regression R^2 between NH_4 and flow, polynomial	0.74	0.87	0.77	0.76	0.73	0.58
lag time between peak flow and peak dilution in minutes	105	120	45	45	0 / 15	60

NH_4 is diluted down to between 0.305mgl^{-1} and 0.136mgl^{-1} during releases on the Holme catchment with these figures being between 33% and 59% of the levels recorded at the same time of day through the rest of the respective weeks. 09/05/13 again is the exception with a short 1 hour dilution duration and no significant correlation between NH_4 and flow.

Table 3-10 Dilution achieved during wave passage of NH_4 on the River Don and Ryburn catchments. The percentage dilution is a comparison between peak dilution and the NH_4 concentration at the same time of day on three days either side of the experiment. This method was used to account for the diurnal fluctuations that can be seen in figure 35. A * denotes that a correlation as a p value > 0.05. The presence of n/a indicates a lack of data for statistical comparison.

Site	Ryburn	Don Stocksbridge	Don Stocksbridge	Don Blackburnmeadows	Don Blackburnmeadows
experiment date	21/03/13	27/02/13	05/06/13	27/02/13	05/06/13
peak dilution NH_4 concentration	0.19	0.447	0.119	n/a	0.118
release/dilution duration	02:30/03:00	01:15/03:00	02:30/03:45+	00:45	02:45
Regression R^2 between NH_4 and flow, Polynomial	0.80	0.77	0.66	0.50	0.63
lag time between peak flow and peak dilution in minutes	90	15	15	n/a	n/a

As with conductivity the dilution percentage at Ryburn, 62%, is in part a reflection of the high NH_4 concentrations found in river on other days of that week. R^2 values for the polynomial fitted lines range between accounting for 50% and 80% of the relationship between flow and dilution. The equations derived from each line were different. On the Don peak dilution of NH_4 at both sites on the Don during 05/0/13 is very similar at 0.119 and 0.118mgl^{-1}. Lag times at Stocksbridge during both experiments are short at only 15 minutes.

3.5.2.3 DO

DO levels were largely saturated within the data sets collected. The DO levels on the River Holme shown on figure 3-26 are greater than 10mgl^{-1}in five of the six graphs, the exception to the norm being 09/05/13, which was a night experiment. The Diurnal properties of DO can be seen in figure 35. All the other STW graphs show a slight increase in DO during wave front passage, ranging between 1.33 mgl^{-1} on 29/05/12 and 0.14mg/l on 26/10/11. Response to water release at the reservoir is mixed with an increase of 1.48mgl^{-1} during 14/03/12 and a decrease of 1.27mgl^{-1} during 13/03/13.

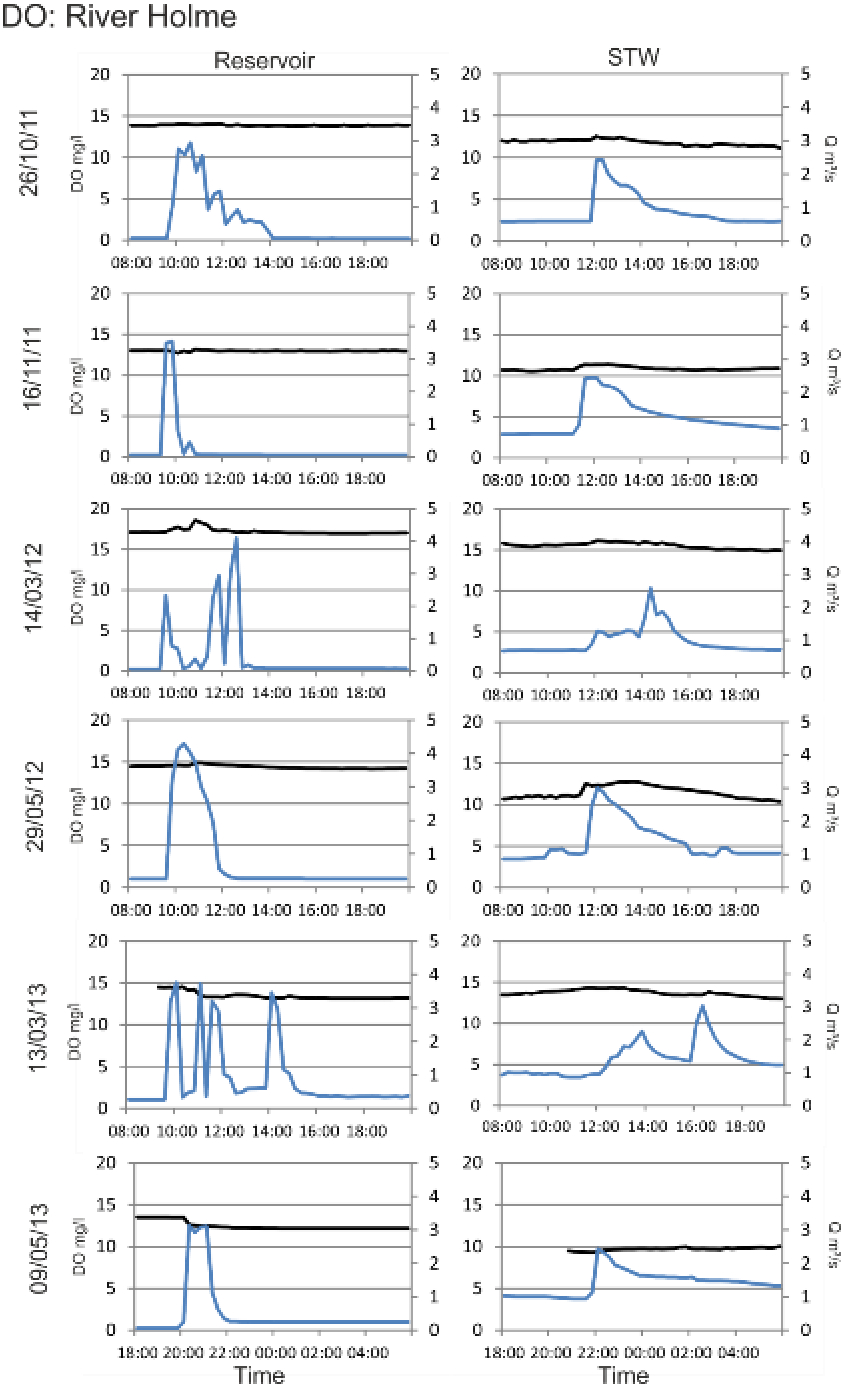


Figure 3-26. DO time series data from the primary River Holme site, blue line is discharge, black DO concentration.

On the Don concentrations are similarly high, with a notable water quality impact at Stocksbridge STW on 05/06/13. During this experiment DO levels rise from 9.1 mgl^{-1} to a peak of 11.58mgl^{-1}. Whilst the responses might be minor (0.04 -2.48 mgl^{-1} increase range) six of the above 11 STW graphs show a rise in DO with the passage of the release wave. Fluctuations in DO do occur during the release of water from the reservoir with a significant drop of 0.26mgl^{-1} at Underbank on 27/02/13 and an short increase peaking at 1.32mgl^{-1}.

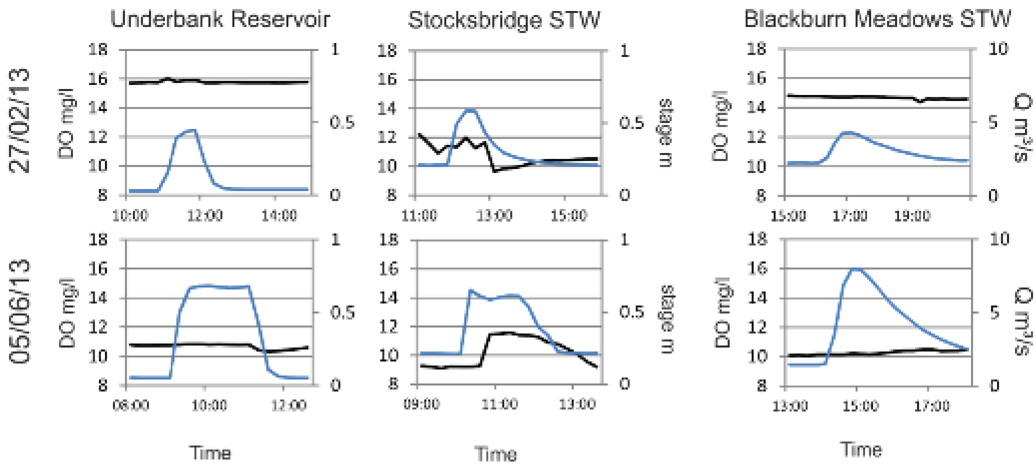


Figure 3-27. DO time series data from the secondary River Don site, blue line is discharge, black DO concentration.

Wave passage at Ryburn reservoir is followed after a 15 minute delay with a 0.87 mgl^{-1} drop in DO, downstream of the STW the wave peak coincides with a 0.13 mgl^{-1} drop in DO.

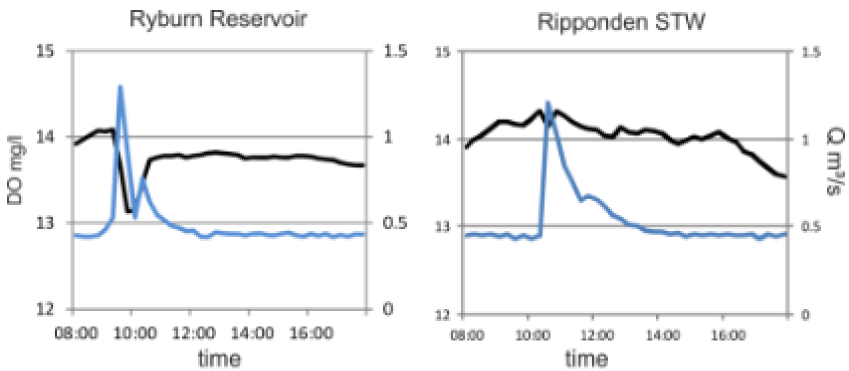


Figure 3-28. DO time series data from the secondary River Ryburn site.

3.5.2.4 pH

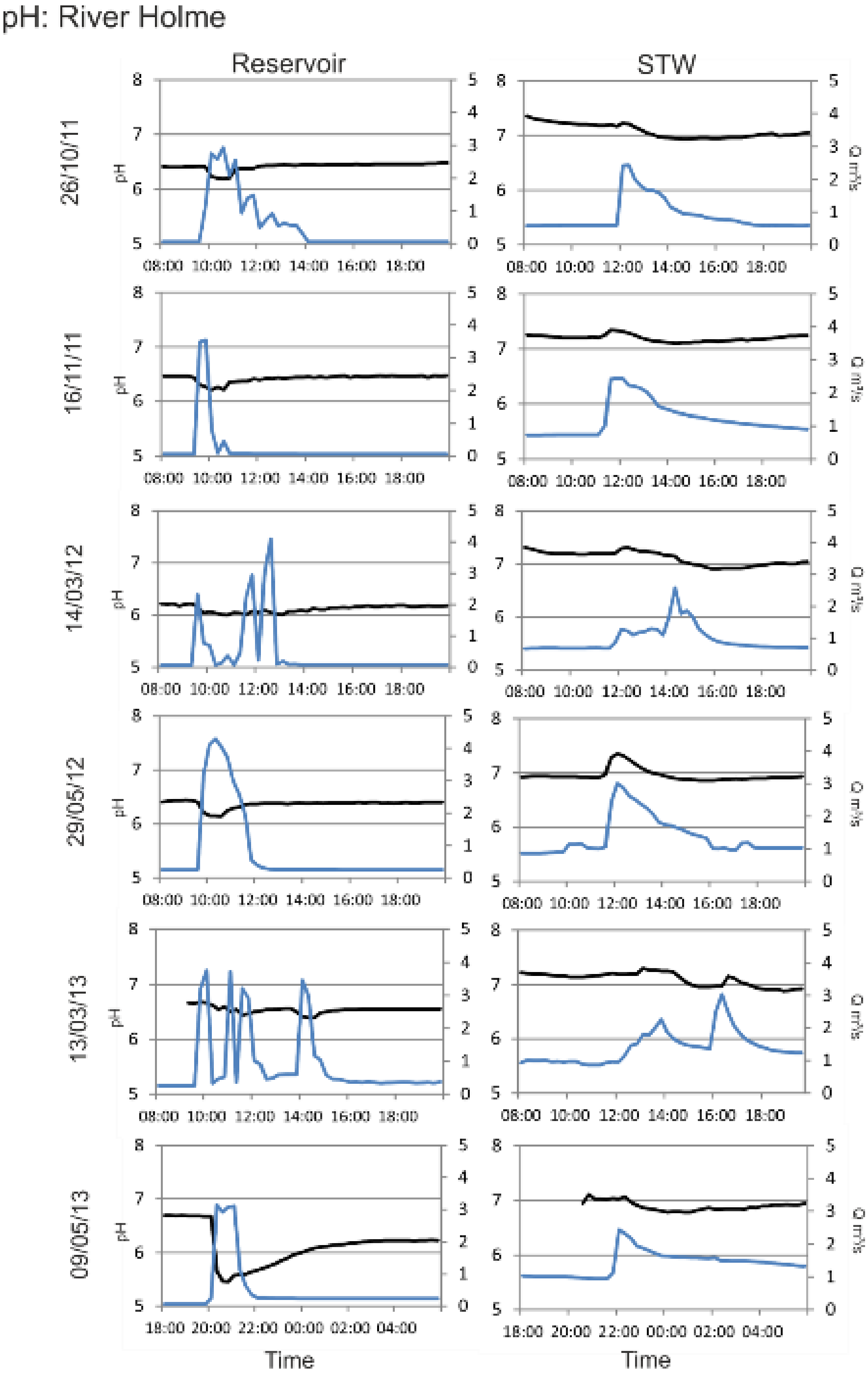


Figure 3-29. Time series data for pH and flow during wave passage at the primary River Holme site. The blue line depicts discharge and the black pH level.

pH responds differently at the STW to the reservoir. At the STW in all of the Holme experiments, the Ryburn experiment, and on 27/02/13 at Stocksbridge the wave passage is associated with an increase pH. During every recorded reservoir release the acidity of the water increases. Rises in pH vary between a high of 0.42 at the STW on the River Holme on 29/05/12 and low of 0.08 on 09/05/13. Declines in pH at the reservoir at range between 1.2 on 09/05/13 and 0.05 at Underbank on 05/06/13. STW graphs do often show a decline in pH during the falling limb of the hydrograph, this however is numerically consistent with diurnal fluxes in pH at each site. If only a single experiment had been carried out this result might be judged as erroneous, however extensive replication of results indicates a process at work. This a potential adverse effect of reservoir releases, contributing additional information for meeting aim A3.4.

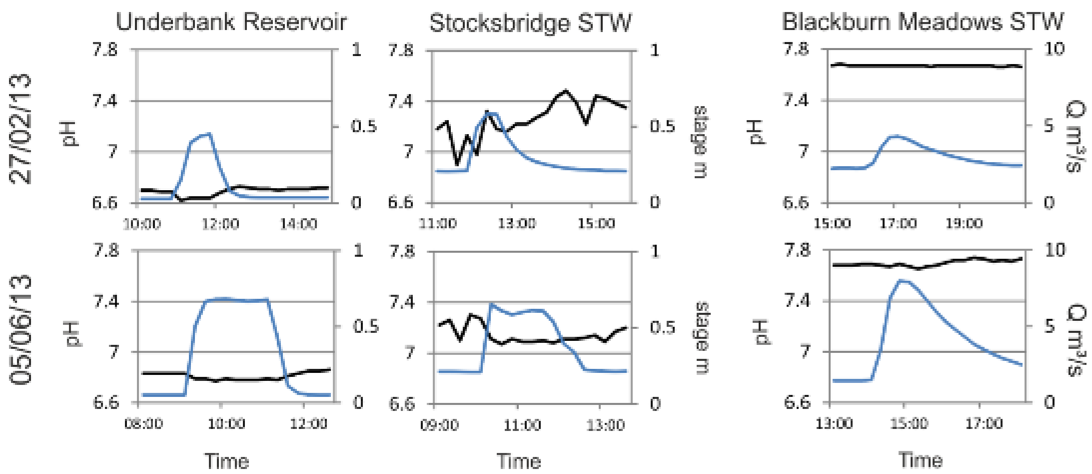


Figure 3-30. Time series data for pH and flow during wave passage at the secondary Don site. The blue line depicts discharge and the black pH level.

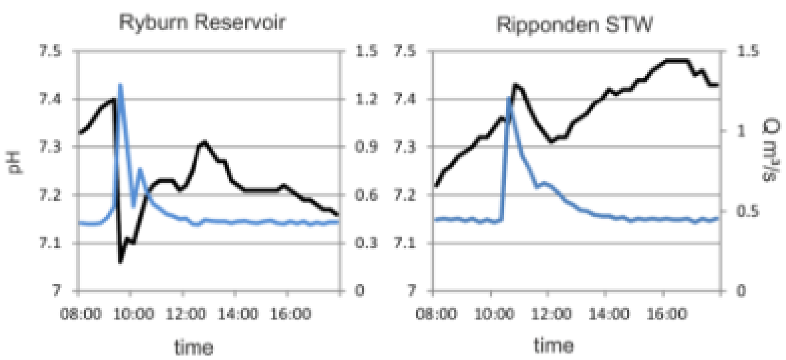


Figure 3-31. Time series data for pH and flow during wave passage at the secondary Ryburn site. The blue line depicts discharge and the black pH level.

3.5.2.5 Turbidity

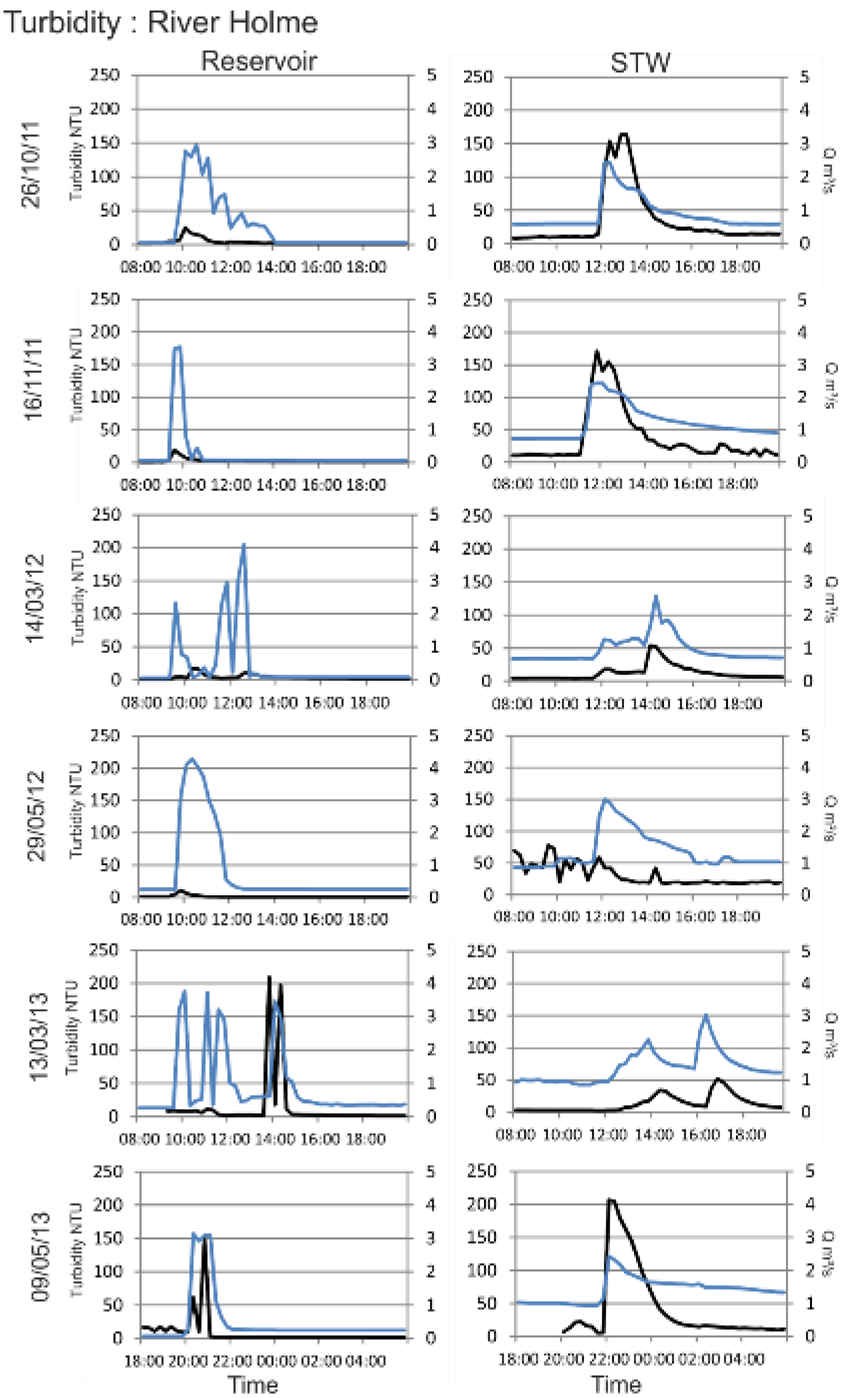


Figure 3-32. Turbidity time series graphs for the primary River Holme site during wave passage. The blue line represents discharge, the black turbidity.

Release of a wave from the reservoir, and arrival at an STW site are associated with a spike in turbidity in 19 of the 20 graphs shown in figures 3.32-34 below . At the high end substantial fluxes of turbidity are seen of 163.4, 171.4, 206.3, and 249.3NTU at STW sites with numerous other spikes in the 30-60NTU range. At the reservoirs peaks are generally lower with only Underbank on 05/06/13 and Digley on 13/03/13 and 09/05/13 displaying peaks over 100NTU. A general trend of wave waters increasing in turbidity as they move down stream can be seen. Lag times between the hydrograph peak and turbidity peak are non-existent during the majority of releases with only Stocksbridge on 27/02/13 and Digley on 13/03/13 showing lags of 15 minutes and 45 minutes respectively.

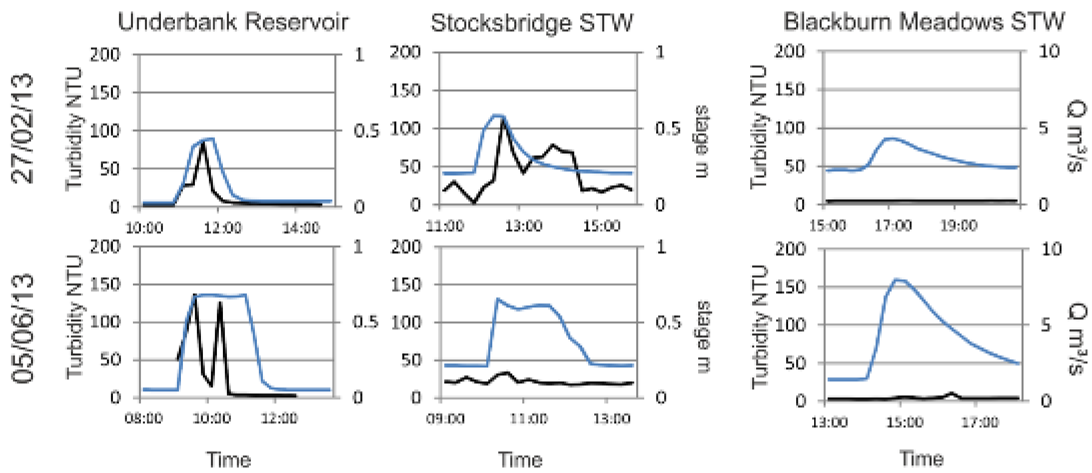


Figure 3-33. Turbidity time series graphs for the secondary Don site during wave passage. The blue line represents discharge, the black turbidity.

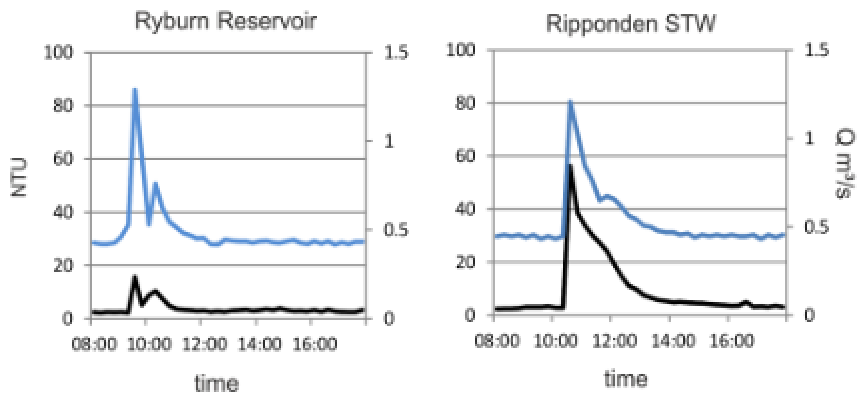


Figure 3-34. Turbidity time series graphs for the secondary Ryburn site during wave passage. The blue line represents discharge, the black turbidity.

3.5.2.6 Temperature

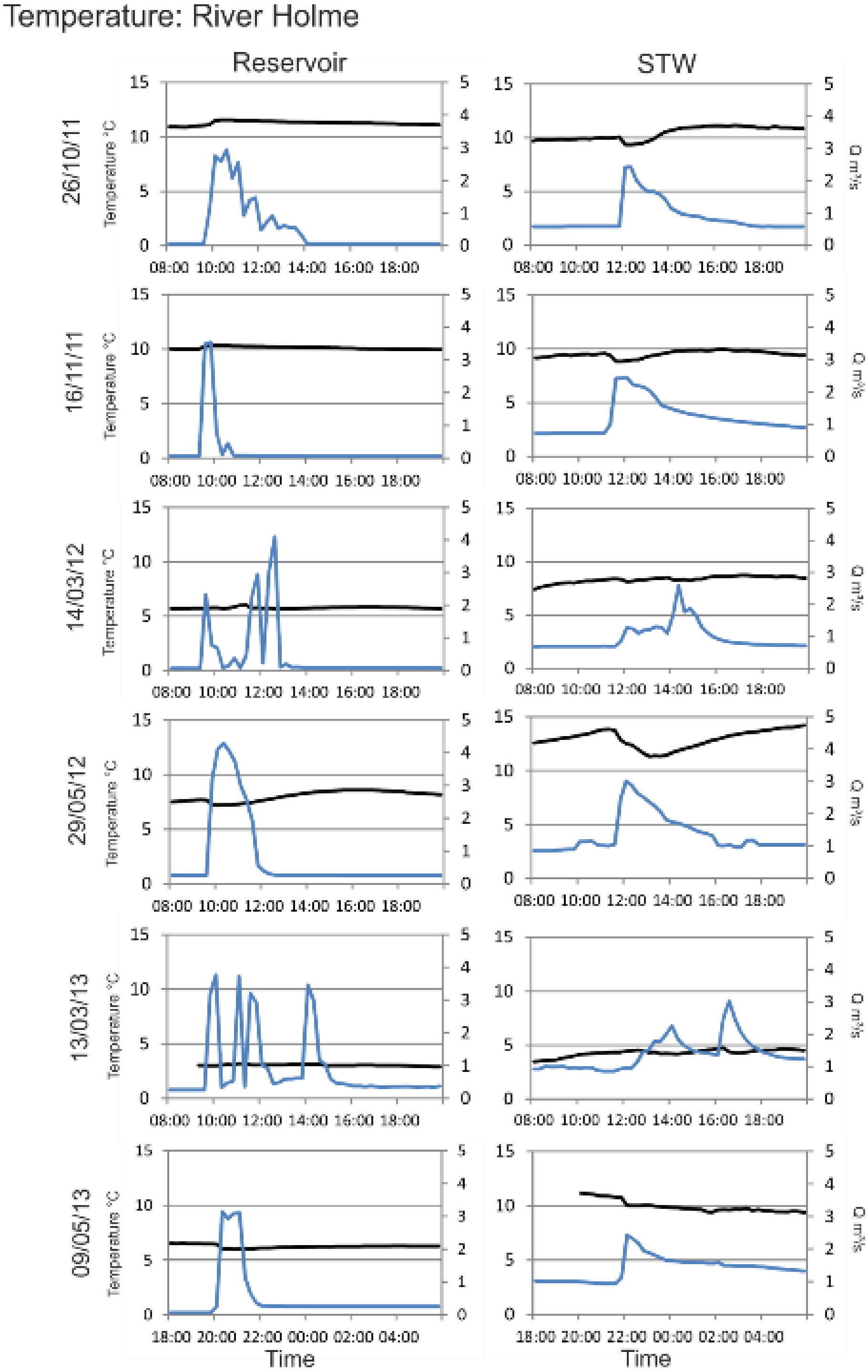


Figure 3-35. Temperature time series graphs for the primary Holme Field site. The blue line represents discharge, the black temperature.

The temperature changes associated with the release of water from the reservoir are seasonal. During winter, autumn and early spring months, temperatures rise with the release of water from the reservoir. The experiments of 26/10/11, 16/11/11, 14/03/12, 13/03/13 on the Holme at the reservoir (Figure 3.35), and 21/03/13 at Ryburn (figure 3.37) all feature a temperature rise, to a maximum of 0.6ºC. During the two summer releases on the River Holme there is a maximum decline of 0.3ºC (figure 3.35). With the exception of 26/11/11 temperatures at the reservoir are always lower than those at the STW sites. Net temperature declines in each experiment on the Holme STW, and Ripponden STW with a maximum cooling effect of 2.5ºC on 29/05/13. The River Don data is atypical (figure 33). Underbank reservoir temperatures slightly decline in winter (down 0.32ºC) and increase in summer (up 0.49ºC). The data from both Stocksbridge graphs shows a spike in temperature (1.9ºC and 0.5ºC) with wave arrival followed by a decline. Temperature does not respond to wave passage at Blackburn Meadows. The differences between the Don site and the other two again emphasizes the value in having multiple sites.

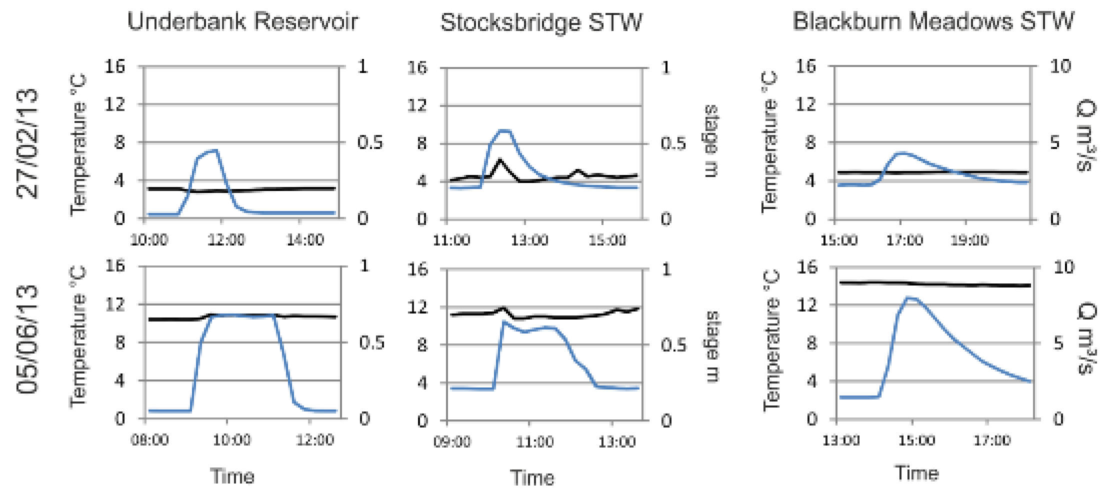


Figure 3-36. Temperature time series graphs for the Don site. The blue line represents discharge, the black temperature.

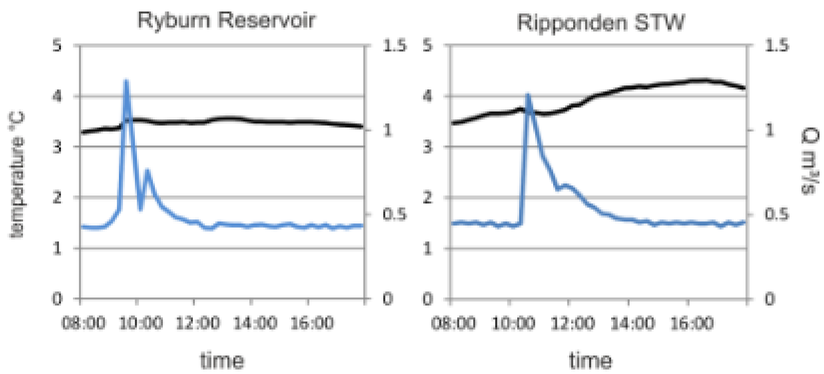


Figure 3-37. Temperature time series graphs for the Ryburn site. The blue line represents discharge, the black temperature.

3.5.2.7 Long Term 24 Month Data Set for the Holme Primary Catchment

The function of tables 3-11 and 3-12 is to provide an over view of the water quality conditions in the river Holme outside of the release events. Consequently the interdecile range has been used to give an upper and lower bounding to the most common (80% of samples) conditions for a given water quality parameter. Standards have been employed to give an estimation of how often the river could be considered polluted under a simple one dimensional metric.

Table 3-1. STW 24 month data set; the annual mean, interdecile range, seasonal means, and percentage of values that fail to meet a specified standard. Standards were taken from either the EU 2006/44/EC Fisheries Directive, or 75/440/EC Water Abstraction Directive. Where two standards are given, one is for salmonids and the other for cyprinids.

	NH_4 (mgl^{-1})	Conductivity (µscm)	DO (mgl^{-1})	pH	Temperature (ºC)	Turbidity (NTU)
2 year mean	0.31	315.30	12.60	7.06	9.56	35.30
Decile 10%	0.14	200	9.52	6.95	4.82	3.5
Decile 90%	0.47	421	16.99	7.28	14.25	74
summer mean	0.34	324.55	10.73	7.11	12.83	42.98
winter mean	0.24	281.49	14.31	6.96	7.21	31.26
Standard	$1mgl^{-1}$ / $0.2mgl^{-1}$	1000	$9mgl^{-1}$ / $7mgl^{-1}$	6-9	21.5	n/a
EU Directive	2006/44/EC	75/440/EEC	2006/44/EC			n/a
% below standard	77 / 0.3	0	5 / 0.2	0	0	n/a

At the STW water quality meets the standards as defined in EU 2006/44/EC and EU 75/550/EEC for the majority of samples for all the parameters excepting NH_4. There are two standards given for NH_4, an advisory of $0.2mgl^{-1}$ and a standard of $1mgl^{-1}$. 0.3% of samples failed the standard, but 77% of samples were greater than the advisory with the mean for the data set being 0.31 mgl^{-1}. 90% of samples were below 0.47 and only 10% were below $0.14mgl^{-1}$ showing that there is an appreciable concentration of NH_4 in the river 80% of the time. Each of the 6experiments on the Holme occurs in antecedent conditions with concentrations of over $0.2mgl^{-1}$ with 5 over $0.4mgl^{-1}$. The pre-experiment conditions can therefore be considered representative. Conductivity never fails the 1000µscm advisory with a mean of 315.3 and an interdecile range of 221. DO is saturated and only fails its standards of 9 and $7mgl^{-1}$ 2.2% and 1.1% of samples. The lower 10% decile is $11.02mgl^{-1}$ so 90% of samples are more saturated than this. pH is close to neutral with a mean of 7.06, temperature has a mean of 9.56ºC and an interdecile range of 9.43 showing that it is highly variable. Turbidity has a mean of 35.30NTU giving the river a cloudy brown colour. Across NH_4, conductivity, DO and turbidity the river is cleaner in winter than summer, with winter being a mean $0.1mgl^{-1}$ NH_4, 43µscm, 3.58 mgl^{-1} DO, and 11.72 NTU lower.

Table 3-12. Reservoir 24 month data set; the annual mean, interdecile range, seasonal means, and percentage of values that fail to meet a specified standard. Standards were taken from either the EU 2006/44/EC Fisheries Directive, or 75/440/EC Water Abstraction Directive. Where two standards are given, one is for salmonids and the other for cyprinids. Both salmonid and cyprinid fish are found within the Holme (Tosney 2012).

	NH_4 (mgl^{-1})	Conductivity (µscm)	DO (mgl^{-1})	pH	Temperature (ºC)	Turbidity (NTU)
2 year mean	0.09	67.54	13.39	6.48	7.45	4.01
Decile 10%	0.03	56	11.02	6.3	3.58	0.9
Decile 90%	0.158	74	15.86	6.7	11.2	4.6
summer mean	0.11	67.87	12.45	6.45	9.24	4.13
winter mean	0.08	67.4	14.47	6.33	5.73	1.55
Standard	1/ 0.2	1000	9 / 7	6-9	21.5	n/a
EU Directive	2006/44/EC	75/440/EEC	2006/44/EC			n/a
% below standard	0 / 4	0	2.2 / 1.1	1	0	n/a

Water quality parameter numbers are lower at the reservoir than the STW with the exception of DO. For NH_4, the mean of 0.09mgl^{-1} is 0.22 lower than that at the STW, and the 90% decile of 0.158 is only fractionally higher than the 10% decile at the STW. NH_4 at the reservoir does exceed the advisory standard of 0.2mgl^{-1} in 4% of samples, the majority of these occurring in the summer months. Conductivity is very homogenous with a mean of 67.54µscm, summer and winter means only >0.4 outside of this and an interdecile range of 18. DO is higher across most measures compared with the STW. With only 10% values being under 11.02mg/l the water can be considered saturated. pH is generally lower at the reservoir than the STW, with a difference in means of 0.58, temperatures are also cooler by a mean difference of 2.11ºC. Temperature is less variable at the reservoir both seasonally, with a mean 3.51ºC difference and by interdecile range with 80% samples being stretched over a 7.62ºC range. Turbidity is comparatively low with a upper interdecile of 4.6NTU.

If the river is considered as a system, the reservoir is one input, the water downstream of the STW is the output. The STW itself is another input, and the water between the reservoir and the STW is a through put. Table 12 below summarises water quality data for the STW input and the through put of water 10m upstream of the STW. The difference between the results in table 12 below, and table 10 above is broadly the polluting influence of the STW on the river, the subject of the dilution experiment at the primary site.

Table 3-13 gives mean results for NH_4, conductivity and DO at two sites on the Holme River, one being the STW outflow and the other being 10 upstream of this. This data is based on 50 hand samples collected on release days prior to wave release with the exception of the NH_4 data which was provided by YWS and comprises of 62 samples spread between 2010 and 2012.

Water Quality Parameter	NH_4 (mg l^{-1})	Conductivity (µscm)	DO (mg l^{-1})
STW outflow	0.4	608	6.6
10m Upstream of STW	unmeasured	182	11

The NH_4 concentration of 0.4mgl^{-1} is diluted by 0.09 as it enters the water course and moves 200m further down river down to a mean of 0.31 in table 3-11 . The mean conductivity of 608µscm raises the 182µscm that comes down river to a mean of 315.3µscm over the 200m stretch. DO increases from the mean 11mgl^{-1} in table 3-13 to the mean of 12.6mgl^{-1} in table 3-11.

The dilution statistics reported in tables 3-8 and 3-9 above were compared with a seven day mean for a reason. Any water quality dilution achieved during a release experiment must be compared to a longer data set than simply the values measured immediately prior to the event as the output from the STW of a given ion could vary during the release. This is because water quality parameters such as NH_4 (Crumpton and Hersh 1987; Scholefield *et al.* 2005; Xia *et al.* 2013) temperature (Neal *et al.* 2006), conductivity (Bourg and Bertin 1996; Vogt *et al.* 2010), Bourg et al., 1996), DO (Guasch *et al.* 1998; Laursen and Seitzinger 2004), and pH (Neal *et al.* 2006) have diurnal cycles. Evidence for these cycles is provided in figure 37 below. This data is a necessary control for the water quality data presented in this chapter. This data is also important for informing the question on diurnal variation under aim A3.4.

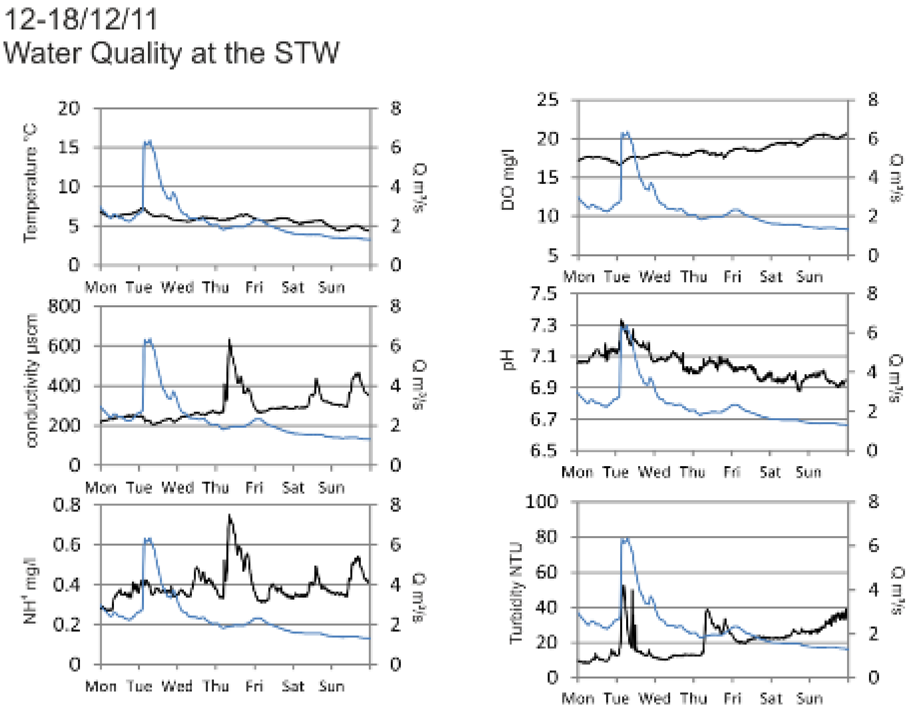


Figure 3-38. A week of data from the Sonde for 6 water quality parameters <200m below the STW. Discharge is represented by a blue line, water quality parameters the black.

Temperature, conductivity, NH_4, DO, and pH data all show evidence of diurnal cycles. Some of these cycles such as temperature, DO and pH are strictly diurnal, in that they are driven by the solar cycle (Xia *et al.* 2013). Conductivity and NH_4 are far more erratic in their cycles because they are tied to diurnal processes of the STW output (Butler *et al.* 1995; Almeida *et al.* 1999). STW tend to have two peaks per day, a few hours delay from people preparing to leave work, and returning home from work during week days, as these are the times that the water closet is used and other waste outputs (Friedler *et al.* 1996). Spikes in NH_4 occur daily with at least two peaks during in each day. If a release wave were to arrival downstream of the STW at around 11am any further rise in NH_4 through the day would not be accounted for on the time series graphs shown in figure 20. During Wednesday 15/12/11 a large spike in both NH_4 (a peak of 0.728 mg/l) and conductivity (615 µscm), occurs. The duration of this spike is 26 hours.

A natural flood event occurs during this week of data, which peaks at $6.1m^3s^{-1}$ and lasts 28 hours. During this precipitation induced event there is a conductivity decline of up to 22µscm to a low of 221µscm for the duration of the event. When compared with the results shown in table 6 it can be seen that the reservoir releases bring conductivity down to a lower level of <170µscm with one exception. This result is relevant to the comparison between natural flood events and reservoir release events included under A3.4.

Under A3.4 of this chapter is a question concerning the adverse impact of release waves on water quality. Figure 3.39 below details 24month rainfall and discharge data for the Queens Bridge site at the bottom of the catchment. The scale of the reservoir release hydrographs can be placed into perspective against natural flood events generated over the two year period.

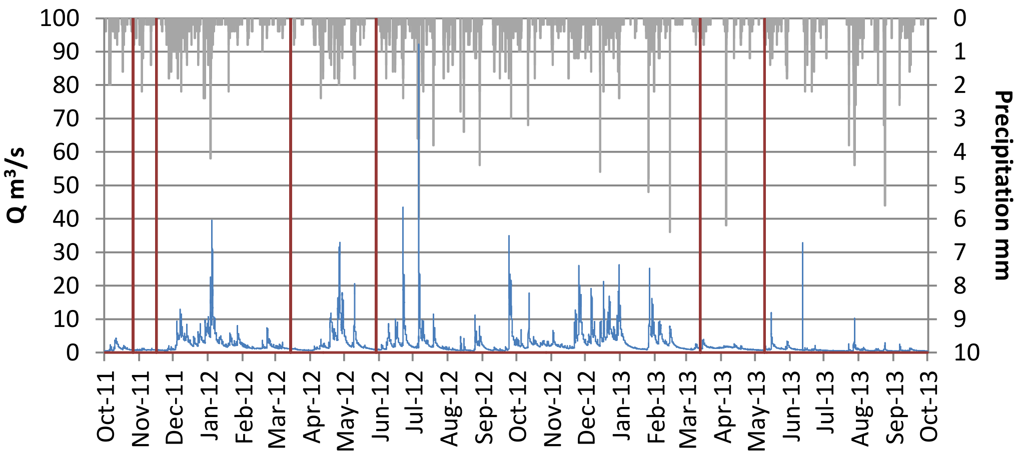


Figure 3-39. 24 month rainfall (Ludhill farm EA tipping bucket gauge) in grey, discharge at Queens Bridge EA gauge (Huddersfield) in blue. The red lines locate the reservoir release experiments. Locations of these gauges can be seen in figure 10.

The six reservoir release experiments shown by the red lines on figure 38 above had peak discharges no greater than $3.5m^3s^{-1}$ and durations no longer than 7 hours. By comparison the highest magnitude event recorded in the two year period peaked at a flow of $92.2m^3s^{-1}$, the river took in excess of two weeks to return to pre event levels. There were 12 flood events that peaked at over $20m^3s^{-1}$ and 27 that exceeded $10m^3s^{-1}$. The hydrological significance of the six reservoir releases can therefore be considered minor. This is relevant when considering the adverse impact of releases upon the river system which is considered under aim A3.4 of this chapter.

3.5.3 Mixing in the Water Column

Aim A3.3 for this chapter was to examine mixing process through the water column. Conductivity data recorded from three depths (top, middle, bottom) in the water column at high temporal resolution during wave arrival is presented below. The top level conductivity probe responds to the arrival of the wave front more rapidly in four of six panels in figure 41 below, the exceptions being 13/03/13 and 14/03/12, the two scour tests on the River Holme. For instance on 16/11/11 the wave arrives at 11.15, conductivity at the top probe drops from 362µscm to 291µscm by 71µscm or 19%, whereas over the same period the middle probe has only declined 17% and the bottom 2%. The other clear example being the 09/05/13 where in conductivity response is delayed with the wave front arriving at 21.45 but the major drop in conductivity happening at 22.08 at the top probe and 22.09 at the middle. By 22.13 the top probe has decreased 22µscm which amounts to a 6%. The middle probe drops 26µscm or 7% over the same time. The pattern in the two Don experiments at Stocksbridge is comparable but with lower percentages. There is a pattern but it involves relatively small changes in conductivity over a few minutes.

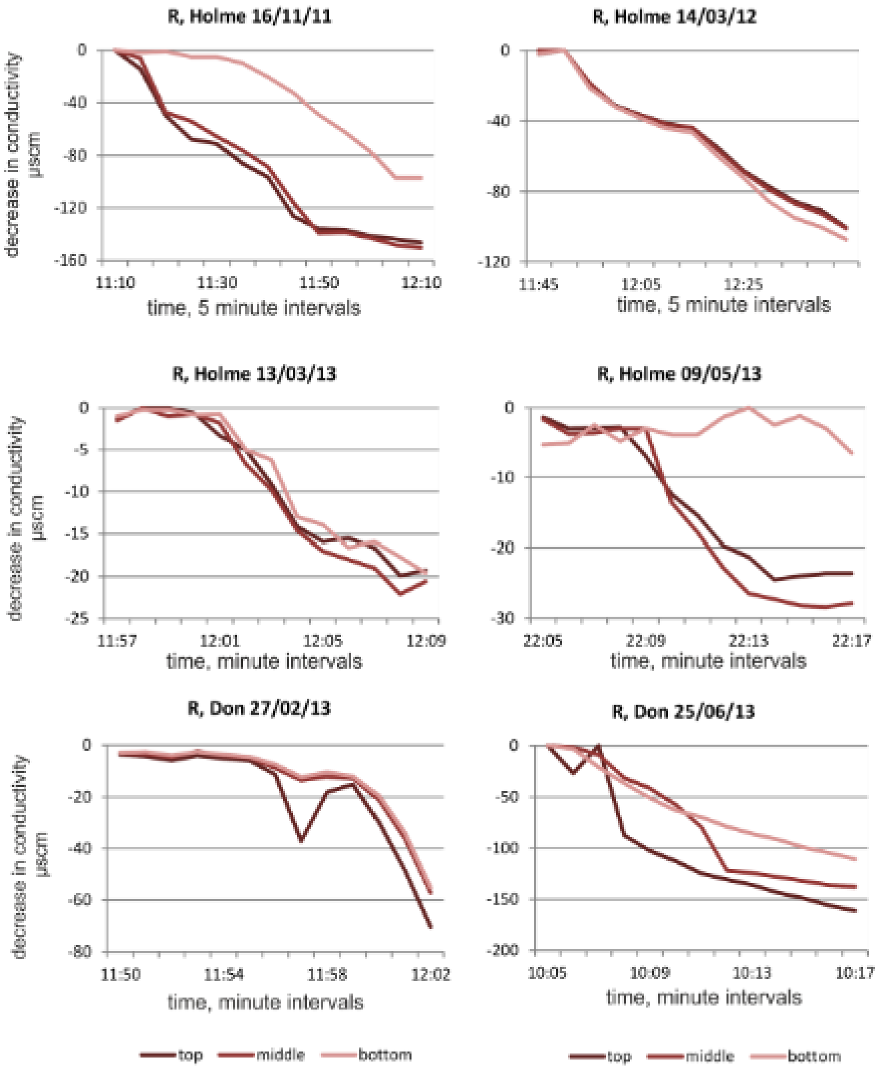


Figure 3-40. A graphical display of conductivity data from three depths in the water column labelled top middle and bottom. The values presented here have been zeroed and shown as a decline rather than actual in river conductivity values. This has been done to communicate the relative declines in conductivity at different heights in the water column, rather than simply reiterate conductivity trends already described in this results section. The time frames displayed start less than <10 minutes before the arrival of the wave front and continue until an appreciable drop has been shown.

3.6 Discussion

The discussion section of this chapter follows the same format as the methods, and results sections with the argument being ordered by the four aims set out in the aims and hypotheses section.

3.6.1 Can a Release Wave Catch a Pollution Incident

The current project has shown that wave velocity is greater than the baseflow velocities that a pollutant would travel at under non flood conditions. Therefore in answer to key question Q1 from chapter 1, and aim A3.1 of this chapter, in the River Holme a wave could catch up with a pollution slug if it wave was released within 6 hours and 30 minutes of the pollutant being released from the STW. The following paragraphs will explain and defend these statements.

If an incident were to occur at the STW, or another point source, and be detected, it could be diluted by an increase in discharge in the river if that increase in discharge can move down the river faster than the pollutant. In the experiments conducted in this chapter a reservoir was selected as the source of the increased flow, and an STW was selected as the source of pollution. In the case of the dye experiment a simulated pollutant was released from this site. On the primary River Holme site a further location, Queens Bridge, at the bottom of the catchment was selected as a compliance point. That is a position at which it is determined whether or not the wave would have caught a pollution slug.

The wave velocities calculated for the River Holme are at slowest $0.9ms^{-1}$ and at fastest $1.81ms^{-1}$ taking between 3 hours and 3 hours 45 minutes to cover the 14.7km reach of the catchment. By comparison flow velocities recorded in the river at four different sites in baseflow conditions had a mean of $0.36ms^{-1}$. The majority of the river is typified by the slower pool, and channel topographies which have mean flow velocities of $0.18ms^{-1}$ and $0.19ms^{-1}$. These velocities are substantially lower than those of the wave. Therefore the wave travelling at or in excess of $1ms^{-1}$ will catch a slug of pollution travelling around $0.2ms^{-1}$. This finding coincides well with the literature that gives measurements for wave celerity velocities in comparison to flow velocities (Bull 1997, Einstein 1943).

With the primary Holme site having a range of $0.54ms^{-1}$ between the slowest and fastest wave and the secondary sites having a velocity as slow as $0.86ms^{-1}$ on the Ryburn and as fast as $1.63ms^{-1}$ on the Don the question has to be posed, what controls wave velocity? If wave velocity can be influenced river managers can exert some control over their response time to pollution incidents.

According to Gilvear (1989), working in a small 2.2km long river reach, baseflow velocity, release magnitude and Mannings N are all factors found to have a direct relationship with wave velocity. Mannings N reduces with stage increase, and consequently flow as edge effects are proportional to flow depth. Anecdotal observations made during each experiment did not note any major changes in river morphology, and thus Mannings N between visits. Equally the Ryburn, Holme and Upper Don rivers are all upland gravel bed rivers with a riffle pool system, and therefore their Mannings N should be comparable. Jarrett (1984) suggests that flow depth is the main determinant in flow resistance in upland streams. It is therefore unsurprising that the wave celerity on the Ryburn, the river with lowest discharge and stage, are the slowest and the Don the highest.

Flow velocity is proportional to discharge, which in turn is proportional to stage. A higher stage and consequently discharge produces a higher velocity (Govers 1992). Prior to the arrival of the wave front at the Blackburn Meadows gauge on 27/02/13 discharge was 2.26m^3s^{-1}, on 05/06/13 discharge was 1.44m^3s^{-1} however the wave on 05/06/13 was 1 hour 45mins faster. This can be accounted for by the large difference in wave magnitude rather than the pre-release baseflow discharge. Peak flow at Blackburn Meadows on 05/06/13 was 3.72m^3s^{-1} greater or 47% higher than that of 27/02/13. In management terms a greater release of water from the reservoir can be used to buy time. At the primary Holme site waves take between 3 hours (14/03/13, 29/05/13) and 3 hours 45 minutes (09/05/13) to cover the 14.7km river reach. The two fastest flows, 14/03/13 and 29/05/13 are the two with a peak discharge at the reservoir site above 4m^3s^{-1}, whilst this is only marginally higher than the peak discharges of the other experiment waves, all of them being greater than 3m^3s^{-1}, magnitude does best account for the difference in wave velocity.

The dye experiment data provides further empirical evidence that the wave velocity is not only higher than the base flow velocities but is also capable of catching a simulated pollution event. Two key pieces of evidence can be noted. First when a wave was released from Digley reservoir and a dye slug from Neiley STW, the wave reaches Queens Bridge at the bottom before the dye was detected. The wave front arrived 2 hours 45 minutes before dye detection. Either the wave had over taken the dye, or the dye concentration in the wave front had been diluted sufficiently for it to be undetectable. This result is a proof of concept for the mitigation system being proposed in this study. Second, during the control test, dye took 9 hours 45 minutes to travel the 6.5km between the STW and Queens Bridge, giving the fastest moving dye a velocity of 0.185ms^{-1}. This suggests that the slower velocities recorded in figure 3.18 are dominant in the low flow conditions of the control dye test. Given that the wave released from the reservoir reached Queens Bridge within 3 hours 15 minutes on 09/05/13 it can be estimated that a response window of 6 hours 30 minutes exists to dilute a pollution incident. Wave travel times in the Holme experiments vary between 3 hours and 3 hours 45 minutes, additionally the dye control test took place during end of summer low flow conditions. The response time estimate should be viewed I light of these facts. The only other account of a dye experiment in conjunction with a reservoir release in the surveyed literature was reported by Malatre and Gosse (1995) and Barriller *et al.* (1993). In this case the wave front was reported to outpace the dye by 36 hours over a 64km reach, whilst the scale of experiment is different the results complement each other.

3.6.2 Can a Release Wave Dilute Pollution?

3.6.2.1 Conductivity dilution

Once the time frame for the wave's to catch up with a pollution slug has been demonstrated it is logical to shift topic and examine the evidence for dilution itself. Key Question 2 of this chapter and the study as a whole, is how much dilution can be achieved? The time series data presented in this chapter shows the dilution of both conductivity and NH_4 with these two water quality parameters being considered representative of pollution. Conductivity declines during reservoir release generated wave passage have been recorded by Malatre and Gosse (1995), Petts *et al.* (1985), Kurtenbach *et al.,* (2006) Flouger and Petts (1984) and Krein and De Sutter (2001) showing good agreement with the results from the river Holme and Ryburn. In each of these papers peak dilution of between 20% to 48% of pre-release levels is reported making the magnitude of dilution slightly lower than all but one of the results reported in table 3-7 for the River Holme. The data sets collected from the Don however differ, with no conductivity dilution apparent at Blackburn Meadows STW. There is a lack of pared catchment studies in the literature and therefore a published comparison for this result cannot be given. Under the equation 2.1 an increase in total volume (Q) would be expected to produce a decrease in concentration. The only reasonable exception to this would be if the diluting water had the same or higher concentration of conductivity than the polluted water. It is therefore likely that by the time the wave reaches Blackburn Meadows, it has either sufficiently mixed with urban drainage water, or other river water, to raise its conductivity to that of the water downstream of the STW outflow, or the kinematic nature of the wave has resulted in the lower conductivity water being left behind by the wave front (Glover and Johnson, 1974). Malatre and Gosse (1995) working in a similarly long catchment, a reach of 65km, describe a delay between the arrival of the wave front and reservoir derived water arrival. The authors record a drop in conductivity several days after the arrival of the wave front. It is therefore very possible that the reservoir derived water had not arrived during the sampling period and its effect is not recorded.

Table 3-14. Dilution of conductivity during release wave passage in the literature.

Paper	Release Magnitude m^3s^{-1}	Release duration	distance to sampling point (km)	peak conductivity dilution	dilution duration
Malatre and Gosse, 1995	26	10 days	25	310 diluted down from 600	8 days +
Petts et al., 1985	50	5 hours +	10.05	62 diluted down from 90	4 hours+
Kurtenbach et al., 2006	1	1 hour	10~	100 diluted down from 220	6+ hours
Krein and De Sutter, 2001	0.3	20 minutes	3	190 down from 235	1 hour
Flouger and Petts, 1984	12.62	not stated	6~	25 down from 45	2 hours +

There is some evidence from the literature that dilution duration is longer than the hydrograph that generated it. This is not clear in table 3-14 above as many authors do not, either in a statement or graphically, precisely show both of these measures of time, however Krein and De Sutter (2001) explicitly state this result. In the results shown in this chapter flow returns to pre-event state faster than conductivity. Both Kurtenbach et al., (2006) and Krein and De Sutter, (2001) are primarily concerned the lag time response between the rise in flow and conductivity response. Whilst there is a delay between peak flow and peak conductivity dilution, no such delay in initial response was recorded in this study at either Holme or Ryburn sites. This is unlikely to be a result of the 15 minute sampling regime as both references indicate lags of greater than 15 minutes in their respective results. Rather the results in this study are concerned with dilution of a point source rather than diffuse and therefore response to changes in flow is more instantaneous.

3.6.2.2 NH_4 dilution

Dilution of NH_4 concentration to varying degrees was witnessed in all the time series graphs with the exception of the data collected from the Blackburn Meadows STW site during 27/02/13. The second experiment on the Don, on 05/06/13 did show a decline in NH_4 by 2.01mgl^{-1}, which would equate to 63% dilution if NH_4 inputs remained constant with the arrival of the much higher magnitude wave (7.97m^{3}s^{-1} against 4.3m^{3}s^{-1} on the 27th). This demonstrates the value in carrying out multiple experiments at a single site, something rarely reported in the literature with Kurtenback *et al.* (2006) and Krein and De Sutter (2001) being the only articles found with replicated results. The difference in dilution between the two experiments could have a twofold explanation. Firstly a greater volume of water is present in the wave so more dilution is induced. There is however another potential contributing factor. The Stocksbridge STW site during 27/02/13 has a pre-release NH_4 level of 1.15mgl^{-1}, the highest recorded during an experiment. It is possible that the higher levels of NH_4 upstream are carried within the wave to reduce its dilution impact downstream. The challenge to this is the distance between the sites. Blackburn Meadows is 20.9km downstream of Stocksbridge STW. It has already been noted in the discussion on the dye experiment results and the literature relating to it that a wave may not consist of the water that was released from the reservoir by the time it reaches a down river point. This point will be further discussed later in this discussion when mixing processes are dealt with. The relevance to the NH_4 data is this; even if the wave would have taken on a significant concentration of NH_4 at Stocksbridge, would this be maintained over 20.9km until the wave reaches Blackburn Meadows? Neither the data nor the literature gives enough information to move beyond speculation. The majority of results recorded in this chapter are below pollution standards set by legal statutes such as Directive 200644EC. One exception is the result from Stocksbridge on 27/02/13 which exceeds1mgl^{-1} NH_4 prior to the arrival of release wave. If this input level is assumed to be constant then the dilution factor of 0.7mgl^{-1} can be given as 61%. This is a demonstration that release waves are able to mitigate a classifiable pollution incident.

Whilst there are many papers that consider the effects of waves on water quality, nutrients are far more rarely considered in the literature. Bariller *et al.* (1993) does record dilution of NH_4 during wave passage, Chung *et al.* (2008) records dilution of total nitrogen, NH_3 and NO_3 and Flouger and Petts (1984) a dilution of NO_3-N. Conversely Henson *et al.* (2007) reports total N rising with discharge during wave passage, and NO_3 data produced by Petts *et al.* (1985) responds in a both varied and minimal fashion. In Henson *et al.* (2007) the changes in nutrient flux are attributed to substrate sources. Whilst the upstream influence on NH_4 dilution can be debated on the Don secondary site, on the Holme Primary site the distance between the reservoir and STW site is smaller. On 09/05/13 the water released from Digley Reservoir has a high NH_4 concentration, and this is the most likely cause of the lowest NH_4 dilution factor recorded at the downstream STW. Putting together the mixed results in the literature and the two stand out results from these experiments a basic conclusion can be arrived at; reservoir releases definitely have the potential to dilute nutrients, but the magnitude of dilution will vary depending on the quality of the diluting water.

One of the purposes in carrying out nine field experiments was to identify any consistency in the relationship between flow and dilution. Polynomial models fitted to the discharge dilution data for both NH_4 and conductivity were highly variable both in their R^2 values and the equations that describe them. Whilst dilution has been established and quantified a consistent relationship has not been found. This indicates that a reliable estimate of dilution cannot be derived from total release volume and pollution concentration alone.

3.6.2.3 DO dilution

Dissolved Oxygen, whilst an important water quality indicator, and a key limiting factor for aquatic ecoystems (Davis 1975) is saturated in all of the data recorded during experiments carried out. Indeed table 3-11 reports that only 1.1% of samples collected over the 24month study period on the Holme fell below the 7mgl^{-1} standard laid down in EU Directive 2006/44/EC. Coupled with the experimental results from the Holme showing a slight increase in DO levels during wave passage and a result from the Don at Stocksbridge replicating this trend, it is difficult to describe this phenomenon in terms of the remediation of a water quality issue. In a sense dilution is achieved because DO increases to an improved state. DO responses to release wave passage are varied in the literature with Chung *et al.* (2008) reporting minimal change, and Bariller *et al.* (1993) reporting a decline. DO concentrations in the River Holme, Don, and Ryburn benefit from aeration at the numerous weirs that line the rivers and the violent fashion that water is released from the reservoir (see the picture figure 3-2). Increases in DO during wave passage likely result from the increased aeration produced by both the release of water and the increase flow over the weirs (Hanson 1995). Such structures are not reported on the larger rivers being studied by both Chung *et al*. (2008) and Bariller *et al*. (1993).

3.6.2.4 Lag times between dilution and wave arrival

Lag times are often a discussion point for papers studying solute and SSC response to the arrival of either artificial or natural wave fronts. The rational being, that lag times are indicative of the kinematic wave theory detailed in Singh (1996), and Glover and Johnson (1974). This discussion has already dealt with this subject in the context of the conductivity data, however the majority the relevant literature focuses on the transport of SSC within the wave (Heidel 1966a; Gilvear and Petts 1985; Gilvear 1989; Leeks and Newson 1989; Bull 1997; Krein and De Sutter 2001; Henson *et al.* 2007; Petticrew *et al.* 2007).Whilst these papers show a range of wave magnitudes and catchment characteristics they all describe a lag between the arrival of the wave front and the rise in SSC or turbidity of varying degrees. Turbidity data presented in this chapter generally does not show a lag time with only one experiment on the Holme and the two on the Don providing any evidence of a lag effect. The reaction of temperature and pH to wave arrival is also very rapid, though also very limited. Changes in temperature are specifically related to a change in source water, rather than simply an increase in discharge. In figure 3-37 a natural flow event has a very minimal impact on temperature compared with that of the release waves. Therefore any lag effect associated with the arrival of reservoir water being delayed against the rise in discharge should be evident in temperature change with the cooler reservoir waters being delayed. There is evidence for a delay in peak temperature drop in the Ryburn result and the Holme on 29/05/12, but otherwise the data presented in this chapter does not appear to corroborate well with the literature. It is difficult to dismiss the kinematic wave conclusion however turbidity was dominantly sourced from the river by the wave front, and the temperature response was so limited. As noted in the discussion on conductivity the critical difference could be the influence of the point source pollution. Papers such as Barillier *et al.* (1993), and Malatre and Gosse, (1995) are concerned with urban pollution but the majority of the other references given in this paragraph are describing water quality in the absence of specific identified inputs. Whilst there is minimal lag between the discharge and the general water quality parameters there was a lag between peak flow and peak dilution of both conductivity and NH_4. A linear relationship between discharge and dilution was not established. This lag was consistent across field sites and times of day suggesting that temporal variations in the STW input are not the explanation. A kinematic effect could explain this result with the wave front having a higher conductivity and NH_4 concentration than the reservoir waters further back within the wave and thus producing a lower dilution.

3.6.3 Mixing in the Water Column

The subject of vertical mixing in the water column is discussed in depth in chapters 4 and 5 with reference to the literature and is relevant to aim A3.3 of this chapter. Of the papers concerned with reservoir releases explicitly little attention is given to any in water column processes. There is some limited evidence from the three tiered depth conductivity probes that conductivity declines at the surface faster than the rest of the water column. There is clear evidence that the probe near the bed is the slowest to respond to wave arrival. It is therefore unclear whether there is an even stratification of conductivity throughout the water column, but the bed does impose an edge effect. Beer and Young (1983) in their proposed model for longitudinal dispersion discuss the importance of edge effects, or what they term dead zones. Areas of much lower velocity water near the bed or banks that from a mixing point of view can be considered in temporary storage. Kilpatric *et al.* (1970) in their description of dye progression during tracer experiments note the slight lag in chemographs near the river bank. A differential in conductivity between the majority of the water column and the area close to the bed or a bank corroborates well with this view of river mixing. The picture of vertical mixing shown in the results of chapters 4 and 5 indicate a faster moving area of water near the surface that spreads particles down the river rapidly. Such results cannot easily be verified based on the data shown in this chapter. The most likely explanation for this are time, and complexity. The time scale of relevance in the flume and CFDM results is seconds. In the field all results are in minutes. At a single point in a river, from a management perspective anything less than 15minutes has questionable meaning. Flume and CFDM experiments are a simplification of reality. It is possible that the clear stratification of flow shown in such environments is lost in the increased turbulence found in an actual river course. A combination of these two factors would make the results of the two later chapters hard to identify in the field.

3.6.4 Testing Variable Scenarios

Aim A3.4 covers a series of questions that have been grouped under the catchall of variable scenarios. Some of these questions such as; what effect does wave magnitude have on dilution and wave progression have already been dealt with above and will not be handled again to avoid repetition.

3.6.4.1 Seasonal Change

The most notable influence of season is on the water quality at the reservoir. Both summer experiments at Digley Reservoir on the Holme catchment feature elevated concentrations of NH_4 with 09/05/13 being particularly high. This influx of NH_4 during the release does impact downstream dilution with NH_4 dilution at the STW on 09/05/13 being minimal compared with the rest of the Holme data set. Aside from these two releases NH_4 concentrations in the reservoir stilling basin are marginally higher in the summer months with 4% of samples exceeding $0.2mgl^{-1}$. There are two possible causes of summer nutrient enrichment. One is increased runoff (Turner and Rabalais 2003), the other is the release of sediments during the spring turnover (Petts 1984; Horn 2003). High concentrations of nutrients in upland water bodies are associated with agricultural input (Berman *et al.* 1984). Given the dominance of low intensity sheep farming in the catchment agricultural inputs seem an unlikely source.

3.6.4.2 Diurnal Change

The one night experiment 09/05/13 can be considered an exceptional case for reasons already discussed that are not necessarily influenced by diurnal cycle. The NH_4 spike generated from the reservoir cannot be explained from the literature as a diurnal scale phenomenon. As mentioned in the results description for figure 3.39, output from the STW is diurnal, with NH_4 and conductivity typically being at a lower concentration at night after 23.00 and before 08.00. This is a potential second factor influencing the low dilution of NH_4 achieved on this date. It is however difficult to separate this from the influence of the reservoir water quality. The only other divergent result from the day time experiments is the trend in DO flux. During the day time experiments on the Holme, there is a slight rise in DO during wave passage. During the night experiment there is a marginal decline. This result is too minor to warrant significant attention though. From a management perspective diurnal fluxes should not be a major consideration in release management.

3.6.5 Releases as Pollution Incidents

Some of the reservoir releases carried out by water companies in the UK have been considered pollution incidents due to the impact they have on rivers. The argument laid out by the regulator, the EA, under the Common Incident Classification System (CICS) is that any activity that significantly changes the colour of a river could be considered a pollution incident (Tindeall pers. comms 2012, EA 2007). Indeed one of the experiments on the Holme detailed in this study received an advisory warning from the EA on these grounds. It is therefore important to consider the case that reservoir releases have a negative impact on water quality. The distinguishing feature of reservoir releases that leads them to qualify as pollution incidents is that they are manmade. Figure 3.38, depicting discharge on the River Holme for a 24 month period, illustrates the small scale of the reservoir release events when compared with the natural. Secondly the natural flood event shown in figure 3.37 generates a turbidity spike comparable to a reservoir release event. An argument could be made that the turbidity generated in reservoir releases is reservoir sourced, but this is negated by the results shown in this chapter. Turbidity concentrations are higher at the STW than the reservoir in five out of six experiments on the Holme and on the Ryburn. Sediment is entrained in river in the same manner it is in natural flood events. DO flux during wave passage has already been discussed but it is relevant to repeat that DO is not adversely effected by release events. In general terms then, it is difficult to consider reservoir releases as pollution incidents. There are however two caveats. First, as already mentioned, if nutrient concentrations within the reservoir are high, this can have a downstream effect. In the data presented here this appears to limit dilution rather than creating an incident of itself though. NH_4 concentrations did not rise through wave passage at the STW. And second, natural flood events typically follow rainfall and overcast conditions. Under these conditions the general public and river stakeholders are used to discolouration in the river. It would therefore could have a negative impact on the public's perception of the river if a reservoir release was carried out in a dry period.

3.7 Conclusions

Two of the key questions posed at the start of this thesis have been answered with empirical data in this chapter. It has been shown that a release wave does move substantially faster than the baseflow of a river and could catch a pollution slug within a defined time period. It has also been shown that a release wave does have a diluting effect on effluent associated water quality parameters such as conductivity and NH_4. Replication of results across sites of differing scales, different seasons, and different wave release profiles has shown that although dilution can be produced reliably, but the magnitude and duration of the effect varies dramatically. A consistent relationship between discharge and dilution was not established. A water manager would therefore have to consider all of the variables required for a mass conservation equation if an accurate prediction of dilution was required. A lag time was consistently reported between peak flow and peak dilution for both conductivity and NH_4, but on two experiments demonstrated a lag for turbidity or temperature. Evidence for stratification of water quality within the wave front was mixed. It is clear that the river bed imposes an effect on conductivity but it was unclear as to whether the wave front had a significant impact on conductivity fluctuations at the surface of the water column giving a very limited result for A3.3. This result makes verification of the flume and CFDM chapters that follow challenging.

Chapter 4. The Flume Study Approach

4.1 Introduction

This chapter presents results from a series of flume tank experiments that investigate waves moving down a tank interacting with pollution substitutes such as rhodamine dye, kaolinite, jelly, and olive oil. The results are then used to comment upon mixing processes, placed into the context of the literature.

It is difficult to ascertain a two dimensional picture of mixing processes from a field study such as that described in chapter 3 due to equipment requirements and a lack of control over physical variables, such as water quality, and flow regime. For instance, in theory water samples could be collected at a high 3D resolution throughout the water column during wave passage, but this would require tens if not hundreds of samples to be collected simultaneously from the mid-stream during high flow. For this project, such resources were not available. In a flume tank environment these variables can be strictly controlled and these difficulties mitigated. Using digital video, data can be collected in both vertical and horizontal dimensions providing a detailed picture of the physical processes involved.

4.2 Literature and Background

In justifying this study approach, and both this and the following CFDM chapter inclusion in this thesis, the key question is; does mixing matter? Does mixing have a big influence over dilution rates? When a wave front catches up with a slug of pollution do the mixing processes that occur have a significant impact on the resulting dilution? Mixing in a river occurs across three axis; the vertical water column, horizontally cross channel, and longitudinally up and down stream. Each of these has been described in detail in Rutherford (1994). Given that dilution can be expressed as the volume of a substance divided by the volume of the water, spreading the substance throughout a greater volume of water will increase dilution. Within a river, the increase in water volume from a given point is limited in both the cross channel and water column axis, but longitudinally it is great. Question Q3 from the QAM in chapter 1, which was stated "What mixing processes occur when a wave catches pollution?" can therefore be reconsidered as a question of longitudinal mixing. To what extent will a wave of water increase longitudinal mixing? This question is worth investigation since it provides an alternative dimension to increasing dilution beyond simply increasing the depth of flow, and therefore volume of water at a polluted point in a river. Both cross channel and mixing through the water column may have a profound effect upon longitudinal mixing. Beer and Young (1983) in their description of longitudinal mixing models states that models often divide the river channel in to different mixing zones. The zones near the bed and banks being dead zones that are best described by Fickian diffusion, otherwise known as molecular diffusion, (Fick 1855) as they are dominated by boundary conditions which mitigate turbulence and slow downstream flow velocities. The zone within the main river channel approximates more to non-Fickian particle diffusion models due to the dominant effect of turbulence (Elhadi *et al.* 1984), and therefore is considered turbulent diffusion. Longitudinally dispersion, dispersion being a catchall term for the mixing of a substance, of a pollutant could be considered as a product of velocity of the main river channel zone, and the rate of transfer of pollution to the dead zones at the bank and bed. As the pollution slug moves down river, to what extent is the polluted matter entrained by the boundary conditions and left behind, thus extending the reach of river the pollution is distributed down and diluted, is the key question. Equal to these processes the differential advection must be considered. A substance will move at the mean rate of flow with the current down river. This is known as advection (Nepf *et al.* 1997). If flow velocity is not uniform, advection will be varied and the pollutant will move at different velocities and longitudinally spread down river (Elhadi *et al.* 1984). Longitudinal dispersion is then the total result of advection and diffusion, with diffusion moving particles between zones in the stratigraphy and promoting forward motion of particles. There is a third process that drives mixing, secondary circulation which occurs when a bend in a channel redirects particles. In a pipe this is often a spiracle pattern (Hawthorne 1951) although in a river this is more complex (Lane *et al.* 2000). Producing a bend in a flume tank was beyond the resource capabilities of this study and so is not considered in this chapter, but flume tank results must always be viewed as a simplification of reality.

With this view of fluid mixing in mind, the question that arises is; how does a rapid rise in flow in the form of a wave affect these processes? Studies examining longitudinal dispersion do report shorter residence times at higher flows (Wallis *et al.* 1989; Richardson and Carling 2006). Higher flows involve higher velocities and therefore the solute under study departs the study reach in a shorter time period. Richardson and Carling (2006) report a higher dispersion fraction at higher discharges, whilst Wallis *et al.* (1989) report variation between river reaches attributed to the boundary layer conditions associated with the bed and bank roughness. Neither of these studies deal with unsteady flow situations though. Graf (1995) carried out a series of dye experiments in Grand Canyon reaches of the Colorado River, some in steady state low flow conditions and two during dam release unsteady scenarios. In contrast the aforementioned studies, a far greater dispersion of the dye at low flows was found, even those involving the release of a wave. This is attributed to the greater importance of bed geomorphology, such as bars and riffles, in increasing the sinuosity and variation in velocity gradients, and therefore dispersion. This paper does appear to stand alone however, with few others considering wave motions or decreases in dispersion with a rise in flow, suggesting that this result maybe site specific.

It has been demonstrated that flood waves have a higher travel velocity, or celerity to use the specific term, than mean water velocity (Heidel 1966). This relationship between celerity and mean water velocity is best described by kinematic wave theory (Ponce 1991). Glover and Johnson (1974) further develop this study area by examining the effect of this lag between wave front and mean water velocity upon chemographs. Graphing the lag between the rise in flow and the drop in Ca^{2+} ions the authors demonstrated a backing up of regular river flow against the storm flow of the wave. What is the relevance of this to mixing? The kinematic nature of the wave has a key implication. Over a long enough reach of channel, the wave could leave the flood waters behind or, in the case of this study, the polluted waters behind. The effect of the wave upon the polluted water must be considered temporal. Therefore, a key question that this chapter needs to answer is; to what extent is the wave kinematic? In so far as, what proportion of the water within the wave at the entry to the flume tank remains in the wave at the bottom end of the wave tank.

Waves in the marine environment are oscillatory motions of energy (Masselink and Hughes 2003). This cyclical motion, or the breakdown of this motion is considered in mixing studies in the marine environment (Ivey and Nokes 1989). When considering mixing during wave passage with in a river, it is worth a brief survey of wave theory papers with a focus toward rivers and mixing processes. In the study of rivers, hydrologists have a tendency to consider flood waves as temporal shifts in stage and flow, rather than as waves. Considering early works on hydrograph theory such as Nash (1957) it is clear that the response of a river system to a water input is considered as a volume of water over time. Consequently literature concerning waves is generally authored by mathematicians and oceanographers, rather than riverine scientists. The founding papers of the field (Stokes 1847) demonstrate this. Flood waves have a greater length than height, and therefore are categorised as long waves as described by Madsen (1971). In standing water, irrotational waves have been shown to create a longitudinal drift in particles near the bottom boundary (Longuet-Higgins 1953), this has further been demonstrated with low amplitude, longer waves by (Iwagaki and Sakai 1970). These papers, and the flume tank observations they used, are based are predominantly in standing water with limited turbulence. Given the scarcity of papers considering mixing during wave passage in rivers and flume simulations of such situations, papers examining wave transport from a pure mathematics perspective will be considered in the discussion of the results displayed here. However the lack of interaction between the research disciplines riverine mixing and on wave mathematics makes comparing these two sets of research difficult.

Whilst there is a wide range of literature that approaches mixing within channels, or wave motion from different angles, no studies were found that detail the impact of a wave upon particle distribution in three dimensions. Two papers were found, Mannina and Viviani (2010), and Ani *et al.* (2010) that do model the impact of a wave on longitudinal dispersion coefficient. Neither of these papers examine the particle processes in detail with no reference being made of vertical stratification of flow.

Flume tank methods are generally employed to validate a theory which cannot easily be tested in the field for instance D'Agostino *et al.* (2010) and Toniolo *et al.* (2004) both model debris flows in unstable upland areas. In other cases tanks are used as they provide a high degree of control over variables and allow for experiments to be repeated with relative ease. Experiments such as Storey *et al.* (2008) who examine the impact of flow velocities and water depths on turtle surfacing frequency, or Kneller *et al.* (1997) which report velocities within density currents. Thirdly, product, or engineering structure designs are tested in flumes, Guo *et al.* (2004) tested a gating system for vacuum flushing solids build up in sewers. Each of the studies referenced in this paragraph have certain methodological similarities. Fluid is released down a tank and the resulting processes are measured, whether they be particle velocities, sediment movement, or biological response. In each case these processes are intended to be scaled down representations of reality. To consider the impact of waves on the mixing of a pollution substitute both control of variables, such as wave magnitude, and ease of instrumentation and observation are required.

Flumes have also been used to isolate processes and study them in detail, as opposed to scaling down systems. For instance Hardy *et al.* (2009) used a 10x0.3m flume to study the resulting flow fields for flows of set Reynolds numbers over a gravel bed and quantify the role of the effect of increasing bed roughness on turbulent intensity and the coherence of flow structures. Blois *et al.* (2014) used a 4.8x0.35x0.6m flume to demonstrate the role of the hyporheric zone on reducing flow separation in two steady state scenarios, one with an impermeable bed, and one permeable. Another example by Thomas *et al.* (2011) utilised a 5.5m long flume to examine the impact of a bifurification on flow velocities and secondary flows. Each of these papers exploited the availability of high precision flow field measurements such as Ultra Doppler Velocity Profiler (UDVP) or Particle Image Velocimetry (PIV) to produce detailed pictures of flow velocity across 2 dimensions through time. Expanding this approach Hardy *et al.* (2011a) combine PIV data with what is termed Planar Laser Induced Fluorescence, a technique that uses rhodamine dye and a filtered camera to mark water and then quantify it. The purpose being to identify the kinematic effects of bed material on turbulence by comparing the velocity data with the dye tracking. A similar approach is adopted in this chapter with both UDVP and dye tracking being used to study fluid motion.

This flume chapter has therefore has utilised the advantages of a laboratory flume methodology to provide clarity on the mixing processes with a particular interest in the kinematic motion of the wave, and longitudinal dispersion. An understanding of these and the effect of the wave on mixing in general will influence dilution and consequently inform the results of the field chapter (3) and the recommendations of the discussion chapter (6).

4.2.1Terms

The term pollution substitute is used in this chapter to refer to any substance used in lieu of a pollution substance that might be found in the river. The term baseflow has been defined in chapter 3 of this thesis, and is used in the same manner here, to describe flow prior to the arrival, and post the departure of the wave motion.

4.3 Aims and Objectives

All three of the Key Questions and three of the aims associated with them detailed in chapter 1 and the QAM diagram are of interest in this chapter. To recap these aims are;

A4.1; measure the wave speed and compare it with the baseflow velocity.

This aim is dealt with simply by measuring these two elements, the wave celerity with camera footage, and baseflow velocity with UDVP.

A4.2; Identify mixing processes and the physical parameters that determine them.

O.4.2.1; Measure the progress of the wave front, against the water velocity and dye progression to determine the how kinematic the wave is as defined in terms of particle motion against energy momentum.

O4.2.2; Observe vertical stratification of dye motion during wave passage in order to assess the impact of edge effects and wave motion upon mixing.

O4.2.3; Quantify the impact of the wave on the longitudinal dispersion of the dye, or substitute pollutant.

A combination of visual images and UDVP data was used to establish these patterns.

A4.3; Test varied scenarios for both wave generation and pollution incidents

O4.3.1; Test varied wave form scenarios including, wave length, amplitude and frequency

O4.3.2; Test varied pollution substitutes including a substance less dense than water, and substance that would not suspend in water.

Only the form of the wave and the form of the pollution substitute were varied. Other variables such as baseflow velocity, or fluid density could have been studied but were considered of secondary importance.

To qualify aims and objectives A1 through O4.3.2 it was necessary to vary the wave magnitude, water depth, and wave length. By testing iterations of these parameters more detailed conclusions to these aims could be drawn. For instance if no vertical stratification was observed in deep water compared with shallow, one would conclude that the bed edge effect had limited impact.

In addition to these wave forms however wave trains were also examined. As described in chapter 1, one of the purposes of studying various scenarios was to assist water managers in decision making. If water is to be spent, is it better to spend it as one wave, or a series of waves, and if one wave, should that wave have a higher amplitude or wave length?

Consequently both wave length, amplitude and frequency are all reported on.

The method and results describe scenarios for both dye as a substitute pollutant and a number of other substances, each defined by density. Pollution, as described in chapter 2, can take many forms. Whilst chapter 3 dealt dominantly with solute and suspended pollutants both lighter than water pollutants and significantly heavier are studied here.

4.4 Methodology

In a 5m long Perspex flume tank, waves were released in to a volume of ambient or flowing water to interact with either rhodamine dye or another pollution substitute. The results were filmed on high definition camera. In addition to this a UDVP system was used to measure fluid particle velocities.

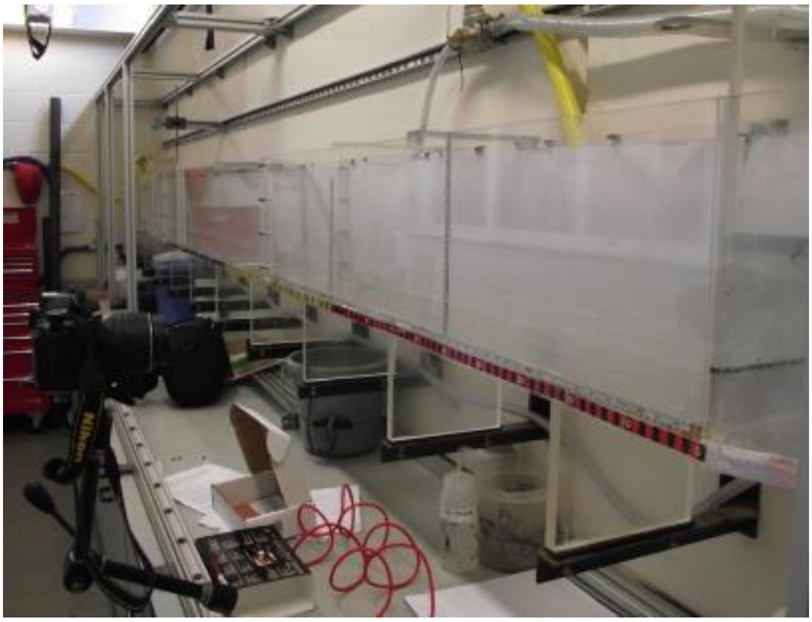


Figure 4-1 a photograph of the flume tank.

Wave water was released from behind a gate at the right hand side of the image and proceeded down tank. The camera was fitted to a steel runner. The overhead pipe was used to introduce flow into the tank.

4.4.1 Materials

4.4.1.1 The Tank

A 5.40/0.30/0.15m Perspex tank was used. The tank was divided in to three sections. Starting from the right hand side of the tank in figure 4.1 (above) or figure 4.2 (below) a Perspex gate is positioned over a 0.05m lip at 0.2m, beyond this there is 5m of unimpeded tank, and then a 0.05m lip dividing off the drainage area that is the last 0.2m of the tank. The gate was lifted in a vertical action by hand to release flow.

The 0.05m lips maintained a minimum stage of 0.05m within the B section of the tank and prevented back flow into area A during wave release. Water is introduced from two points and exits through one. Firstly when the gate above the lip between A and B is drawn upwards the water above 0.05m stage in area A flows into area B. Second, in scenarios with flowing water through section B, as opposed to ambient, water was input through a hose within 0.02m of the gate with section A. The hose was located above the tank and flow controlled by a tap. Unlike many tank systems, this tank has no water cycling system. Water enters at one end, and drains out at the other. A cycling system could not be used as dye would be introduced into the tank. Water leaves through an outflow pipe in the base of area C.

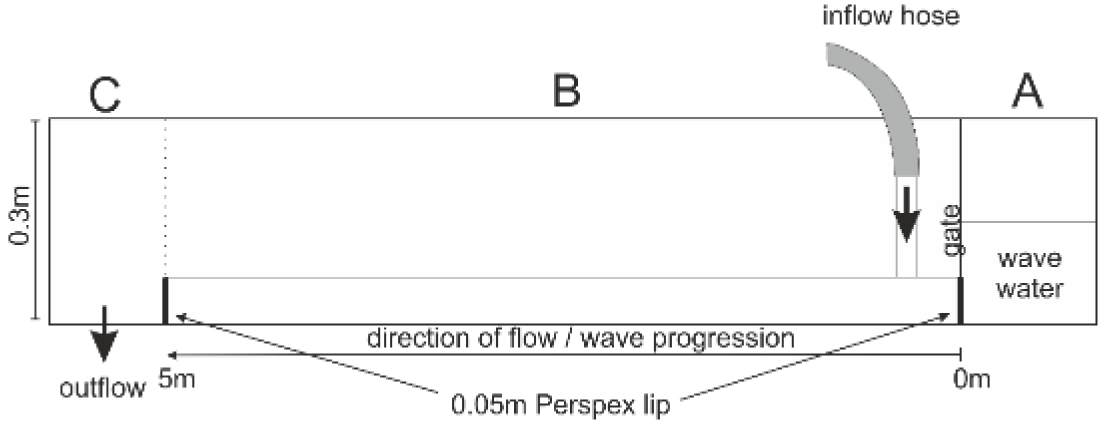


Figure 4-2 A diagram of the flume tank showing the basic components. The tank is divided into sections A, B and C for descriptive purposes. The position of the inflow hose is indicated, as are the two Perspex lips and the gate.

The volume of water released from section A into section B varied with each experiment. The inflow hose was used to generate a mean down tank velocity of $0.035ms^{-1}$ in all experiments. This velocity was used for two reasons. Firstly the on wall tap controlling flow could easily be set to release this velocity reliably. Secondly the River Holme is reported in chapter 3 as having a mean flow velocity of $0.36ms^{-1}$ outside of wave releases. Reproducing this velocity in a 5m tank would require a bigger tap valve than was available so it was scaled down by a factor of 10 to $0.035ms^{-1}$. A bi-product of this method of introducing the baseflow water from the hose was a considerable amount of turbulence over the first 1m of the tank. This turbulence was desirable however as real world rivers rarely exhibit laminar flow (Chow 1959). The water used in the tank was tap water drawn from the mains system.

4.4.1.2. The Camera

A high definition Cannon EOS 1200D camera recording at an image resolution of 1080x1940 was used to capture the majority of the digital video. On three occasions a Nikon D3200 was used instead. Still images were then taken from this footage, using AVS4YOU editing software, and are presented in the results section.

Both cameras were calibrated by mixing a known quantity of dye into a set volume of water within the tank and then filming the resulting colour. This was repeated for between $5mgl^{-1}$ and $0.0005mgl^{-1}$ with 15 concentrations being measured. Calibration was filmed under natural light around mid-day and therefore was only comparable with experiments filmed under similar light conditions.

4.4.1.3 Ultrasound Dopler Velocity Profiler (UDVP)

Ultrasound Dopler Velocity Profiler ,UDVP was used to measure particle velocity on the down tank axis at 5 heights in the water column. With a 0.05m deep water column these were positioned 0.01m apart for the majority of experiments as the UDVP had to be kept submerged to avoid damage, however one run was conducted with the sensors set 0.02m apart over 0.1m. The UDVP recorded at 2Mhz with velocity measurements taken every 0.02 seconds and every 0.74mm in the water column across 63 for the 0.05m experiments or 127bins for the 0.1m run. UDVP was measured on separate runs to the dye filming because the water had to be seeded with kaolinite particles for the UDVP to pick up the motion. Each sensor bounces ultrasound off the kaolinite particles as they move toward the sensor with the sensors being capable of measuring velocities in a cone of 0.02m on the vertical axis. The UDVP was placed 1m down the tank in section B. The two sets of wave velocities, those measured using the UDVP and the horizontal wave progression, or wave celerity, from the film footage have been used to calculate Froude numbers. A Froude number >1 can be considered to be super critical flow, that is where the flow velocity is greater than the wave velocity, and a number <1 can be considered sub critical, that is where flow velocity is lower than wave velocity.

4.4.1.4 Pollution Substitutes

Table 4-1 details substances that were used to either colour the water, or as substitute pollutants. Rhodamine B (40% conc) was used as a general purpose dye for marking water, or a soluble pollutant substitute. Numerous papers report solute pollutants (Meybeck and Helmer 1989; Carpenter *et al.* 1998; House and Warwick 1998). Whilst rhodamine is conservative it is a suitable substitute for visual mixing. Rhodamine B is visually distinct, and would therefore be visible on camera. Olive oil was used as petroleum or crude oil based substitute. A Density of 800kgm$^{3-1}$ is lower than that of water and comparable to that of pump petroleum at 737kgm$^{3-1}$ (Toolbox 2010) To give the oil a darker colour, so that it would show up in the video footage, it was mixed with rhodamine dye. Fruit Jelly was used to simulate a substance denser than water. A sewerage sludge can have broadly any liquid or solid mixture properties. A fruit jelly with its higher density is a good extreme for a sewage sludge substitute. The physical properties of sludges are determined by their water content, however in generalised terms they have a higher viscosity, and plasticity than liquids (Sozanski *et al.* 1997). This lends to a higher shear stress being generated when in motion, a physical property that the jelly shares. The density given in table 4-1 for jelly was measured in the laboratory as a volume by weight. Kaolinite dust was used as a suspended solids substitute. Whilst the Kaolinite has a much higher density than the fruit jelly it is in a powdered state and therefore closer to neutral buoyancy. Suspended solids and sludges are commonly reported in riverine pollution papers (Klekowski and Levin

1979; Chunguo and Zihui 1988; Woitke *et al.* 2003). Between the four substances in use a wide spectrum of solubilities and densities has been represented.

Table 4-1 A summary of the density of the pollution substitutes and what they were intended to represent.

Substance	Density (kg/m^3) at room temperature	Usage
Rhodamine B	n/a – water soluble	Colouration of water, as soluble pollution substitute
Olive Oil	800	As oil or petroleum substitute
Fruit Jelly	1300	As sewerage solids substitute
Kaolinite	2650	As suspended solids substitute

4.4.2 Experiment Sets

The experiments carried out can be divided into two sets termed the bulk tests and the drop tests. The bulk tests involved dying the wave bulk water prior to releasing it into main tank (section B on figure 4-2). By releasing dyed water into the tank the kinematic nature of the wave could be visualised. If the wave moved at a faster velocity that the dye particles it could be deemed kinematic in nature and resolve objective O4.2.1.

The drop tests involved dropping a volume of pollution substitute into a tank of flowing water, and then releasing a wave into it. The interaction between the wave and the substitute pollutant in question would contribute to aims A4.1, A4.3, and with A4.4 being dealt with by using the three different pollution substitutes and variations in the wave release profile.

The following table summarises the experiments, explanation of these sets then follows.

Table 4-2 Each experiment set consisted of taking one variable and running experiments over a number of iterations of it.

variable	iterations
Ambient water depth	0.05m depth, 0.1m depth, 0.15m depth
Dispersion time for rhodamine dye	4 seconds, 6, 10 seconds and a constant injection
Pollution substitute	Rhodamine dye, kaolinite dust, fruit jelly, olive oil.
Wave treatment	Normal wave, 0.005m, 0.01m, 2 peaked wave.

4.4.3 Bulk Tests

4.4.3.1 Ambient Bulk Tests

The ambient bulk tests were carried out by releasing a dyed volume of water (12.5 litres dyed with 1ml of rhodamine B) from section A into section B of the tank (see figure 4.3 below). Section B was occupied by ambient (standing) or flowing water with a depth of 0.05m. This depth was varied in a number of tests detailed in table 4-3 above. A camera was positioned at the 0.5m down tank mark to film the displacement and resulting wave generation. The camera was positioned at 0.5m as this was the approximate distance to which the dye travelled, as shown in preliminary tests. The priority in this set of tests was to produce video footage that clearly identified how much dyed water was transferred with the energy in line with objective O4.2.1. 12.5l for the wave bulk and 0.05m of ambient water depth provided a clearly identifiable wave. 1ml of rhodamine B was sufficient to give this volume of water a clear colouration.

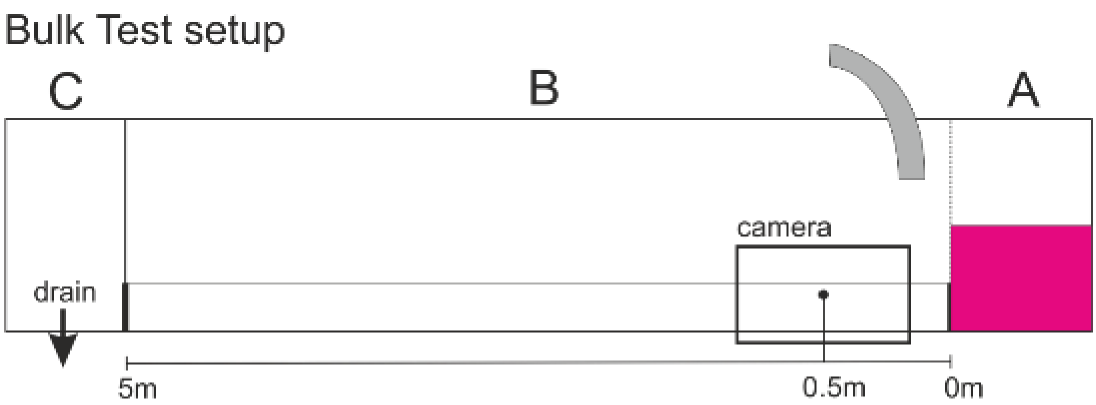


Figure 4-3 The flume tank setup for the bulk tests. Dyed water is indicated in pink, held in section A behind the gate. Note that no flow is released from the hose above the tank.

As described in the literature section of this chapter, boundary effects generated by the river bed or bottom of the tank in this circumstance can have a significant effect on wave motion and mixing processes. Section B was filled to depths detailed in table 4-3 under the variable ambient water depth. Three depths all 0.05m apart were used. By comparing the results from three depths the relationship between wave magnitude and the progression of the dyed wave water down the tank.

4.4.3.2 Flowing Bulk Test

The flowing bulk test was carried out in the same fashion as the ambient bulk test baseline scenario with two modifications. Firstly, flow at a velocity of 0.035ms^{-1} was introduced into the tank, and secondly the camera was repositioned to the 2m down tank mark to capture the extent of the dye progression as seen in preliminary tests. Both of these changes are illustrated on figure 44 below.

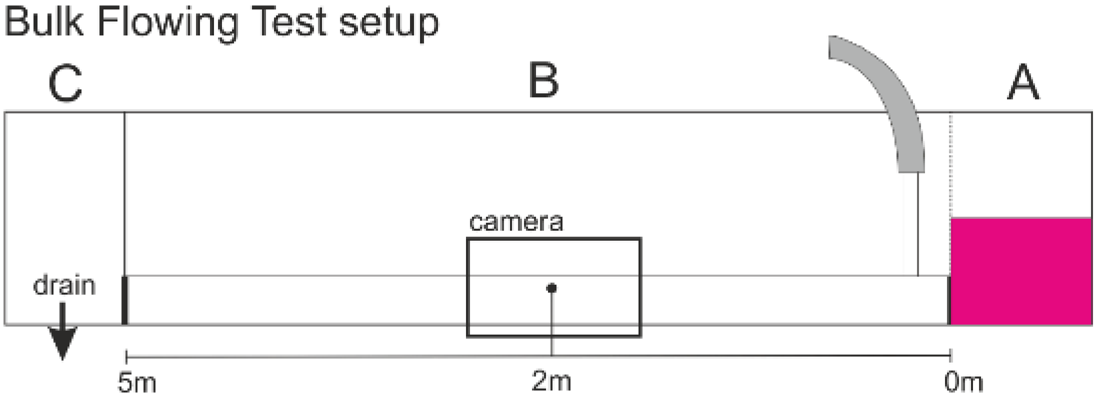


Figure 4-4 The flume tank setup for the flowing bulk tests showing the dyed wave water in section A behind the gate, the input of flowing water from the hose, and the position of the camera 2m down the tank.

A flow rate of 0.035ms^{-1} was applied to section B moving down tank toward section C. This rate of flow created a water depth of 0.055m as the water remained impounded to the height of 0.05m by the lip dividing section B and C of the tank (figure 4.4). This impoundment was necessary to reflect the impounded nature of the rivers studied in chapter 3 of this thesis. The Rivers Holme, Don, and Ryburn have multiple impoundments that artificially raise stage along their course. The lip between sections B and C emulates this.

4.4.4 Drop Tests

The drop tests involved dropping or injecting a volume of a substitute pollutant into a flowing tank and then releasing a wave down tank for the interaction to be filmed. In these experiments section A (figure 4.5 below) contained 12.5litres of water. This was released into 0.055m deep water flowing down tank at a rate of 0.035ms^{-1}. The camera was deployed in two setups for these experiments, either stationary 1m down the tank, as in the ambient tests, or 1m down tank as a tracking camera. This camera was moved down the tank with the wave front to capture the ongoing interaction between the wave and the dye.

For the tracking camera runs the dye or substitute was dropped in 2 seconds prior the release of the wave at the 1m down tank point. For the standing camera runs the substitute was dropped in 4 seconds before wave release at the 0.5m point. The difference in dispersion time before wave release of 2 and 4 seconds was a necessary compromise between producing a realistic scenario and identifying mixing processes (aims A4.3 and A4.4). In the tracking camera experiments identifying mixing processes through the water column was the priority; a shorter dispersion time kept the dye more compact and allowed for the wave to affect the whole dye mass. A 4 second dispersion allowed for the dye to distribute evenly across the vertical water column, a process that is typically rapid in the riverine environment (Day 1976). These experimental designs are shown graphically in figure 4.5 below.

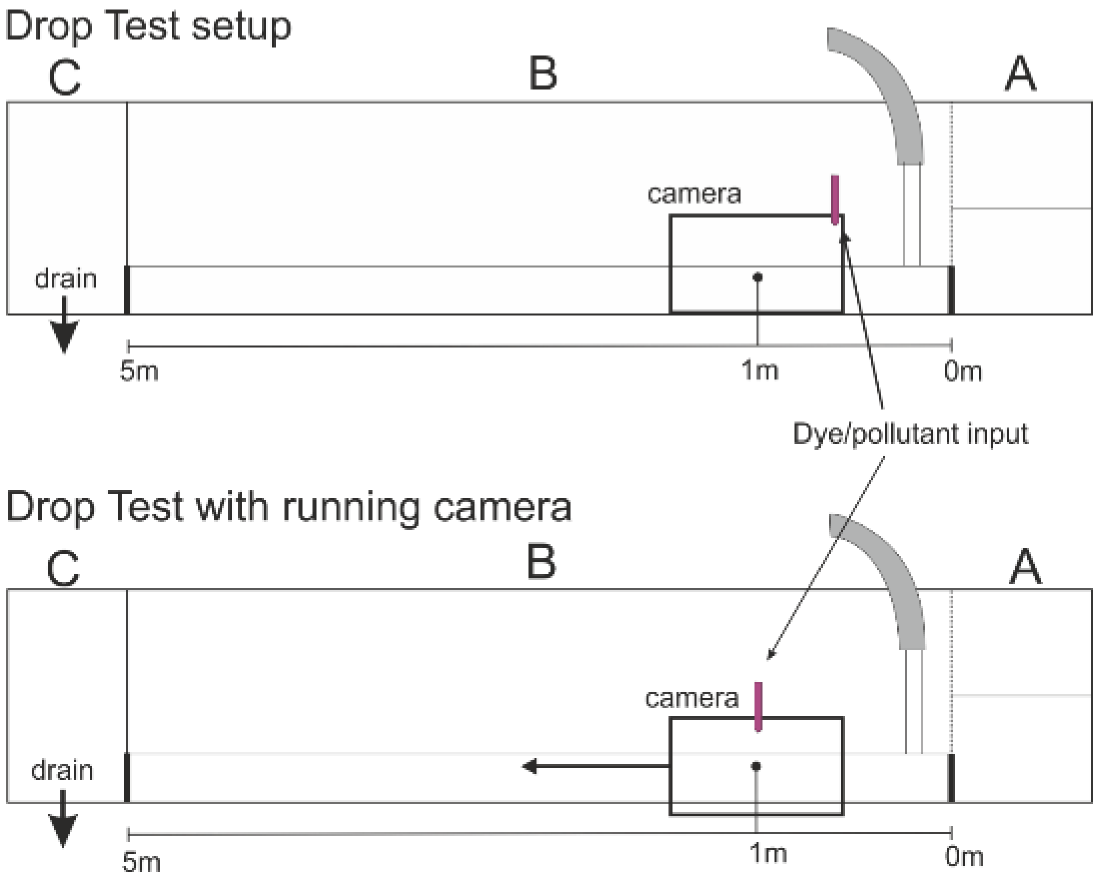


Figure 4-5 The flume tank setup for the drop tests with both a stationary camera and a running camera. The dye is injected with pipette shown above the camera and clear water is held in section A behind the gate.

The input point of 1m (figure 4.5) for the dye / pollutant was used as it was suitably far from the wave and flow inputs to avoid eddies or excess turbulence whilst giving the maximum down tank distance for tracking footage to be filmed over. Preliminary runs showed that dropping dye in too near the hose inflow resulting in it being caught in an eddy.

Where the bulk tests were purely concerned with establishing mechanical processes, the drop tests were concerned with both identifying mechanical processes and examining realistic scenarios. Allowing for varied dye dispersion times prior to releasing the wave, applying different pollution substitutes, varying the wave profile and using a constant rather than instantaneous dye injection are all variables that were tested to meet aim A4.3 in addition to engaging A4.2 and the hypotheses and key questions that proceed both aims.

4.4.3.1 Drop Test Scenario Sets.

Three scenario sets were run for the drop tests as shown in table 4-2 at the start of this section. These included variations in dispersion time allowed for wave release, variations in pollution substitute to examine the effect of a wave on substances of different properties, primarily density, and differing wave treatments.

Dispersion Times

Variation in dispersion times were tested because the degree to which a pollutant might have dispersed down a water course could vary considerably prior to wave arrival. Dispersion times of 4, 6 and 10 seconds were tested and compared with a no-wave scenario. The no wave scenario presents the control, 4, 6 and 10 seconds give a range of comparators for this control.

Continuous Injection

A continuous dye injection 25ml per second through the wave passage was also tested as a more realistic scenario. The focus of this thesis is on short term transient incidents but longer duration incidents did need to be considered to achieve A4.3 of this chapter.

Pollution Substitutes

Pollution substitutes including rhodamine dye, kaolinite dust, fruit jelly and olive oil were all used. These substances have all been detailed in the materials subsection above. In all experiments rhodamine dye was injected at a volume of 1ml. 30ml mixtures of kaolinite dust, fruit jelly, and olive oil with water were dropped into the tank. Kaolinite dust was mixed to a concentration of $1g30ml^{-1}$ water, fruit jelly $2g30ml^{-1}$ of water. 30ml of oil was mixed with 1ml of rhodamine dye to give it a darker colour.

Wave Treatments

In chapter 3 waves off different lengths, magnitudes were examined, as were waves with multiple peaks. The effect of such variations in wave treatment on 1ml of dye was tested. Wave treatments were varied between the ‘normal’ instantaneous release wave, in which all the water in section A of the tank is released with one rapid raising of the gate. Longer waves with a lower amplitude, or peak, but a longer wave length were produced by opening the gate a set amount and forcing the water to drain into section B through a smaller gap. A 0.01m and 0.005m gap were both used. Gaps greater than these did not produce waves visibly different from the normal wave released. Releasing the water through a 0.01m gap took approximately 4.2 seconds to drain section A, a 0.005 gap took 8.5 seconds, and a normal wave took <0.5 seconds. This gave the 0.1m and 0.005 gap waves a longer wave length. A double peaked wave was also released. For this wave, the gate was fully opened for 1 second, then closed for 0.5 seconds and then fully opened again until section A was fully drained.

Table 4-3 Each wave treatment with its Reynolds number as calculated from wave height and velocity recorded in the results section.

Wave treatment	Reynolds Number
Normal Wave	1.1×10^{5}
0.01m	3.8×10^{4}
0.005m	3.3×10^{4}
2 peaked wave	8.1×10^{4}

4.4.5. Footage Presentation

To clearly communicate results still images were taken from the footage. In the results section these were then presented with bathymetric diagrams alongside showing the progression of the wave and dye over time. Solute dispersion clouds do not have a precise edge, but a gradient. In the bathymetric diagram the cloud extents were delineated by colour. The colour picker tool in Corel Draw was used to pick consistent colouration in an image. All figures in the results section are given to the nearest 0.01m as a finer resolution would be unrepresentative of the precision in the data set.

Figure 4-6 a colour map of three different dye concentrations within tank as seen in the results section.

$0.05 mgl^{-1}$ as displayed in figure 4.6 above is defined as having an RGB colour mix of 198-204 red, 53-54 green, and 91-97 blue, $0.025 mgl^{-1}$ is 187-190 red, 56-60 green, and 107-115 blue, and $0.0164 mgl^{-1}$ 194-197 red, 69-72 green, and 130-135 blue. With the relationship between concentration and the blue pigment approximating to linear, this was used to determine concentration. Due to the change in lighting these colour bands could change by as much as +/-10 and so only broad categorisations of dye concentration are used in the results for the majority of the experiments with $0.025 mgl^{-1}$ taken as a division between the higher and lower concentration in the water.

Counting pixels in an image shows the relative change in area over which the dye, or substitute has spread. The relative change does show the spread or contraction of the dye through the water, thus being a good measure of mixing. If combined with a measure of dye concentration associated with the colouration a broad quantification of the quantity of dye in a given frame or area of a frame can be attained.

4.4.6 Numerical Analysis

With data based on images derived from film footage it is difficult to give numerical descriptions. Where in the results section numbers are given for proportions of dye or pollution substitute within a section of the image or tank they were derived from image analysis using the Matlab software. Images were grey scaled and regions of the pollution substitute for a given section of the image were counted by pixel. Some images were edited in Corel Paintshop Pro to remove non dye dark areas of the image to prevent a bias in this analysis.

Specific concentrations through time for the wave treatment scenarios have been given in the results. These concentrations were given in greater detail than three grades given in figure 4.6. For each of these scenarios colour concentration was averaged over a 0.02m by the height of the water column area at 1.4m down the tank at 0.2s intervals. The grey scale image was then converted to dye concentration using the calibration results. 1.4m was used as this area of the tank had few reflections in the Perspex and thus produced a more consistent set of pixel colours.

Turbulent Intensity was calculated from the UDVP results. The raw UDVP data was not without error. A random distribution of extreme isolated numbers was distributed through the data, this was removed by taking each number that was more than 100% greater than its neighbours and reducing it to a 3 sample running average. For the Turbulent Intensity calculation a moving through time velocity average (Reynolds Averaged) was calculated and this was then compared with the root mean square of the velocity fluctuations under the following formula;

$$I = \frac{u'}{U}$$

Where I is turbulent intensity, u' is the RMS velocity fluctuations and U is the Reynolds Averaged velocity. Effectively this gives a ratio between the fluctuating and mean flow components of flow velocity.

4.4.7 Limitations

The reduction in scale and simplification of the channel topography between the 5m Perspex flume and a river system was a significant limitation imposed by using this methodological approach. A river system has meanders, a pool riffle system, bars, a sub straight of variable roughness and variable depth. The flume tank by comparison is straight, has a low roughness with a Mannings n < 0.030 compared with an upland river of ~0.050 (Chow. 1959), and has a constant depth. Equally the scale of the fluid motions is different. The flume has a flow depth of 0.05m and 0.1m during the wave peak, whereas in the River Holme these values have ranged as high 1.4m depending on location and flow conditions. During a baseflow of $0.8m^3s^{-1}$ the River Holme had a Reynolds number of $2.2x10^5$ by comparison the flume tank during baseflow yields $1x10^4$. The wave profile is another point of difference, in the flume the normal wave treatment resulted in a very rapid rise in flow over <0.4s, on the River Holme a release wave took 15 minutes to near peak flow, through waves on the Don had a shorter rising limb. Therefore a question has to be raised over the results presented in this chapter, do they scale to the size of a river reach? The model presented in chapter 5 deals with this question.

The lip installed at the bottom end of the tank was intended to replicate the prominence of impoundments within a river system that artificially raise the stage. The results of this chapter must therefore be considered limited to reaches of river affected by such an impoundment.

Dispersion time prior to the arrival of a wave in a river situation could vary considerably. The dispersion times used in this chapter were counted in seconds and consequently short. Variable dispersion times were studied by with 10 seconds being the longest period considered.

The UDVP system relied on kaolinite particles in the water to bounce ultra sound off. Kaolinite, whilst buoyant, is not neutrally buoyant having a greater density than water. It is then possible that the velocities recorded by the UDVP are not completely representative of the water particles. Secondly the presence of the UDVP within the tank creates an obstruction to the flow that would further slow velocities and potentially create secondary flows around it.

Dilution is an ongoing process that varies with space and time. When comparing the dilution resulting from releasing a wave against dispersion in the channel with no wave release the key question is where and when do you measure. It was not possible to place cameras along the entire flume length, so the total dispersion of the dye at any given point, and thus dilution could not be measured. Rather the camera was used to take a sample at a given point for a period in time, either tracking the wave front or standing at a set distance. To measure the total effect of the wave within the 5m distance of the tank the optimal position for the camera would be at the far end of the tank. This would show the maximum extent of dispersion before the dye left the tank. This was not done for two reasons. Firstly the lip created a low magnitude refraction wave that invalidated the results if recorded this far down tank, secondly Aim A4.2 was to assess mixing processes during the wave interaction so camera placement had to account for both the resulting dispersion and the interaction between wave and dye at the point of contact. This created a limitation as dispersion was measured at the 1-1.25m point with dye concentrations being estimated through time. Whilst these results are sound they only detail dispersion over a very limited time and longitudinal distance.

The colour and brightness of pixels of footage change with the light conditions. The majority of footage was shot during the middle of the day in good sunlight however natural light levels do fluctuate. This had an effect on the comparability of footage between experiments and the camera calibration. To counter this the brightness of the images was manipulated within Core Draw to a benchmark, however the precision of such a technique cannot be assured. The concentrations that have been given for the dye are therefore of limited accuracy.

4.5 Results

The video footage results from the flume experiments are displayed as a series of bathymetry graphics paired with stills from the footage. Each diagram represents the movement of dye and water surface level over time using a colour gradient.

4.5.1 Bulk Tests with Ambient Water

The baseline graphic (figure 4.7) shows the result from the standard ambient bulk test.

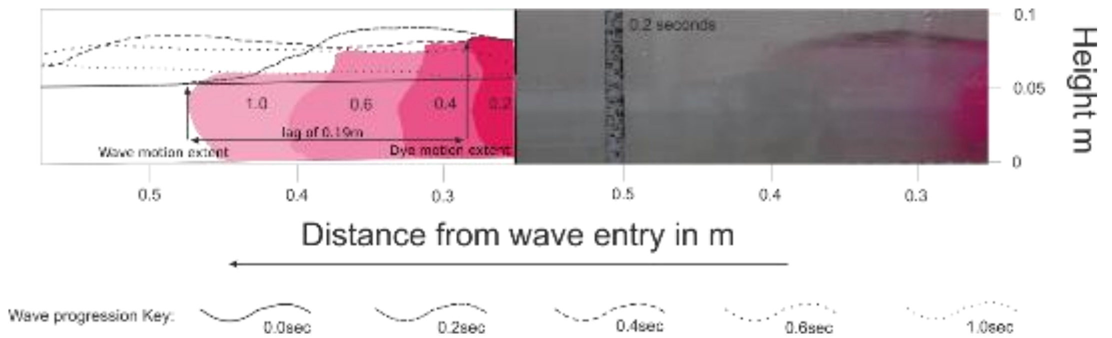


Figure 4-7 Results from the standard ambient bulk test. The dotted lines represent the water surface at various points in time defined in the key. The progressively lighter pink coloured areas represent the area of tank occupied by dyed water at a given point in time. Labelled in seconds since the appearance of the wave on frame. The right hand panel is a still image for comparison.

At 0.2 seconds after the wave front has arrived on frame (passed 0.25m down tank) the wave is 0.19m further down tank than the dye front. This lead is rapidly extended off camera and as the wave front accelerates down tank the dye does not maintain pace. At 1 second, the wave has progressed off frame and the dye is at 0.48m showing that this wave is definitively kinematic, as energy is transferred from the water released from section A into the ambient water being displaced. Whilst the water here is not flowing this result contributes to O4.3.1.

In the following diagram, the results from increasing the ambient water depth from 0.05m to 0.1m, and then 0.15m are displayed. This change in ambient water depth had two effects. Firstly it reduced the wave magnitude, or amplitude (the height between the wave peak and base), and secondly it moves the system closer to a deep wave scenario, where the bottom effect is more limited.

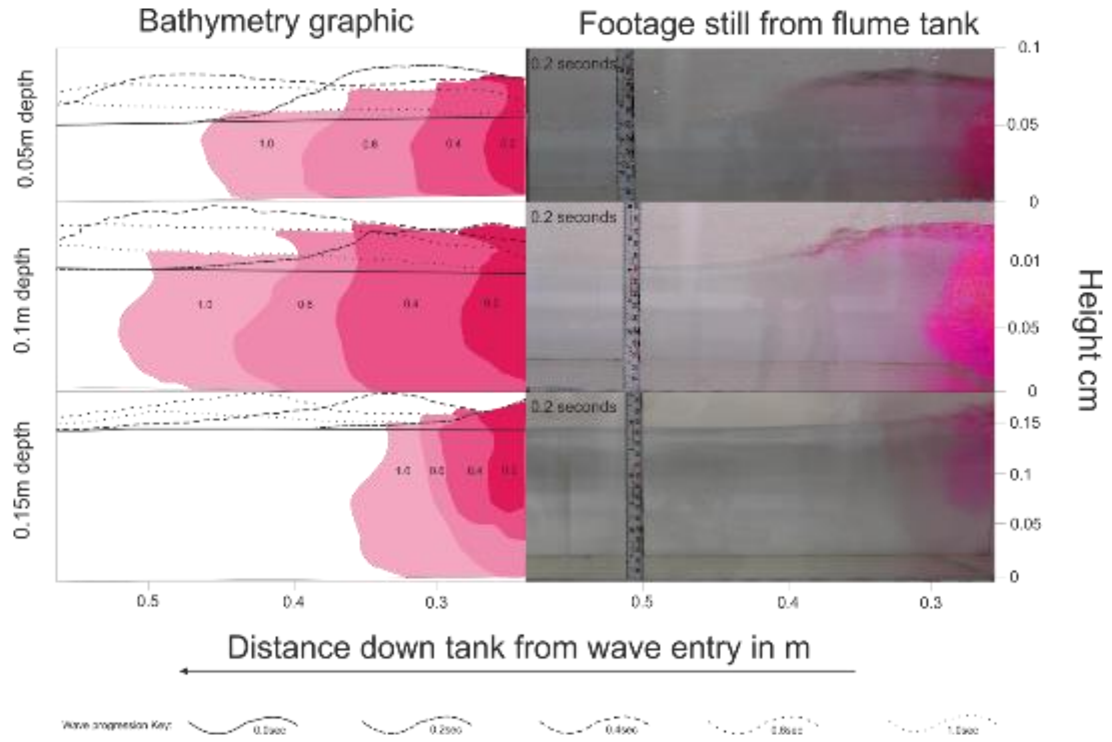


Figure 4-8 Results of the ambient depth experiments. Results from the three water depths tested are shown in descending panels with a bathymetric graphic and the original footage side by side. Dotted lines and gradations in colour have been used to show the position of the wave and dye mass through time.

The reduced amplitude of the wave is clearly visible in the 0.15m scenario (bottom of figure 4.8) with an amplitude of 0.023m compared with the 0.04m of the 0.05m scenario and the 0.036m of the 0.1m scenario. The dye progression in the 0.15m scenario is also the most clearly different, with the dye failing to progress beyond 0.36m down tank by 1 second after the wave arrived on frame. The 0.1m and 0.05m manage to progress 0.53m and 0.47m respectively. Again the wave proceeds down tank in a kinematic fashion, but most clearly in the deeper 0.15m scenario.

4.5.2. Bulk Tests with Flowing Water

With a flowing rather than ambient water body in section B of the tank the experiment progresses on from an Archimedes' bath tub scenario to a more realistic in-river situation. Figure 4.9 below shows camera footage from 2m down the tank at the point where the dye starts to lag behind the wave front.

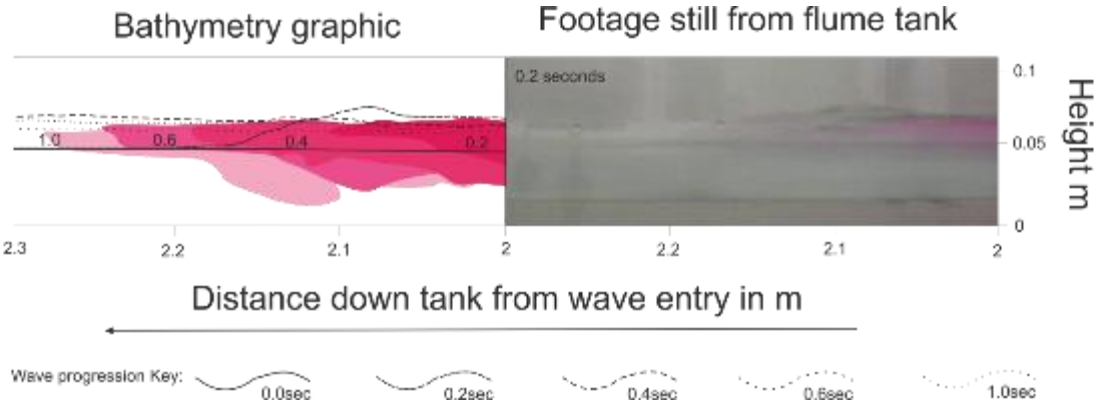


Figure 4-9 Results of the bulk test in flowing water. Two panels are given, on the left a still image at 0.2 seconds after the wave arrives on shot and a bathymetric diagram through time on the right. The position of the wave front (dotted line) and the dye mass are shown at 0.2, 0.4, 0.6 and 1.0 seconds after wave arrival.

With flowing water the dye is forced over the top of the baseflow water. This over the top flow is maintained across this footage frame. Beyond this point dispersion eliminates the stratigraphy to an extent. At 2.1m at 0.2 seconds after the wave has arrived on frame the dye is only 0.04m behind the wave front. By 0.4 seconds the wave front has left the frame leaving the dye behind. The closer relationship between the dye front and wave front suggests that this wave is not as explicitly kinematic as those seen in the ambient water scenarios.

4.5.3 Drop Test with the Camera Tracking Dye

The basic experiment for this test set involved dropping 1ml of rhodamine dye into the flowing water in section B, and releasing a 12.5l wave into the tank after a 2 second pause as described in the methods. The results filmed with a tracking camera are presented in figure 4.10 below.

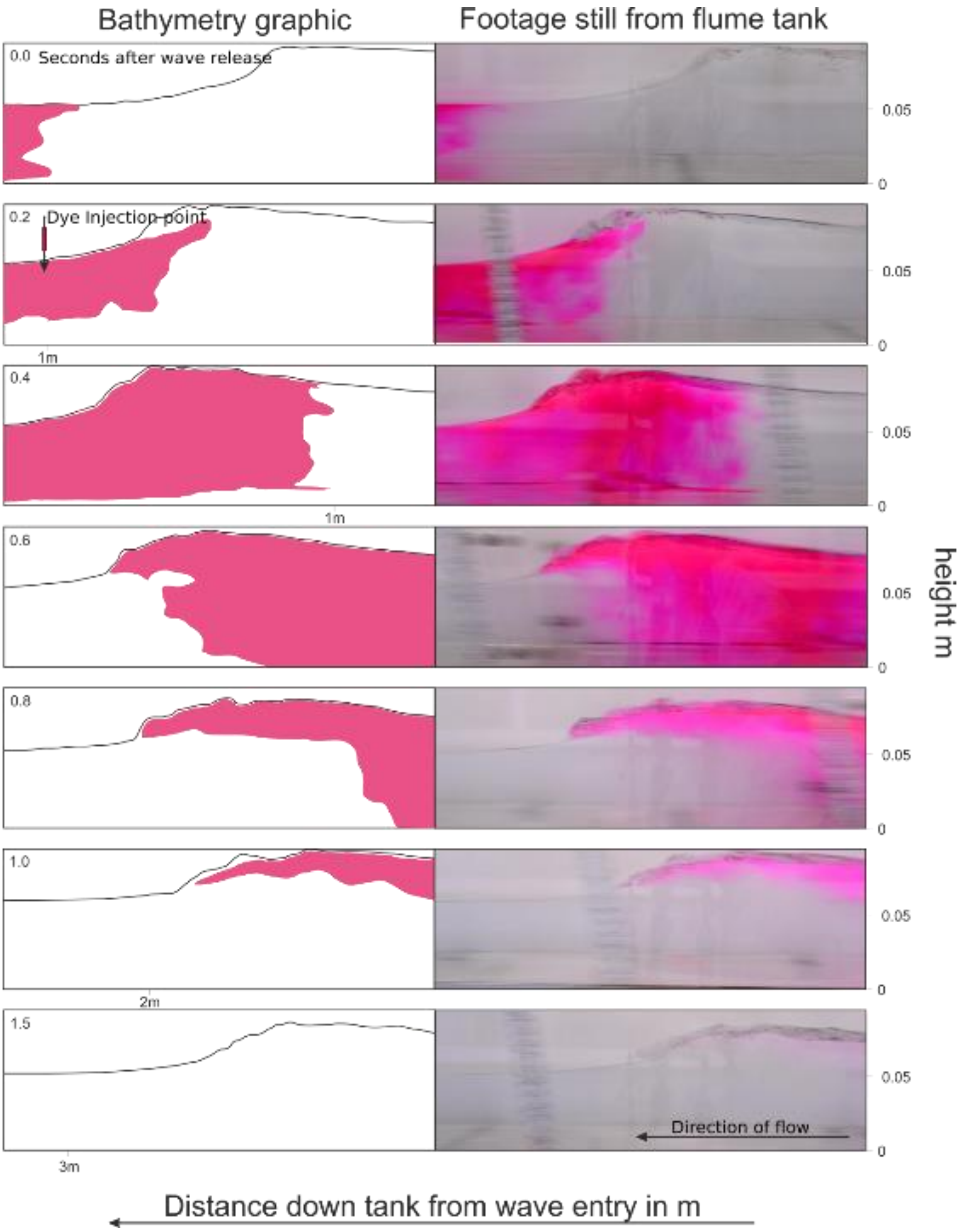


Figure 4-10 Camera tracking a wave and a dye slug down the tank Distance intervals of 1m are marked at the bottom of each frame and time steps are noted in the top left hand corner. Footage stills are shown on the right and bathymetric diagrams on the right.

The dye was added at the 1m mark, and the wave proceeds from the right hand of the frame toward the left dragging the dye with it. The wave catches the dye after 2.2 seconds of dispersion at the 1m point (frame 0.2). The dye is then propelled down the tank with the wave until the 2.3m mark (frame 1.0), by 3m the wave has outpaced the majority of the dye with some lightly coloured water still in transit. The 0.6 second frame is most indicative of the processes at work. The deeper red coloured denser dye is concentrated both in the upper 0.02m of the water column and in a mass behind the wave. In the 0.8 second frame the mass of dye behind the wave has been left behind and the dye in the upper water column is in the process of being spread down the tank. A clear vertical stratification of dye concentration can be seen in the 0.6 and 1.0 frames. In the 0.6 frame there is no visible dye below 0.05m in the water column.

As described in the methods dye distribution can be quantified by counting pixels. These results are summarised in table 4.4 below. The dye quantity within 0.1m of the wave peak increases to 906279 pixels of coloured dye at 0.2 seconds after release, with 73.33% of these being of the darker read colour indicating a concentration in excess of $0.025mgl^{-1}$. This then increases by 238% at the 0.4 second mark, for the dye to then decline over the next 4 frames to there being 0 dye at 1.5 seconds.

Table 4-4 details the change in dye concentration and volume as measured in pixels in figure 4.10.

Time after wave arrival (seconds)	Total Dye Pixels within 0.01m of wave peak	Percentage change	Percentage Dye concentration > $0.025mgl^{-1}$	Percentage Dye concentration < 0.025mgl-1
0.0	0	0	0	0
0.2	906279	0.00	26.67	73.33
0.4	3065649	238.27	43.80	56.20
0.6	2278067	-25.69	35.78	64.22
0.8	1318165	-42.14	29.85	70.15
1.0	707279	-46.34	0	100
1.5	130943	-81.49	0	100

4.5.4 Drop Test with Variation in Dispersion Time Prior to Wave Release

The dispersion results were captured with a standing camera. The Bathymetry diagram below shows results from 5 time steps, 0, 0.2, 0.4, 0.6 and 1 second.

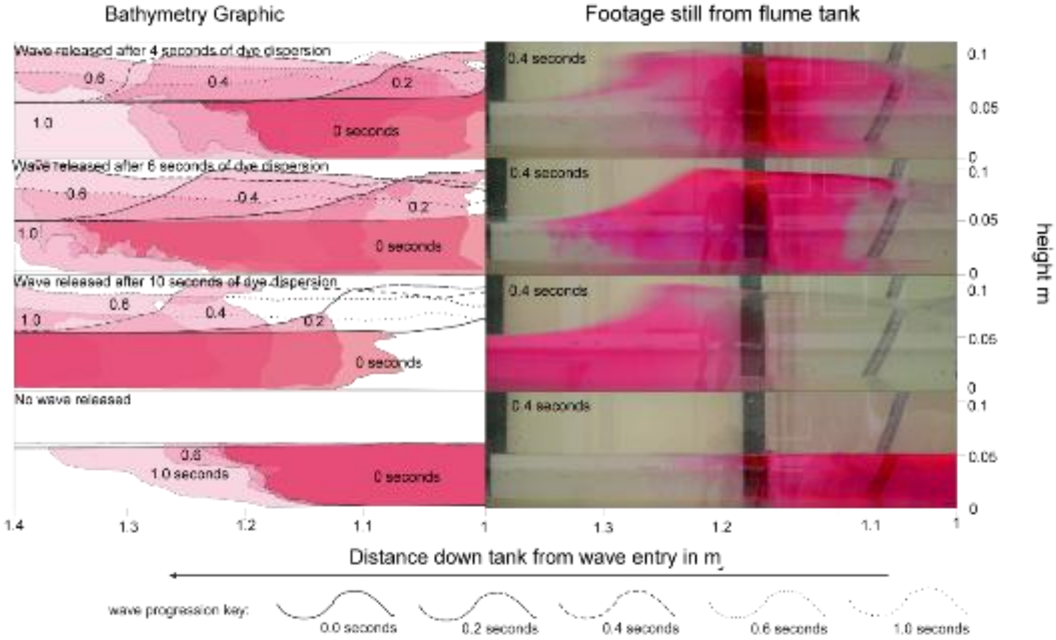


Figure 4-11 Tests with 1ml of dye allowed to disperse over varied time periods prior to wave release. The top frame covers the 4 seconds of dispersion, the second frame 6 seconds, the third 10 and the fourth the no wave scenario. Each bathymetric diagram shows the dye motion from 0 seconds after the arrival of wave front, until 1 second at intervals of 0.2 intervals.

The bottom frame in figure 4.11 above shows the dye progression when no wave is released. Given that the other diagrams start at 0 seconds with the arrival of the wave front a time had to be selected to start presenting results in the absence of a wave. The 0 seconds for this diagram is set 4 seconds after dye injection. This result provides a control experiment case for all of the drop tests. The dye front manages to progress 0.16m down tank over a 1 second period. This is 4.5 times faster than the 0.035ms^{-1} flow rate. Whilst Fickian dispersion is not applicable to all rivers (Nordin and Troutman 1980), in a channel with low turbulence and a low velocity of 0.035ms^{-1} there is time for diffusion of particles to have a noticeable effect on the forward motion of the dye.

At 0 seconds as labelled in figure 4.11 both the top frame of 4 seconds of dispersion and the no wave scenario at the bottom frame, the injected dye has dispersed to the 1.2m down tank point, having been dropped in at the 1m point. Comparing the top frame, 4 second dispersion scenario with the no wave scenario the impact of the wave against the control can be seen. At 0.2 seconds neither dye cloud has progressed more than 0.01m further down the tank, at 0.4 the wave front has moved the dye 0.04m whilst the no wave scenario has progressed 0.01m. After 0.6 seconds, the wave has moved the dye beyond the 1.4m of the footage frame, in the top panel, with the dye only progressing to the 1.27m point in the now wave scenario. The layering technique used in the bathymetric diagrams to an extent obfuscates the precise rearward extent of the dye in 4 second dispersion frame in figure 4.11. However, the back of the dye cloud occurs at 1.12m. In the no wave scenario, the low flow rate and slight eddying of the water has allowed dye to disperse upstream off-frame. The longitudinal progressionof the dye is clearly greater in each of the wave scenarios, however given that the rearward progression of the dye cloud is not captured for the control scenario a definitive statement on the total longitudinal dispersion cannot be made since only the forward extent of the dye cloud can be described numerically.

The scenarios for 6 and 10 seconds of dispersion prior to wave release show a similar forward motion of the dye to the 4 second scenario. In the 10 second scenario the dye cloud has had sufficient time to progress beyond 1.4m down tank and the tail of the cloud has reached 1.08m down tank. The wave front then moves the tail of the dye cloud forward to 1.21m by 0.6 seconds and 1.41m by 1 second. It can be observed therefore that the impact of a wave on the front of a dye cloud that has had less time to disperse is greater in terms of longitudinal motion than that on the tail of a cloud of longer residence time. In 0.6 seconds in the top frame the front of the dye is moved >0.19m and the tail of the dye in the third frame down, the 10 second scenario has moved 0.13m.

The down tank movement of dye shows vertical stratification in the no wave, 4 second, and 6 second dispersion scenarios relating to objective O4.3.2. In the no wave scenario this is best illustrated at the 1 second point with dye at the water surface being 0.19m down tank of the dye at the bed of the tank. In the 4 second scenario at the 0.6 second point the dye at the surface has moved >1.4m and at the bed is at the 1.29m point.

4.5.5 Drop Test with Constant Dye Injection

The footage gained from the constant injection scenario highlights the temporal nature of using a wave to dilute, or disturb polluted water. The system recovers to its original state 6.8 seconds after wave release.

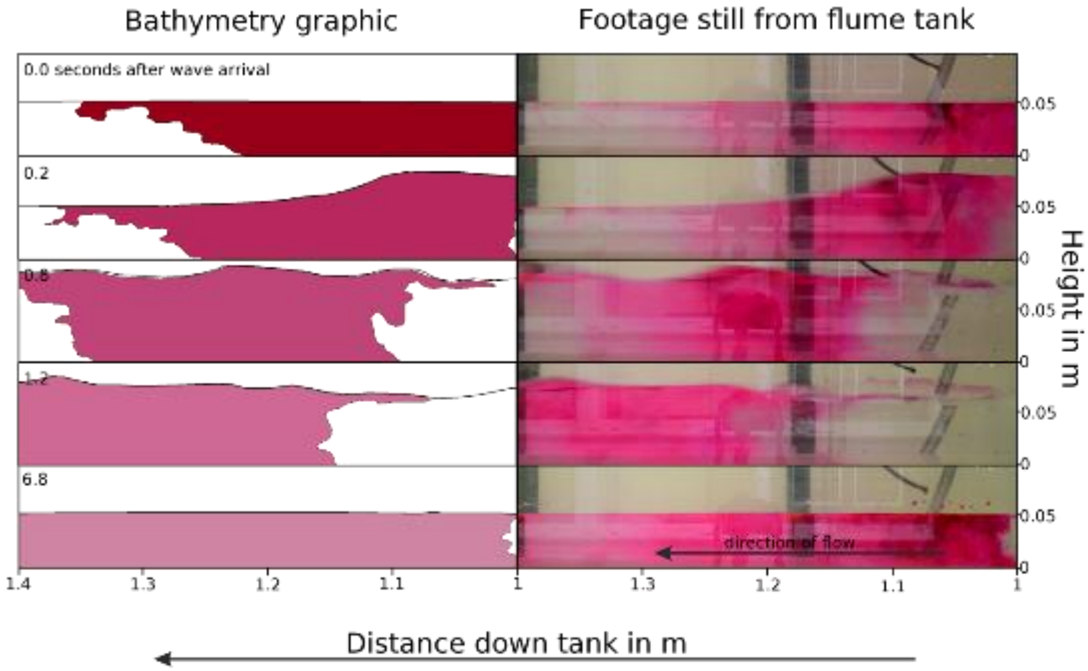


Figure 4-12 Results from the constant dye injection. The Bathymetric diagrams on the right show the progression of the dye as it is moved by the wave. The time after wave arrival on footage frame is noted in the top left corner of each panel. Time frames were chosen to illustrate the progression of the dye and the time it takes for the dye distribution to return to pre-wave arrival state.

The passage of the wave front and its ability to entrain and redistribute dye is initially comparable to the 6 second dispersion time scenario show in figure 4.11. With the arrival of the wave front in figure 4.12 stage increases from 0.05m to 0.09 after 0.2 seconds of the wave arriving on frame. At 1.2 seconds the wave has entrained all of the dye currently in the tank and moves it down tank. By 6.8 seconds, the bottom frame, the dye input has filled the frame area again, the flow as slowed sufficiently for the dye to diffuse against the direction of flow to below 1m down tank at 6.8 seconds.

4.5.6 Drop Test with Oil

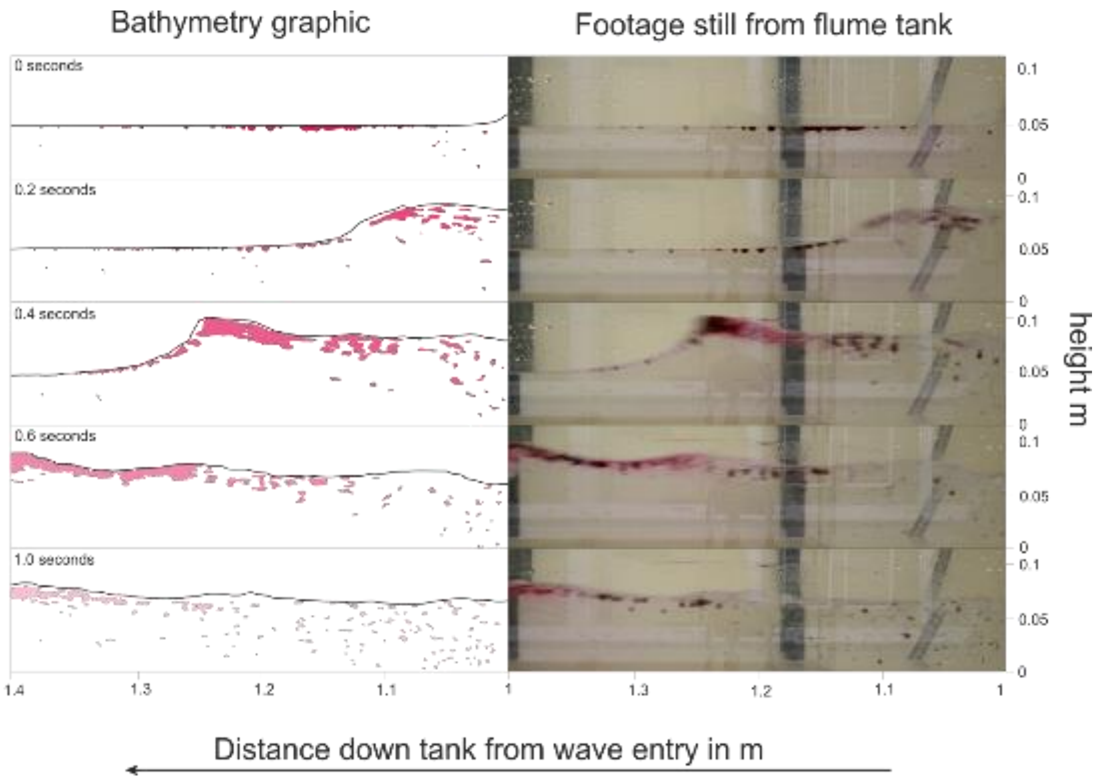


Figure 4-13 Results of 30ml of oil interacting with a normal 12.5l wave. The oil, a mixture of olive oil and rhodamine dye, appears dark red in the footage and is represented on the Bathymetric diagram on the left. The time after wave arrival is indicated in the top left corner.

Having a lower density of 800kgm$^{3-1}$ compared with waters 1000kgm$^{3-1}$ the olive oil concentrates on the surface of the water, occupying the top 0.01m of the water column, as can be seen in the 0 second frame. Some particles were caught in turbulence beneath the surface both before, and to a greater extent during and after wave passage. This was seen in the footage, as droplets would oscillate rapidly throughout the water column. This is evident in the frames for 0.4, 0.6 and 1 seconds. If defined as being particles below 0.01m deep this accounts for 15.2% of colour pixels at 0.4 seconds and increases to 37.8% by 1 second after much of the dye has left the frame. Wave passage does entrain the 60.1% of the oil within the wave front in the 0.4 second frame, with a steady volume (39%) of oil droplets being lost into the turbulence that follows the wave front. As the wave progresses a greater number of oil pixels are present in the turbulent tail of water behind the wave front with an increase of 60% between 0.4 and 0.6 seconds, and a further 13% by the 1 second frame.

4.5.7. Drop Test with Oil and a Tracking Camera

The pollution substitute diagrams above give a good account of the ability of a wave to influence various pollutant types with a standing camera. Further detail however can be gained from seeing the on-going interaction between a wave and the oil in particular. Figure 4.14 below shows the camera tracking the wave through a slug of both oil and dye. The dye gives a comparison for the oil.

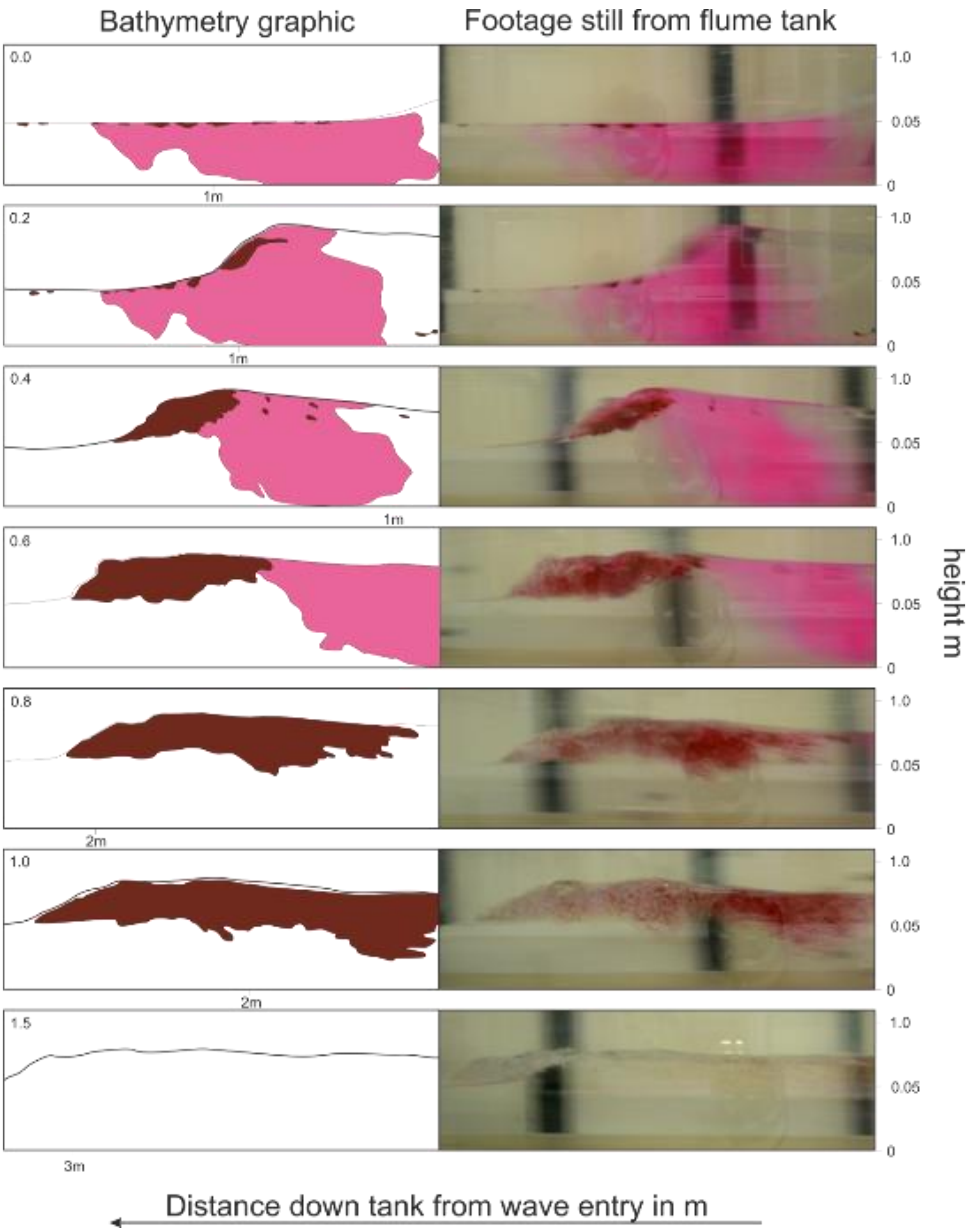


Figure 4-14 Panels through time of the camera tracking the dye and wave interaction. The oil shows up as a dark red in the video footage, and is represented in that colour on bathymetric diagram. Some dye moves from the oil into solution and appears pink. Distances down the tank are shown at 1m intervals along the base of each panel, time after wave arrival on frame are shown in the top left corner.

The pollution substitute separates into two parts in figure 4.14, with some of the dye dissolving into the water creating a pink cloud in addition to the darker oil particles.

This creates a contrast between the behaviour of the solute, and oil droplets. The oil droplets are carried in the wave front to the 2m down tank point, but are left behind by the wave front by 3m. Through the course of this down tank motion, the initially dense droplets in the 0 second frame spread over a larger area of the image. At 0.2 seconds the number of dark reddish brown pixels has increased 141.5% on that of the 0 second frame, but 0.4 there is a further increase of 271.4%. The expansion of the reddish brown pixels in the image is detailed by percentage growth or decline from the previous frame in table 4.5 below.

Table 4-5 percentage change in oil pixel count through time from the bathymetric diagrams in figure 4.13.

Time	Percentage change in oil spread from previous frame
0.0	141.6
0.2	271.5
0.4	104.7
0.6	84.9
1.0	41.2
1.5	-99.6

In the 1.5 second frame (bottom frame of figure 4.16 above) only a small quantity of oil droplets, amounting to a 99.6% reduction in brown pixels from the 1 second frame, is still carried in the wake of the wave, in the 1 second frame, at 2m down tank the oil is still very much in transit.

4.5.8 Drop Test with Kaolinite

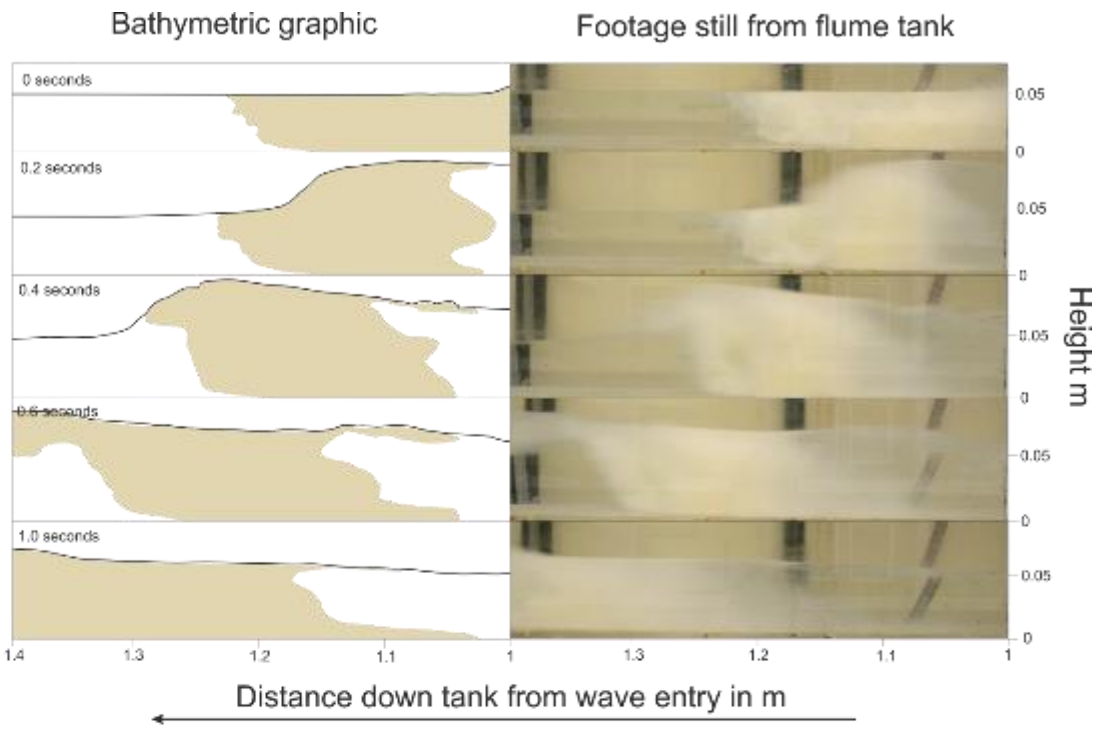


Figure 4-15 Results of 30ml of kalinite interacting with a normal 12.5l wave. The kaolinite has a pale cream colour both in the footage shown on the right and the bathymetric diagram on the right.

Kaolinite responds to the passage of the wave in a comparable fashion to the rhodamine dye. The main body of dye remains distributed though out the water column but a smaller quantity is entrained by the wave front and dragged down tank. The front of the kaolinite mass is moved >0.18m over the first 0.6 seconds. The key difference between the kaolinite and the rhodamine dye is solubility. The dye extends further than the visually identifiable coloration whereas the boarders of the insoluble kaolinite are more distinct.

4.5.9 Drop Test with Jelly

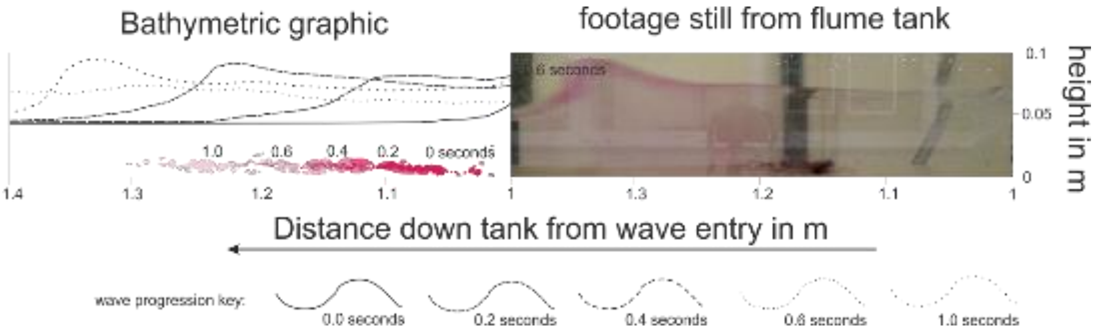


Figure 4-16 Results of 30ml of jelly interacting with a normal 12.5l wave. In the footage frame on the right the jelly can be seen in dark red at the base of the tank. Progression through time of the jelly, in shades of red and pink, and the wave front, dotted lines, are show in the bathymetric diagram on the left. The time of the footage frame is shown in the top centre left corner.

The fruit jelly is moved 0.19m, as measured from the centre of the mass to the centre of the mass, over the 1 second of dye passage. The jelly is not suspended in the water column at any point, and remains as bed load.

4.5.10 Drop Tests with Wave Regime Treatments

The four wave regimes are presented on the two figures below (figure 4.17 and 4.18). As the double peaked wave is takes a longer time period to pass the camera, and is a generally more complex picture it has been presented on a separate diagram, figure 4.18. Figure 4.17 shows a ‘normal’ 12.5l wave released in one instantaneous slug, and two longer waves, one of 12.5l released though a 0.01m gap and one released though a 0.005m gap as described in the method. Furthermore the following two diagrams have been designed to illustrate the effect of having a longer wave length, and so capture a longer period of time than those shown above, with the frames being 0 seconds after wave release, 0.6 and 2 seconds.

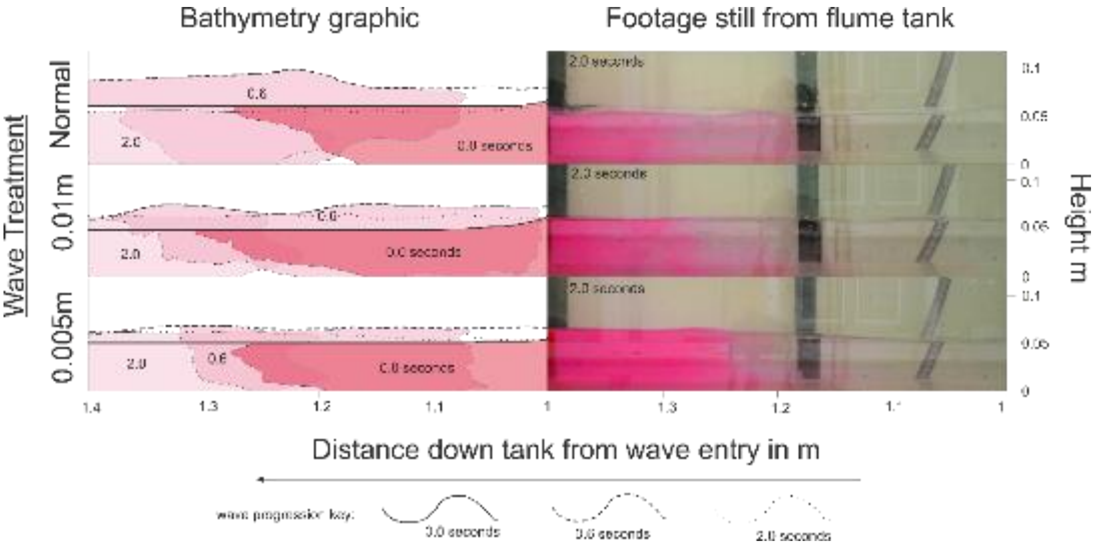


Figure 4-17 The effect of three wave magnitude treatments upon 1ml of dye. The dye was allowed to disperse 4 seconds before wave release. All times on the graphic, top left in the footage frame, are given from the point of wave arrival. The bathymetric diagrams detail the dye and wave progression over three time steps.

In figure 4.17 the footage still used is set at 2 seconds after the arrival of the wave front to illustrate the key result of this experiment. Each of the wave heights; the normal release that generates a wave height of 0.1m, the 0.01m gap wave that generates a wave height of 0.07m and the 0.005m wave that generates a wave height of 0.06m, achieves a similar result in so far as moving the dye off frame with the ‘normal’ wave taking dye off frame by 0.6 seconds and the 0.01m and 0.005m waves progressing the dye 1.37 and 1.32m respectively. The effect of these different wave regimes on the front of a pollutant substitute is described in table 4.6. This illustrates that a greater wave amplitude has a greater effect on the forward motion of the dye. The normal wave leaves the tail end of the dye at 1.2m, the 0.01m gap leaves it at 1.21m and the 0.005m at 1.19m. Having a lower amplitude, longer wave length wave does not have a significant visually discernible effect on the tail of the dye slug. Given the negligible difference between the wave treatment effects on the tail of the dye, the high amplitude wave can be said to have had the greatest impact upon dye motion.

4.5.11 Drop Test with Two Peaked Wave Treatment

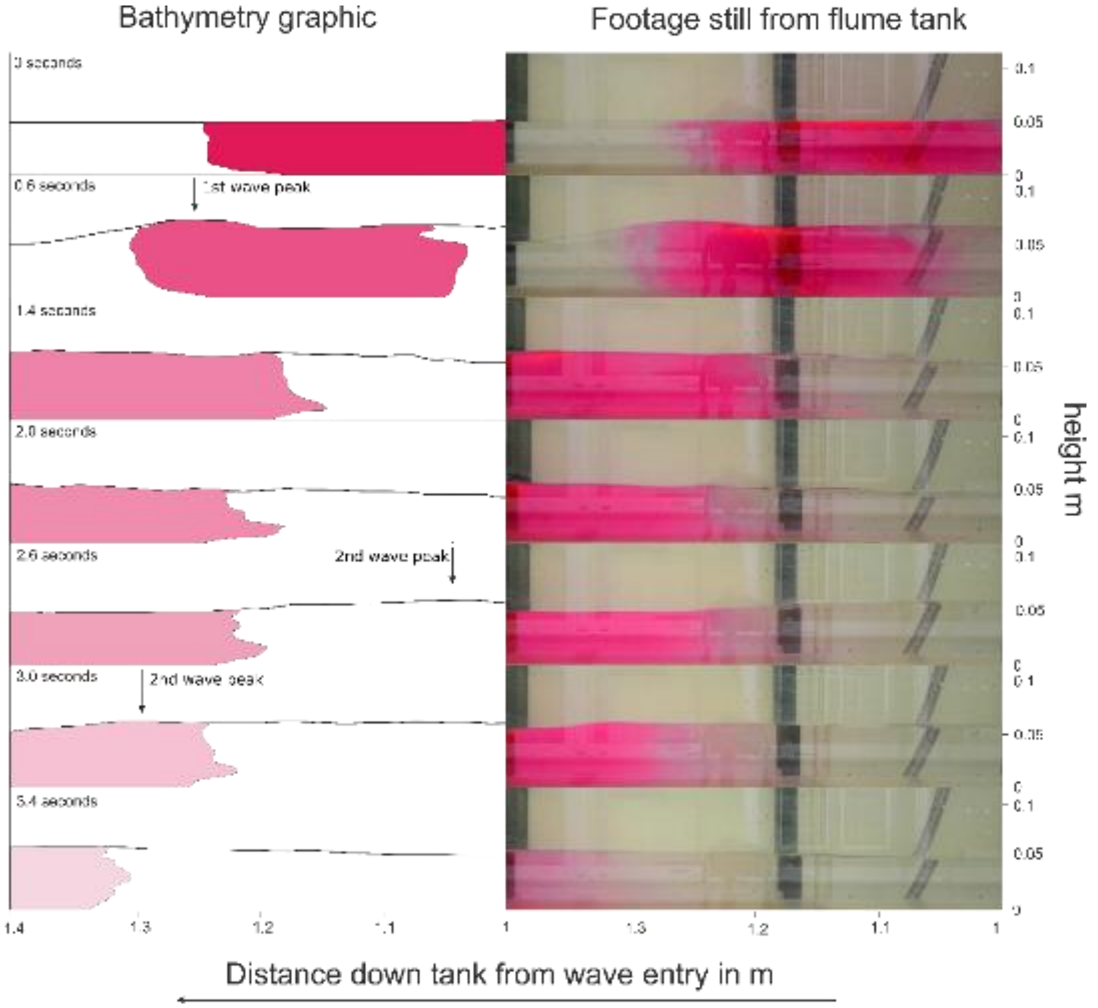


Figure 4-18 The effect of the two peaked wave on 1ml of dye. The dye was allowed 4 second s of dispersion time prior to wave arrival. The time step for each panel is given in the top left corner.

The two peak wave achieves a similar result to the three regimes presented in figure 4.17 at the 2 second mark, the back end of the dye mass is at the 1.2m mark. The second wave peak arrives at 2.6 seconds. Over this time period the dye, under the influence of the 0.035ms-1 baseflow has barely moved. Over the next 0.8 seconds the second wave peak is able to move the dye a further 0.12m down tank.

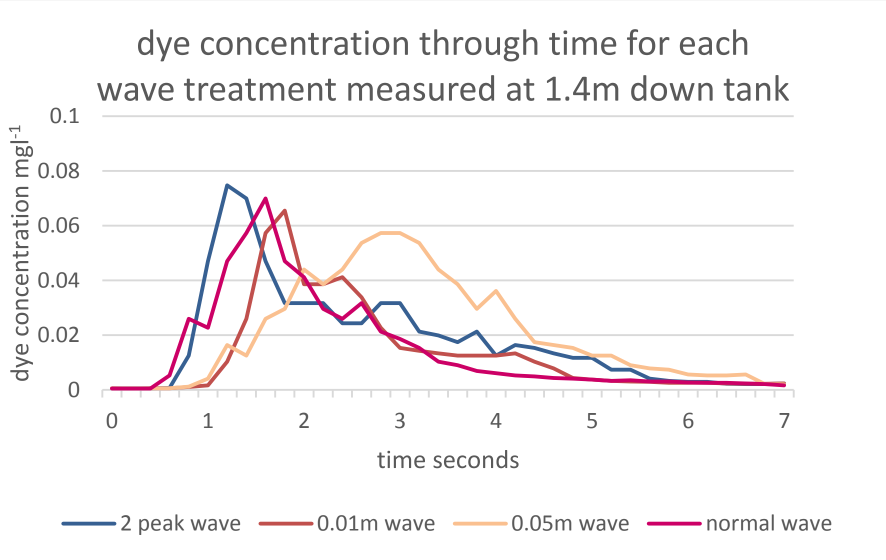


Figure 4-19Time series lines for each wave treatment of dye concentration through wave passage. 0 seconds in the time line represents the arrival of the wave front on camera frame.

At 1.4m the dye concentration through time for three of the wave treatments is similar with the normal wave, 2 peak wave and 0.01m wave all producing a similar peak dye concentration and longitudinal dispersion. Both the normal wave and 0.01m wave produced a total longitudinal dispersion of 4.4 seconds, the 2 peak wave 5 seconds and the 0.005m wave 6.2 seconds. In practice this shows that at 1.4m down tank the results are very similar with the 0.05m being the exception. The 0.05m produced a greater longitudinal dispersion and lower peak concentration at $0.057mgl^{-1}$. It could be argued that this wave generates the greatest dilution. However there the 0.005m wave is slower to transfer the dye giving the cloud ~2 seconds longer than the other treatments to undergo diffusion.

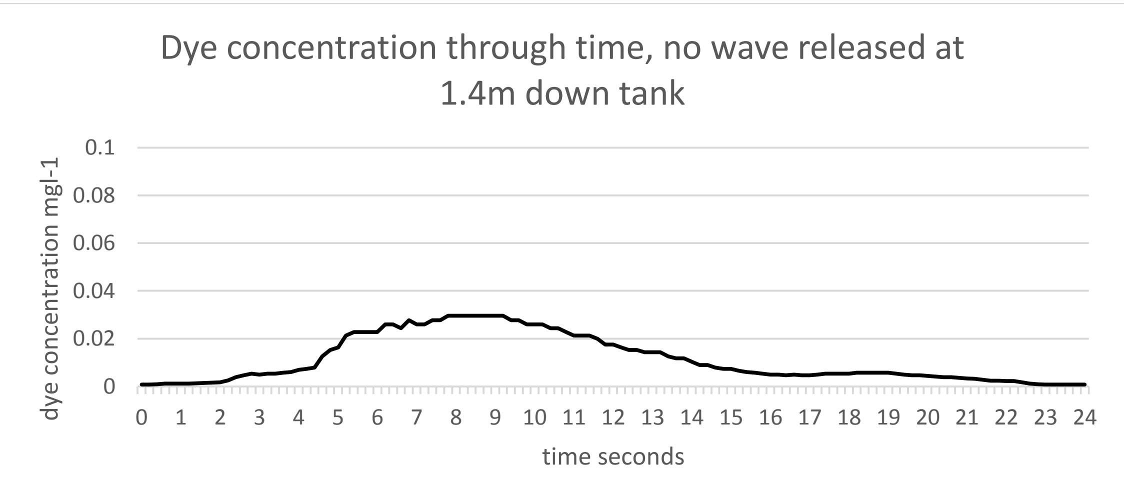


Figure 4-20 Time series of dye concentration through time for the no wave scenario.

In the absence of a wave the dye has a longitudinal dispersion of 20 seconds and a peak concentration of $0.29mgl^{-1}$. This dispersion is considerably longer than any of the no wave scenarios, though it does come with a few limitations. Firstly as mentioned in the limitations section of the method when the wave reaches the lip at the far end of the tank a small refraction wave (with a peak of ~0.01m) is sent back down the tank. This refraction wave possesses enough energy to stall the movement of the dye. Secondly without a wave release the eddies and secondary flows generated by the hose inputting the baseflow persist in entraining dye. These eddies are arguably a feature of realistic flows in rivers, this point will be expanded upon in the discussion.

4.5.12 Summary Tables for the Wave Regime Treatments

The Froude numbers based on UDVP data (table 4-5) are all subcritical, that is water particle velocity is lower than wave celerity, with the highest value being 0.492 for the 1st peak of the 2 peak wave scenario. The Froude numbers based on the wave progression velocities are in the case of the normal wave super critical, and in all others sub critical but far closer to 1. The difference between the velocities is in part the result of the vertical distribution of the UDCP probes, as described in the method section, the probes occupied the bottom 0.05m of the water column and do not account for the overtopping water shown in footage such as that shown in figure 4.16. The UDVP probe data measures the velocity of water from within the baseflow water column rather than the water flowing over it.

Table 4-6 details the mean (µχ), velocity (V) of the baseflow, the UDVP output during wave passage, the wave velocity as calculated from the footage, and Froude numbers derived from the numbers.

Treatment;	µχ V baseflow (ms^{-1})	µχ V wave vertical profile, UDVP (ms^{-1})	Froude number based on UDVP	V wave from wave progression footage (ms^{-1})	Froude number based on footage	Discharge at the peak m^3s^{-1}
Normal Wave	0.035	0.505	0.449	1.266	1.126	0.0068
0.01 m wave	0.035	0.255	0.292	0.88	1.006	0.0027
0.005 m wave	0.035	0.19	0.209	0.875	0.964	0.0019
2 peak wave, 1st peak	0.035	0.452	0.492	0.92	1.002	0.005
2 peak wave, 2nd peak	0.035	0.264	0.300	0.85	0.966	0.0024
no wave	0.035	n/a	n/a	n/a	n/a	0.00027

In figures 4.17 and 4.18 the effect of different wave regimes on the back of the substitute pollutant, the rhodamine dye is displayed. Table 4-6 below both details descriptors of these waves such as wave duration, and rising and falling limb times but also quantifies the effect of these waves on the front of a substitute pollutant. This data is based on kaolinite rather than rhodamine because the arrival time is more distinguishable for an insoluble pollutant.

The normal wave takes 4.16 seconds to propel the kaolinite to the 2m point being 3.26 seconds faster than the 0.01m, 3.39 seconds faster than the two peak wave and 4.89 faster than the 0.005m wave. The baseflow takes a full 19.24 seconds to achieve the same condition. The normal wave is least dependent on the baseflow to move the kaolinite. The wave lengths/durations are substantially longer with the lower amplitude waves.

Table 4-7 Summary kaolinite progress down tank with different wave treatments.

Treatment;	wave duration (s)	rising limb duration (s)	falling limb duration (s)	Kaolinite arrival at 2m point (s)
Normal Wave	1.50	0.08	1.42	4.16
0.01 m wave	4.19	0.2	3.99	7.42
0.005 m wave	6.30	0.18	6.13	9.05
2 peak wave, 1st peak	1.28	0.14	0.99	7.55
2 peak wave, 2nd peak	0.93	0.14	0.78	7.55
no wave	n/a	n/a	n/a	19.24

4.5.13 UDVP Velocity Data for Wave Treatments

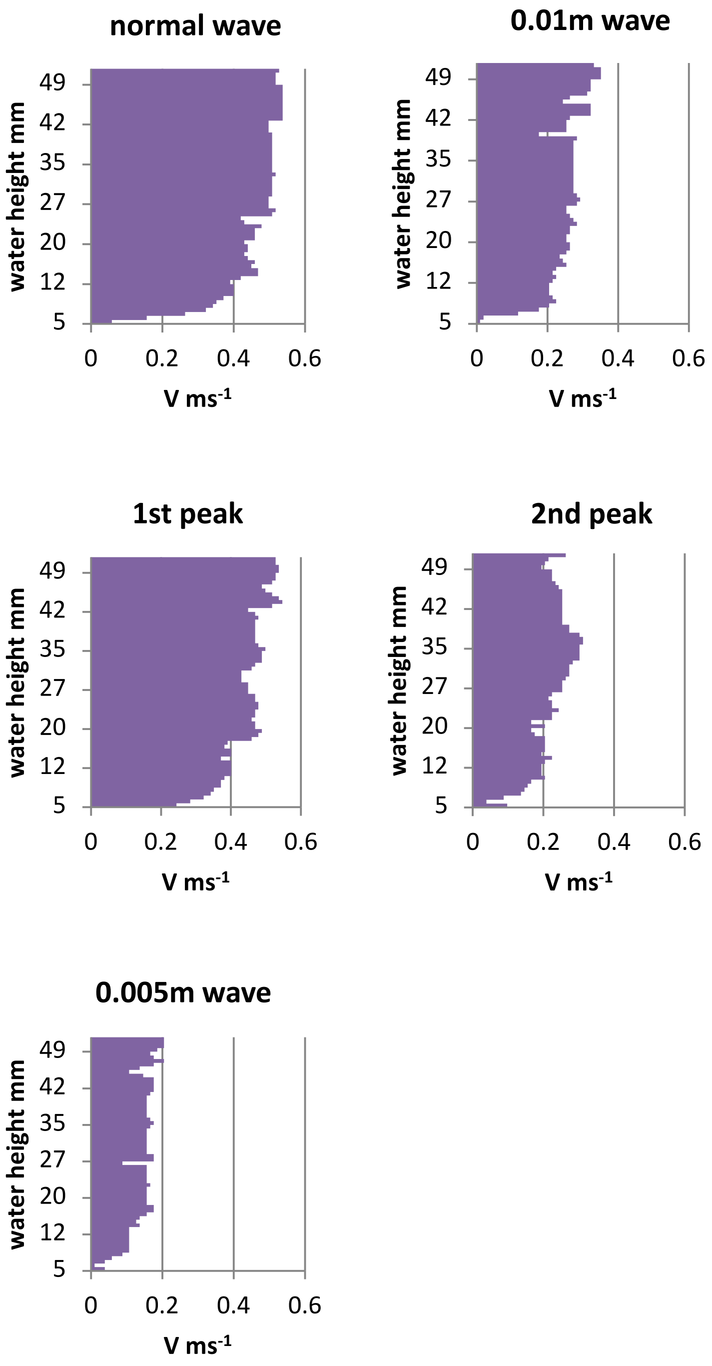


Figure 4-21 Bar graphs of velocity in ms^{-1} against water height at the time of the wave peak arrival. The 1st and 2nd peak are the consecutive peaks of the 2 peak wave.

The velocity profiles for the five wave treatments clearly show lower velocities near the bed. The up curve in velocity with height is not smooth in any of the 5 scenarios with dips in velocity at heights such as 27mm and 45mm in the 0.005m treatment, the 25mm height in the normal treatment, and 38mm in the 0.01m treatment.

4.4.14 Turbulent Intensity

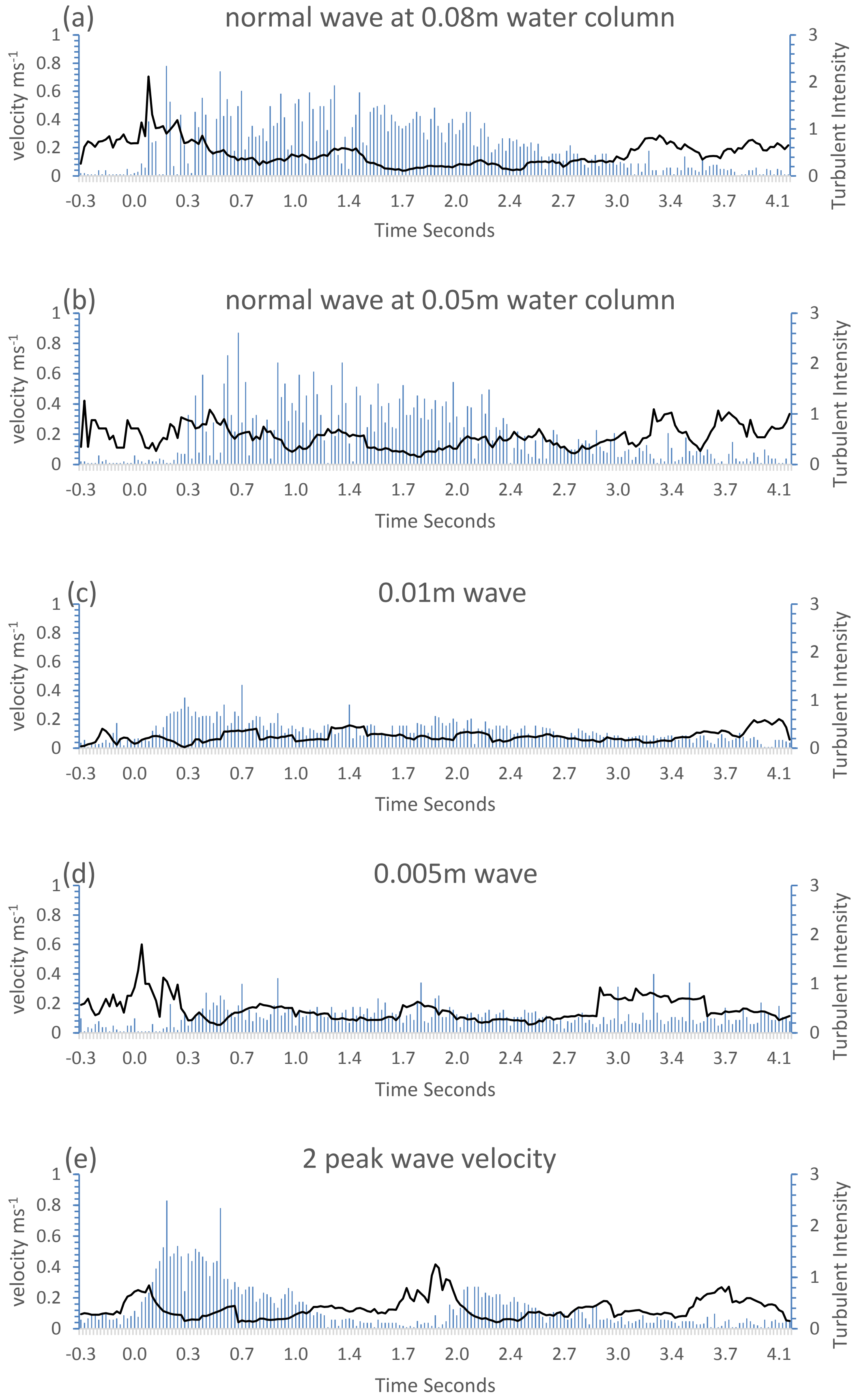


Figure 4.22 Turbulent Intensity and velocity time series for each wave treatment. 0.0 seconds is the time when the wave was released, all measurements were

taken at 0.05m stage with the exception of frame (a) at 0.08m. The black line represents turbulent Intensity and the blue bars velocity.

Turbulent Intensity increases with the arrival of the wave front to just over 1 under each of the treatments and briefly exceeds 2 when measured at 0.08m in the water column. However once the wave front has past turbulent intensity reduces and only again increases when flow returns to low velocities. This suggests that the more uniform velocities created by the wave release reduces turbulent intensity. Turbulent Intensity is heavily influenced by the time period assigned to the moving Reynolds Average and the RMS fluctuations. This is most notable when examining the arrival of the wave front. Using a longer time period effect of the wave arrival will be seen in an increase in turbulent intensity before the wave arrives as the moving average to RMS ratio will skewed by a few high velocities. To counter this the time period was reduced from 1 second to 0.5 for the period immediately preceding wave arrival. If this moving average were to be reduced or increased the resulting turbulent intensity would change, as the calculation is to an extent a function of the time period used.

4.6 Discussion

The key aim of this chapter was to identify mixing process when a wave of water meets a pollutant in a river and the physical parameters that determine them; aim A4.3 from the QAM in chapter 1. Testing varied scenarios (A4.4), and measuring and comparing baseflow and wave velocities (A4.1) are also dealt with in the data presented in the results section but are of secondary focus. The discussion presented here will approach each of these aims with A4. and A4.1 first, and then A4.4.

4.6.1 The kinematic nature of the wave

The lag effect phenomenon described by Glover and Johnson (1974), and Heidel (1966b) indicates a separation between the movement of a flood wave, as defined by the rise and fall in river discharge and the downstream movement of water within the river. When considering using a wave of water to alter river chemistry or dilute pollution, some logical consequences of this lag effect must be considered. If there is a separation between the water released, and the water in the wave that reaches a pollution slug, does the quality of the reservoir water matter? How significant is the quality of the water between the reservoir and the polluted water? The answer to this question will be determined by a measure of how much water in the wave was derived from these two sources. Secondly, if the wave motion is a movement of energy it is important to understand what will happen when this kinematic motion moves beyond the polluted water. The discharge and stage will return to baseflow levels whether this will allow the polluted water re-concentrate or whether the increase in longitudinal dispersion will be sufficient to retain dilution needs to be established.

Bulk tests in standing ambient water clearly demonstrate a rapid separation between the released water and the propagation of the wave . The notion that a wave is primarily a transfer of energy is not a new one (Stokes 1847; Longuet-Higgins 1953), with the deeper ambient water low amplitude experiments behaving in a similar manner to an oceanic waves. The three ambient water depths in figure 4.7 do not show an identifiable linear relationship between dye progression and wave progression. There is a transition between a shallow water wave scenario, where in the wave motion is inhibited by the tank bottom, and a deep water scenario which occurs between 0.1 and 0.15m of depth, fitting with the wider observations on the behaviour of oceanic waves (Davis and Acrivos 1967; Galvin 1972). Such waves are not relevant to the situation in rivers. The more complex shallow waves in flowing water, better approximate the riverine environment.

Both the motion of the dye in the standing camera and tacking camera drop experiments show a more complex result. The dye is still left behind by the wave, and as the UVDP data demonstrates, the water velocities are considerably slower than the wave celerity, but some dye particles are entrained and move with the wave down tank. The wave is kinematic in the sense used by some hydrologists (Glover and Johnson 1974) that is effective in transferring its energy, and thus relative rise in discharge to catch every pollution substitute dropped into the tank. This gives a clear measurement for A1 and shows that in a 5m tank, a wave can catch the polluted water.

The Kinematic wave is often (Ponce, 1991) given as;

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = cqL$$

Equation 4.2

Where *Q*=discharge; *c*=kinematic wave celerity, *x*= spatial variable; a *t* = time, *qL* cross sectional inflow. This equation whilst giving differentiations of discharge, time treats the wave celerity and cross sectional inflow, c and qL respectively, as constants. Indeed Ponce (1991) notes that the kinematic wave theory is limited in its ability to describe diffusive waves, that is waves losing energy. The variable progression of the dye, and the vertical stratification of dye motion demonstrate that whilst the wave has kinematic properties, the kinematic wave theory cannot fully account for its behaviour.

4.6.2 Mixing in the vertical water column and the role of edge effects

Vertical stratification is clear in the motion of the dye both in the flowing bulk test and in the drop tests the same can be seen with the kaolinite. These results support Beer and Young’s (1983) model of the river channel as having a ‘dead’ mixing zone, and a central faster moving zone. The area near the bed exerts considerable drag upon flow and dye. The wave itself acts as an accelerator to the main flow zone as it both increases particle velocity and raises stage. These in turn have an impact on longitudinal dispersion which is discussed further in a later section.

4.6.3 Mixing in the vertical water column, and wave theory

Whilst models of riverine mixing line up well with the vertical stratification of dye, literature concerning this topic does not explicitly cover wave motions. It is therefore worth attempting to set the findings of this chapter against publications in the field of wave motion and mechanics.

With the exception of the waves in deeper ambient water seen in figure 4.7 all of the waves reported can be considered long waves as their wave length is greater than the mean water depth (Stoker 1958). The majority of papers concerned with long waves in shallow water are limited to mathematical descriptions of wave propagation, in particular acceleration, diffusivity and changes in wave form and amplitude (Kadomtsev and Petviashvili 1970; Hirota 1973; Constantin and Escher 1998; Constantin 2000). It is therefore difficult to draw comparisons with the experimental data concerned primarily with mixing processes reported here. Given the abstraction of using a flume tank over a flood wave in a river there is limited value in this chapter providing a detailed discussion of changes in wave amplitude over time.

When a wave bulk is dropped into a tank of flowing water the dyed water (figure 4.9) is forced over the top of the baseflow water. This stark division of the wave and baseflow waters is not described in any of the literature mentioned thus far. Furthermore research concerned with the pathways of individual particles in a wave (Pierson 1962; Chen *et al.* 2012) also do not deal with this phenomenon. Longuet-Higgins (1960) does describe a jet of higher velocities near both the bed and surface boundary during wave passage, but this description again does not match the visual evidence presented in figure 4.8. Experimental data from flumes (Iwagaki and Sakai 1970; Madsen 1971; Svendsen and Staub 1981; Swan 1990; Weber 2011) or numerical flumes, that is a CFDM simulating a flume tank, such as (Dong and Huang 2004) also do not describe this division. The reason for this omission is that they are considering different scenarios. Swan (1990) and Chen *et al.* (2012) are both concerned with waves moving up a slope, such as a beach. Dong and Huang (2004) are concerned with waves generated via a moving plate or piston rather than a gravity wave generated via water input. Longuet-Higgins (1960) is concerned with waves in ambient water. The study of bore waves, or dam break scenarios in with CFDM methodologies do account for water release induced waves, but papers in this field focus on the study of the water surface and volumetric movements rather than mixing, or processes within the water column (Mingham and Causon 1998; Fagherazzi *et al.* 2004).

The wave generation method is probably the key distinction here. A wave generated via a moving plate or piston is not introducing external water to the system and is simply influencing the water already present. In addition to this the wave generation method evenly affects particles across the whole water column. Dropping water over a lip from above on to a water mass could give a bias in forcing the water over the top. If the wave water had instead been introduced through a valve in the bottom of the tank the result could potentially be quite different. A second factor is the turbulence of the flow in the tank. The introduction of baseflow into the tank through a hose at an acute angle to the tank bed creates turbulence in the tank flow, coupled with this releasing a bulk of water into the tank to produce a wave creates more turbulence. Given the straight tank designs with a single uniform flow input described in Swan (1990), Iwagaki and Sakai (1970), and Svendsen and Staub (1981) and the common use of standing water within the literature it would appear that turbulent scenarios are not often considered.

Whilst the separation of particles might be absent from the literature Iwagaki and Sakai (1970) and Svendsen and Staub (1981) both describe the horizontal profile of particle velocities at the point of a wave crest. The profile for the hyperbolic wave velocities reported by Iwagaki and Sakai is comparable to that of the dye or pollutant substitute advancement seen in figures (4.10, 4.12, 4.14 and 4.15). The authors go on to suggest that such a wave form is better explained by the theory of cnoidal wave theory (Wiegel 1960) rather than Stokes wave theory (Stokes 1847).

4.6.4 Longitudinal Mixing

If a wave increases the rate of longitudinal dispersion of a pollutant, it will increase the longitudinal mixing and therefore dilution. In Figure 4.11 the 0.16m longitudinal dispersion of the no wave scenario can be contrasted with the 0.9m progression in the tracking experiment shown in figure 4.10 or the 1.2m progression of the dyed oil in figure 4.15. The pollutant concentration in each of these scenarios feature a dense leading slug of pollution substitute with a tail, fitting the description given by other authors of typical pollution concentration curves in the riverine environment (Czernuszenko *et al.* 1998). Of the papers that address longitudinal mixing or dispersion, and in particular those dealing with the dead zone model a significant proportion give minimal attention to transient flow conditions (Nordin and Troutman 1980; Beer and Young 1983; Nepf *et al.* 1997; Czernuszenko *et al.* 1998; Deng *et al.* 2001; Schmid 2002; Seo and Baek 2004; De Smedt *et al.* 2005; Hunt 2006; Tealdi *et al.* 2010). Some papers do refer to variations in hydraulic inputs, but in the abstract as a range of values, rather than a time series of flow changes describing a wave (Tayfur and Singh 2005).

Mechanical observations of the footage show that the dye is driven down tank in the upper water column at an increased rate of advection, that is particles driven by the motion of the current, or velocity as per Taylor (1953). The process of Fickian diffusion then causes particles to be removed to the dead zone at a steady rate generating a spread of pollution substitute particles down the water course. Furthermore, increases in turbulence increase vertical diffusivity (Nepf *et al.* 1997), suggesting that the more turbulent the wave motion, the greater the transfer of pollution particles between the main flow and dead zones as visible in the circular particle motions in figure 4.14. This processes is present in the experiments with oil, which despite its lower density, is drawn into the lower water column in the tail of the wave, as is the kaolinite. By contrast the fruit jelly, which has no vertical stratification in its distribution, remains in the dead zone. This is unsurprising given the lower velocities and higher Reynolds numbers experienced near a boundary (Sychev *et al.* 1998). There is a conflict between the visual descriptive results and the numerical metrics presented. The graphical summary of longitudinal dispersion for each of the wave treatments and the no wave scenario presented in figures 4.19 and 4.20 suggest that in actuality the no wave, or the 0.005m wave treatments produce the greatest longitudinal dispersion, and lowest peak concentration, and thus the greatest dilution. Second, the turbulent intensity data (figure 4.22) suggests that the flow was more turbulent before and after the wave. Dispersion varies with time and space, dispersion through time was measured at 1.4m down tank over a time frame of 7 seconds. If these measurements were taken at the 5m point over a longer period of time the result could be different. The key question is whether the wave reduces to the transfer of dye between the upper water column and the dead zone as Graf. (1995) suggests. If the wave does reduce the diffusion and vertical mixing then the increased spread of dye across the upper water column maybe be insignificant in terms of increasing dispersion in the long term, and certainly in the short term.

With the exception of a brief period during wave arrival, turbulent intensity appeared to reduce during increased flow. The likely explanation is the unidirectional nature of the flow during the wave passage. In the absence of the wave release, the hose baseflow input baseflow controlled flow conditions and generated a series of eddies over the first 1m or so of the tank, as the water entered under pressure. The introduction of the wave reduced the impact of these eddies and lowered turbulence intensity. This has a significant impact on vertical mixing, as these eddies did entrain dye and cycle it up and down the water column. Whilst an explanation has been given, this result is still surprising it as it is counter to the findings of Reynold’s original experiments on turbulent flow (see Chadwick *et al.* (2004) for a description). Reynolds demonstrated an increase in the ratio between inertial and viscous forces with a rise in discharge.

4.6.5 The effect of Wave Treatment on Longitudinal Mixing

If it is assumed that the propelling the dye down the tank with the wave front will, given time, allow for a greater spread of dye and then diffusion through the water column, then which wave treatment would best produce this for given volume of water spent? In this study the waves with a lower amplitude seen in figures 4.18 and 4.17 both provide far more limited increases in particle velocity in table 4-5, and have a more limited impact on dye motion than the ‘normal’ single peak high magnitude wave treatment. Whilst these waves are longer this does not appear to compensate for the initially lower particle velocities. The key difference between these waves is shown in table 4-5. The peak of the normal wave can be considered to be supercritical flow, that is the particles in this area of the wave (figure 4.20 gives a diagrammatic explanation) are moving at a velocity greater or equal to that of the wave motion, in Froude numbers this is expressed as >1(Chow (1959). Such numbers are reported in table 4-5 for wave celerity. Given that that dye and oil particles are entrained and moved with the wave for a limited distance, these particles can be classed as supercritical for a short period. It is this area of supercritical flow that is responsible for the majority of the substitute pollutant that is entrained and moved down tank and consequently the increase in potential longitudinal dispersion. This is visible in the figures 4.10 and 4.15 the tracking experiments in particular. In simpler terms, pollution particles that have accelerated to the velocity of the wave front will move with it. The object of any flow manipulation exercise with the intention of moving pollution particles down river, should therefore be to induce supercritical flow conditions within the river. The friction of the bed and the banks, in creating dead zone low flow areas may then remove particles from this supercritical zone inducing dilution. A clear result for objective O4.3.1 cannot be given even though varied wave treatments were tested, as the longitudinal dispersion results presented suggest that no wave is preferable, as has been discussed. The notion that the normal wave treatment is the best option for increasing longitudinal dispersion can be inferred but without a longer tank and a rough bed this only conjecture. The relationship between flow velocity and sediment transport is well established (Govers 1985; Van Rijn *et al.* 1993), additionally equations that estimate longitudinal mixing use mean particle velocity as an essential function (Hunt 2006), the result presented here is of detail. The mean velocity of the water column is limited in a wave scenario where the vertical stratification of flow is so great.

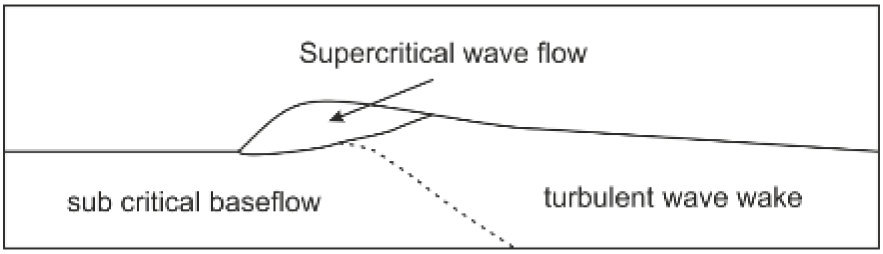


Figure 4-22 A conceptual diagram dividing the normal wave scenario into its supercritical wave peak, sub critical base flow and turbulent wake. The wave is flowing from right to left.

A few papers have used Froude numbers as a basis for estimating the transport and mixing of matter, either suspended (Aguirre-Pe *et al.* 2003) or solute (Besio *et al.* 2012). Besio *et al.* (2012) considered water depth to be a more significant factor in determining solute transport across a compound channel but both papers record a positive relationship. Many papers concerned with either mixing of solutes (Guymer 1998; Boxall and Guymer 2003) or transport of sediment (Pickup and Higgins 1979) are not concerned with Froude numbers. This is largely due to the rarity of supercritical conditions within rivers. Supercritical conditions are however common in dam break scenarios (Pritchard and Hogg 2002), which have been associated with significant sediment movements. A lag time between the leading wave peak, and peak suspended sediment concentrations in transport has been identified in such flows (Carrivick *et al.* 2011).

4.6.6 The Behaviour of Oil Within the Water Column

The behaviour of solutes, the dye, and suspended sediments, the kaolinite, has been discussed. Some extra comment on the oil is necessary, as this pollutant being less dense than water differs in behaviour to the more dense substances. Most of the literature concerning oil base pollutants in water is focused on oil spills in the marine environment (see Reed *et al.* (1999) or Spaulding (1988) for reviews). Papers typically study a scenario with either deep water, and low current (Alofs and Reisbig 1972; Boufadel *et al.* 2007) or breaking waves on a beach (Tkalich and Chan 2002). None of these situations are analogous to the shallow water, unidirectional current, long wave scenario that this study is approaching. However, some of the results of this study; those depicted in figures 4.13 and 4.16, for example, do match with descriptions in the literature. Oil slicks, or masses under the influence of turbulence and wave action break into droplets which are then penetrate the water surface and are further broken down into smaller droplets as they mix with the water column (Alofs and Reisbig 1972; Delvigne and Sweeney 1988; Boxall and Guymer 2003). The difference arises with the direction of flow. In figures 4.13 and 4.16 these droplets are distributed rapidly down a tank. Boufadel *et al.* (2007) in a numerical deep water tank simulation using wave trains describes a much slower motion of particles being driven by stokes drift and turbulent diffusion, processes that whilst present in the results reported here are secondary to the kinematic motion of the wave and the high in tank velocities.

Release waves in the river channel are an order of magnitude longer than those in the tank and moving at a similar velocity (more detail is given on this in section 4.6.7 below), it can therefore be suggested that whilst in river waves might outpace pollution it would take considerably longer in both time and longitudinal distance.

4.6.7 Applicability to the Real River

Flume tank results are meaningless unless they are comparable to the real world situation. This topic will be discussed in greater detail in chapter 6, as this is where the findings of the three results chapters are drawn together to give a complete picture and a detailed application to management. However, since it has a bearing on the design of the following CFDM chapter it is worth outlining at this stage.

There are questions of scale; scale of flows, and scale of time to consider. The waves generated within the flume typically took around 5 seconds to cover the 5m of tank (table 4-5). This progression speed of in the order of $1ms^{-1}$ is comparable to those reported in figure 18 of chapter 4 describing wave speeds within the Holme river. However the scale of the tank is several orders of magnitude smaller than that of the river. The 5 seconds of wave progression time are insufficient for particle processes in particular diffusion to take place. If 1ml of dye is dropped into the flowing tank with no wave (figure 4.16) after six seconds it has dispersed over less than 1m. In the river 6 seconds is not a meaningful timescale for a management orientated study. Hydrologists typically work in timescales of 15minutes through hours, days, or even weeks. The data reported in this chapter is all at the millisecond and second timescale.

Chapter 5 examines a CFDM model designed to be 100m+ long to address questions over scale.

4.7 Conclusion

Using a flume tank to investigate mixing within the water column has allowed a clear visualisation of the processes that occur in a scaled down idealised situation. The bulk tests in standing water showed that a gravity wave is primarily of transmission of energy rather than matter as the dyed water was left behind by the wave. Reproducing this test in deeper water produced a lower amplitude wave that outpaced the dye sooner. This result suggests that a wave released from a reservoir down a river would to an extent outpace the reservoir water and produce a mixture dominated by riverine source water. This may reduce the impact of low quality reservoir waters on dilution. In flowing water a clear vertical stratification was seen, this result was also seen in the drop tests. By using both a visual dye tracer and UDVP measurements it has been shown that the wave was significantly faster than the baseflow, allowing the wave to catch any substitute pollutants dropped into the tank and eventually pass through them. In this way both aims A4.1 and A4.2 are met. The treatment of the term kinematic wave has been given a more detailed definition in the mathematical papers than the hydrological referenced in this chapter. It is clear from the bulk tests that the movement of the wave down the tank is primarily an energy transmission and so can be termed kinematic.

Vertical stratification of flow was evident in the velocity data recorded and the distribution of the dye in the drop tests answering O4.2.2. This vertical stratification of flow was largely driven by the wave front with dye being dragged over the top of the water column. Whether this process would in fact increase dilution and longitudinal dispersion remains conjecture. Numerical results extracted from the 1.4m down tank position suggest that no wave actually produces greater dispersion and a lower peak concentration though over a much longer time period. At 1.4m this result has limited meaning as diffusion, both molecular and turbulent has not had time to take effect and therefore it is not surprising that the slower the wave the greater the dispersion. Without a clear indication as to which wave treatment produces the greatest dilution, or whether waves are beneficial for longitudinal dispersion this success of this chapter must be viewed as limited. Testing varied forms of pollutant was more successful than the wave treatments. An oil based pollutant was mixed into the water column and dispersed and a dense bed load pollutant was almost unaffected by the wave giving some value to aim A4.3. Aside from these issues the scale and smooth bed substrate of the flume make it a questionable comparator to a real river, to counter these problems a computer fluid dynamics model (CFDM) was constructed to replicate the flume tank experiments at a much larger scale. This is detailed in chapter 5.

Chapter 5. Computer Fluid Dynamics Approach

5.1 Introduction

The experiments in chapter 4 demonstrated the kinematic nature of a solitary dam-break wave moving down a channel. It analysed the interaction of this wave with a pollutant as represented by a dye in terms of longitudinal dispersion and thereby dilution. A critical limitation of these experiments was the scale, a 5m flume tank was a considerable downsize from a real river system. To further investigate and validate the experimental results, this chapter presents a numerical model to simulate a flume tank of >100m in length. Waves were sent down this tank to interact with tracked particles. Using a scale representation of the river makes it practical to release waves with a magnitude comparable to those seen in chapter 3. The effects of different wave magnitudes and durations upon longitudinal dispersion are quantified within this chapter. This chapter begins with a review of different numerical modelling approaches dealing with flow changes in open channels. It then follows with a methodological description of the model employed by this research and a display and discussion of the results before concluding with a discussion of the main findings.

5.2 Literature

5.2.1 1D Modelling approaches

Popular one-dimensional (1D) flood modelling software such as MIKE-11, ISIS 1D and Hec-RAS predict flow levels and wave propagation within a channel using differential conservation of mass equations such as the Saint -Venant equation (de Saint-Venant 1871; Gerbeau and Perthame 2001). These Saint Venant equations are solved at river cross sections distributed down the river channel. In this way 1D models have treated the river as a series of vertical slices with a distance between them. This means that conceptually the flow is understood purely in terms of volume and momentum with forces such as friction acting upon it. These 1D models are therefore commonly used for modelling flood waves and estimating the inundation of land around the river channel, which is their primary purpose, rather than for water quality problems (Cox 2003). However, both Hec-RAS, and ISIS have water quality models built in, and other water quality models, such as QUAL2E, have been integrated with the hydrological output from Hec-RAS and ISIS (Fan *et al.* 2009). ISIS Water Quality solves the finite difference approximation of the 1D advection diffusion equation (Lopes *et al.* 2004). The body of water at any given time is treated as a single volume into which a pollutant can be dispersed into at a set rate. This water quality is then transferred down river with the water mass. As hydraulic models such as Hec-RAS, ISIS and MIKE-11 are all based on the conservation of mass in the abstract, they are unable to calculate the mixing of distinct water masses.

The waves demonstrated in the flume tank in chapter 4 were of a kinematic nature; that is to say that the water that reached the far end of the tank contained minimal dye and was made up of a different water mass than that which was released at the other end of the tank. Additionally results from chapter 4 showed vertical stratification in flow velocities with dye moving down tank at a rapid rate near the surface. Whilst the three hydraulic models named all employ a kinematic equation this is only applied to calculating a whole volume rather than the different water masses making up that volume. The majority of 1D models (that is models that account for flow in only one dimension or direction, downstream) operate in a similar manner (Cox 2003). These models are more concerned with mass balance and chemical interactions than mixing processes and are often used in steady state studies (Drolc and Končan 1996; Kannel *et al.* 2011). Whilst there are clear limitations to using a simple hydrological model and an Advective Dispersive mixing model, Mannina and Viviani (2010) demonstrated the usefulness of such an approach producing an accurate simulation of a flood wave and a CSO discharge associated with it, including longitudinal dispersion rates.

Gooseff *et al.* (2008) in a series of dye tests concluded that higher discharges, and hence velocities, will increase advection but reduce longitudinal dispersion as less dye enters temporary storage in dead zones. At lower flow, the inverse is true as the roughness of the bed disperses the dye. Interestingly, the results from flume tank experiments in chapter 4 of this thesis have suggested a different outcome when a wave is introduced into the situation. In chapter 4, both dispersion and advection were seen to increase with the arrival of a wave front. This leads to an important question; do the relationships between flow and both advection and dispersion observed by Gooseff *et al.* (2008) hold for scenarios with rapid changes in flow? This chapter will seek to present an answer to this question.

3D Models have been developed for transient in river hydrodynamic pollution modelling with the Environmental Fluid Dynamics Code of Hamrick and Wu (1997). Whilst this code is 3d it uses internal submodels to simulate transport of pollutants (Ji *et al.* 2002) and therefore has limited suitability for examining particle interactions.

In summary, there are two reasons why 1D models, and models that abstract the mixing model from the flow model are insufficient for the purposes of this chapter; first if the body of water is treated as a single uniform volume any vertical mixing processes within the water column will not be accounted for. Results discussed in both chapters 3 and 4 suggested a vertical stratification of flow that could affect the distribution of a pollutant within the water column. Second, if pollutants are treated as a concentration being diffused within a simple medium then the kinematic nature of the wave, and its ability to outpace the pollutant may not be accounted for. Both the kinematic nature of the wave and the vertical and horizontal distribution of pollution are of interest in this study. Therefore a method that considers the water body and the pollution within the same continuous phase was required, a model where the motion of the water at different locations has a direct effect on individual pollution particles. On this basis a 3D model was used to estimate flow and particle movement changes at a fine resolution both temporally (<1second) and in 3D space (<1m).

5.2.2 3D modelling approaches

3D models of a high spatial resolution are complex and require powerful computers. Consequently their use has been focused in the fields of computer fluid dynamics (CFD), which is sometimes termed numerical wave tanks (NWT), when water based studies are of concern. The development of simulated NWT experiments within fluid dynamics has historically been closely tied to the maritime industry and its technological development (Kim *et al*. 1999). Consequently there has been a large body of research looking at deep water shallow wave interactions with static or moving bodies typically representing ship hulls or maritime structures (Shirakura *et al.* ; Contento 2000; Boo 2002; Park *et al.* 2003). In fact a review by Kim *et al.* (1999) listing the applications of NWT studies does not list any explicitly river channel based work with only wave theory focused papers being added to the list of oceanic studies.

Despite the lack of cross over with many oceanic papers there is a notable volume of CFD studies within the riverine environment. Hardy *et al.* (2011b) use a steady state 1.5m long model to examine a bifurcation of a river and its effect on stream velocities and helicity of flow. A number of studies have focused on features that influence flow resistance such as vegetation, with Marjoribanks *et al.* (2014) using a Reynolds Averaged Navier Stokes (RANS) to examine the movement of flow around plant canopies in a model that represented a small section (0.5x0.2x0.4m) of a flume experiment, or bed topography with Casas *et al.* (2010) and Hardy *et al.* (2010) both running small scale steady state flow simulations over detailed bedforms. Papers that model either a larger reach scale of a river or non-steady state conditions with a flow change or surface deformation are much less common. Nicholas *et al.* (2012) use a CFD simulation to model a 3km section of the Rio Parana river in Argentina, a large river with a sandy bed, the model used a fixed lid for the surface and a 15m x 1m mesh size as the focus was on internal flow velocities. The closest study found to that carried out within this chapter was Rüther and Pedersen (2014) who use a VOF and a realizable K-ε turbulence equation set to model the response of 200m stretch of river to a hydropeaking event, or in other words a reservoir release. The authors do not provide a data output beyond a description of their unsteady simulation but results are presented for both high and low flow steady state scenarios within the river reach. A similar study conducted a few years earlier by Rüther *et al.* (2010) also examined a reach of 50m with a VOF model and its response to a flow event but again only steady state results are reported. As recently as 10 years ago Lane and Ferguson (2005) point out the lack of 3D studies examining changes in flow upon in stream flow characteristics.

If the interest in a wave is removed CFD papers concerned with just pollution transport are fairly common. Modenesi *et al.* (2004) report 3D steady state pollution dispersion model for rivers. A simplified transient model of a confluence of a river and a dye injection was performed by Wendel *et al.* (1997) which treated the water surface as a static boundary. Wai and Lu (1999) report a CFD simulation of entrainment and transport of sediments in a flume channel and there are a significant volume of papers that use computer numerical technics to study flow gradients within river channels (Lane *et al.* 1999; Booker 2003). Whilst these papers may give a good account of contaminant dispersion within steady flow conditions, and do attest the applicability of CFD techniques to solving such a problem they give no indication of how a rapid change in flow would affect the processes being modelled. Papers that examine waves, or changes in flow in a transient simulation are less common. Monaghan and Kos (2000) compared a flume experiment with a transient CFD model in which a block was dropped into a tank to generate a wave. The characteristics and velocity profile of that wave led the conclusion that dropping a box into a tank creates a vortex behind the produced solitary wave. There is a methodological similarity between this study and the work presented in this chapter, both use CFD to validate and detail a flume tank study, however the object of this paper and this chapter differ. The subtleties of wave generation are not the concern of this chapter, but rather the waves effect on particle mixing.

Studies that focused both on replicating results across scales, and a study that examined the effect of a single release wave on particle distribution within a bounded channel were not found in the literature searches conducted (search terms included; CFD, numerical flume tank, waves, non-steady state in various combinations, both Google Scholar and Web of Science were searched on 22/10/14), with the papers described in the previous three paragraphs being the closest approximations. This chapter will address this gap in the published literature. Additionally, the processes observed in the flume tank in chapter 4 will be validated, and the particle processes effect waves of comparable scale to those measured in chapter 3 will be quantified.

For the purposes of this thesis the CFDM approach adopted allows the spatial resolution discrepancy between the 5m flume tank model and the tens of metres scale of a river reach to be bridged. Secondly it allowed for a detailed quantification of particle behaviour in the water column during wave passage. A CFDM approach can easily be scaled between a 5m and river reach model. This results in this chapter provided a finer resolution answer to the three key questions, Q1, Q2 and Q3, of this thesis outlined in the QAM. Furthermore Aims A2 and A3 have been addressed in this chapter. With this in mind the following chapter specific aims are given.

5.3 Aims

The overarching aim of this chapter was to validate the flume tank results of chapter 4 over a longer numerical tank and test the effect of wave magnitudes comparable to those reported in chapter 3. This has been divided into the following specific aims and more detailed objectives;

Aim 5.1 was to demonstrate that CFDM is an appropriate methodology for replicating the conditions of both the flume experiment in chapter 4 and for translating these conditions to a larger river reach scale..

Objective O5.1.1 was to replicate the parameters from the flume tank within a numerical model and to compare the results with the flume tank results of chapter 4.

> If comparable wave and particle patterns were produced by the CFD it would validate the CFD approach against the flume tank.

Objective O5.1.2 was to produce a model scaled to a river reach that was methodologically comparable with the flume to examine any changes in result between scales.

> Should similar patterns be observed in a model of such scale to those produced by the flume tank it would suggest that the flume tank results are applicable at larger scales.

Aim A5.2 was to accurately quantify the effect of waves on particle distribution. Specifically this included;

Objective O5.2.1 quantifying longitudinal dispersion and advection as two measures of specific particle processes.

Objective O5.2.2 quantify the vertical stratification in flow velocities and the particle motions associated with it.

Objective O5.2.3 quantify the effect of differing magnitudes of wave on longitudinal dispersion.

Aim A5.3 was to approximate the riverine environment within the CFDM system by testing boundary conditions, wave magnitudes, and channel slope angle comparable to those observed in chapter 3 within the reach scale model.

The aims and objectives presented here required a compromise. The primary aim was to validate the processes seen in the flume at a larger scale. Therefore the model design is focused on reproducing a flume at a larger scale. However A5.3 also required an approximation of the riverine environment. Within the modelling environment adding channel slope and increase in bed roughness and removing the step from the flume inlet were all possible and were not likely to take the model too far away from the flume experiments. Examining a complex channel with curvature, both cross sectionally and longitudinally, or a riffle pool system whilst valuable would move the model beyond comparability with the flume experiments of chapter 4. Therefore the reach scale model was designed to be a long conduit as the flume tank was with boundary conditions and a slope angle designed with a simple riverine system in mind.

5.4 Methodology

Two numerical models were constructed for this chapter. The first model considered a channel 5m long and so was designed to be a similar as possible to the flume tank described in chapter 4. This 5m model was created to meet Aim A5.1 and objective O5.1.1. The second model was for a channel of 120m long (referred to as the river reach model) and scaled in terms of discharge, wave magnitude, width and depth to be more similar to a reach of the Holme river system described in chapter 3. This 120m model was developed to meet each of objectives O5.1.2 through O5.2.3 . Scale aside, both models were designed to be methodologically as similar as possible so that comparisons could be drawn. The same equation sets were used, the only changes being the physical sizes involved, the mesh resolution and the boundary conditions.

This methodology section has been split into three sections. The first gives an overall description of both models as they share many design parameters and underlying mathematics. The second and third sections give design specifications for the 5m and river reach models respectively.

Both the models described in this chapter were constructed using Ansys Fluent 14.5 computer fluid dynamics software. This was selected as it is capable of solving 3D fluid dynamics problems at a variety of scales and uses the Volume of Fluid (VOF) solution method. The benefits of using VOF will be described later in the method. It is also wide spread within the engineering industry and widely available.

In over view, each of the models represents a conduit with water flowing down it from one end to the other. The water does not fill the conduit but rather has a free surface like a river channel or flume tank. After a set period of time a volume of tracked anthracite particles were released into the tank, then after a set time, the flow into the tank was raised generating a wave, or hydrograph. This wave then proceeded down the tank after the particles. The model was built within a 3D eulerian Cartesian grid, often termed a mesh. The mesh consists of square cells with air, water, and anthracite particles being passed between cells as directed by the motion and conservation equations.

Three modelling algorithm sets were used; The volume of fluid system (VOF) for the determination of the free surface, the Realizable K-Epsilon viscosity model for fluid motion, mass conservation and turbulence, and the discrete phase model for the motion of anthracite particles. Explanations of these models are now given;

5.4.1 Volume of Fluid

The Volume of Fluid (VOF) system is a method of representing a free surface; in this case that of the water surface in contact with the air. With this chapter being concerned with a fluid wave a detailed calculation and rendering of the 3D surface is necessary. The VOF does not deal with the flow of water between cells in the mesh, rather, the movement and conservation of mass is handled by the viscosity equation. VOF assigns a value, typically between 0 and 1 to each cell. If a cell is at 0 there is no fluid and if at 1 it is full, in these two situations the cell is not part of the free surface. If there is a fraction, the surface within the cell is given a direction orientated to the direction of greatest change with neighbouring cells. This method was first detailed by Hirt and Nichols (1981) and is fairly ubiquitous for modelling free surfaces due to it having both lower computational requirements than other systems such as marker particles, and being flexible enough to represent bubbles and complicated surface forms such as waves. Oertel and Bung (2012) are a recent example of a paper utilising the VOF method to study a wave. In this case a dam break wave moving over <22m of a dry surface with an obstacle positioned on it. Whilst the model presented in this chapter is of a larger scale, Oertel and Bung (2012) achieve a sound validation of their model against flume data showing that a VOF method can produce realistic results when modelling complex fluid surfaces.

5.4.2 Turbulence Model

When selecting a turbulence model there were four key criteria that were important for this model. First, the act of a wave motion and the process of entering an increased volume of water into the tank poses an increase in particle strain within the fluid. A wave creates a deformation both in the fluid surface, and within the water column, moving particles near or further apart rapidly. Inputting an increase in flow over time into Fluent is a complex task. Water has to be entered through a defined surface area, which is a constant. If this area is small a high volume of water will enter under high pressure and create an inaccuracy in the flow field, it is then more reliable to enter the water over a wide surface excepting that this will create a hydraulic jump. This is further explained in the input boundary conditions section. The result of this input is a high fluid stress. Consequently the turbulence equation needed to be able to cope with high fluid stress.

Second, the equation had to be solvable and reach convergence within the computing power available. The reach scale model designed had a total length of 121m. As described in the literature review this is a considerable distance over which to model a free surface and few papers have attempted it,therefore all the equations considered had to economise on computational power.

Thirdly, and related to the first criteria, the turbulence model needs to perform well under Reynolds numbers likely to be experienced within the model;

Water was defined as having a density of 988.2kgl^{-1}, and viscosity of 0.001003kgms^{-1}, given reference temperature of 24.85ºC as detailed in Fluent (2014). Therefore for a 5m wide and 1m deep channel or 0.15m wide and 0.05m deep conduit the Reynolds equation below can be used to calculate the Reynolds numbers of 5.91×10^4 and 1.42×10^6 for the 5m and reach models respectively for the desired steady state flow of 0.5ms^{-1} as calculated with the following formulae;

$$\mathrm{Re} = \frac{\rho \mathbf{v} D_H}{\mu}$$

Where Re is Reynolds number, **v** is mean velocity, ρ is density and µ is viscosity, and D_H is defined below

$$D_H = \frac{4A}{P},$$

Where A is cross sectional area, and P is wetter perimeter of the conduit.

These Reynolds numbers can be classified as turbulent flow (Holman 2002) and both could increase dramatically with flow velocities exceeding 1ms^{-1}.Therefore the turbulence equation needs to produce reliable results when experiencing rapid changes.

Fourthly, the model both has boundary interactions, at the walls, inlet, outlet, surface and bed and a significant volume of water further from the boundaries in a free turbulent state. Therefore the turbulence model needed to be able to cope with both fluid environments.

The fluent software package offered four main sets of turbulence models; the K-Epsilon series of models, the K-ω series, the Reynolds Stress Model (RSM) and the Large Eddy Simulation Model.

The original K-ε model was put forward by Jones and Launder (1972). It is a two equation isotropic model that has worked well with high Reynolds numbers. A key limitation however is the models handling of regions near wall boundaries. To an extent this can be mitigated with wall boundary functions (Sotiropoulos, 2005).

Two improvements over the original K-ε model were available in Fluent, the RNG K-ε (Boysan 1995) and the Realizable K-ε (Shih *et al.* 1995). Both are considered improvements over the original equations being more accurate when handling flows with high fluid stress, high Reynolds numbers, and rotational flows all of which were relevant to this study. Both of these iterations of the K-ε model were tested in an earlier iteration of the reach model and the Realizable K-ε was found to resolve faster with the difference in results between the two equation sets not been visually noticeable. K-ε models have been popular with riverine and numerical flume studies with examples including Hardy et al., (2005), (Hardy *et al.* 2011a), Shamloo and Aknooni (2012), and Rüther and Pedersen (2014).

The K-ω models are often attributed to Wilcox (1988, 2008) and Menter (1994) and are described as being an improvement on K-ε models in terms of their ability to represent boundary layer flows but also being limited in their representation of free shear turbulent flows. Whilst there were important boundary conditions in the models presented in this chapter the real interest was in the effect of a wave. In the flume tank a turbulent wake was observed behind the wave which may approximate to free shear conditions. Wilcox (2008) has attempted to compensate for this apparent weakness in the model however an examination, of the 155 papers listed as referencing that paper on the Google Scholar database, yields one paper detailing a riverine application (Blondeaux and Vittori 2014) and no papers examining a detailed water surface. Given that the K-ε is so visible within the literature by comparison it was the favoured solution.

The Large Eddy Simulation abstracts out small eddies and attributes large eddies as being responsible for the majority of moment and transport into the flow field. This abstraction, could have presented an accuracy issue though this was not verified. Further to this however LES requires a long flow time to achieve a stable model (Fluent 2014) and so was considered unsuitable.

The Reynolds Stress model was rejected largely on the basis of computational expense. As a package it uses a total of 13 equations more than the two equation models listed above. Fluent (2014) recommends the model for the study of swirling flows and vortexes, which could be relevant to the interior of a wave, however this detail comes at too higher cost in calculation power.

Fluent does offer a considerable list of alternative equation sets beyond these however many of these other options are either further derivatives of the systems described above or required a higher computational cost.

Of the models described the Realisable K-ε was used as it was viewed as being able to handle the fluid stress, Reynolds numbers and free surface effectively whilst being able to solve with the limited computational power available. However arguments for the use of the RNG K-ε or a K-ω model could be made as the K-ω in particular might of dealt with boundary conditions better.

In addition to the turbulence generated within the model is the turbulence of the water as it enters via input in to the model. Turbulence at the inlets in both models was set as a percentage intensity of 5%. This simplification was used in preference to specifying a complete profile of turbulent kinetic energy and dissipation rates, as data concerning these parameters was not recorded from either the field or the flume tank. Turbulence intensity is defined by the Fluent manual as the quadratic mean of the velocity fluctuations to the mean velocity of flow (Fluent 2014). The manual suggests that an intensity of >10% is considered high, and <1% is considered low. 5% was taken as a mid-point. Both the 5m and reach models should experience turbulence, but the wave form itself should generate this within the tank.

5.4.3. The Discrete Phase Particle tracking Model

Particle tracking was modelled using Fluent's Discrete phase model. The Discrete phase model uses a Lagrangian coordinate system as opposed to the Eurlarian of the turbulence and conservation equations of the continuous phase of the model to track individual particles (Fluent 2014). Typically particle tracking is only done in the continuous phase using a Eulerian system if the feedback between the tracked particles and the fluid volumes in the continuous phase is of interest (Dehbi 2008). This model takes the mass inputs and outputs from a given cell and moves the particle with mean velocity whilst taking into account inertia, hydrodynamic drag, and gravity and how they act upon the particle. The major advantage of this discrete phase approach is that the results are easy to extract and interpret, the major drawback is the higher computational power required (Dehbi 2008).

Fluent uses two methods of modelling turbulence at the particle scale. The first is a random walk model, and the second is the particle cloud model. The random walk model is based on the work of Gosman and Loannides (1983) and in simple terms generates random finite eddies of a Gaussian distribution that influence the movement of particles. This creates a scatter in the particles comparable with the processes of molecular diffusion and diffraction. The particle cloud model generates a Gaussian distribution around a mean trajectory of the possible pathways for the particle, thus creating a cone of possible positions for each particle outward from the source (Baxter and Smith 1993). The position of the particle is then determined probabilistically. This approach is more commonly used in mixing plume scenarios and steady state models (Litchford and Jeng 1991; Perez-Tello *et al.* 2001).

A case could be made for including a random walk model to simulate dispersion processes within the water column, however given the computational requirements this method was not used. The focus of this study is on the effect of waves on longitudinal dispersion and advection, two processes primarily determined by the flow field (Rutherford 1994). Furthermore the fluent manual (Fluent 2014) suggests that in heavily heterogeneous flow fields the random walk method can cause a bias toward particles moving to areas of low turbulence flow.

Anthracite dust was used as the tracking particle in the discrete phase. It was used due to it being both inert, and neutrally buoyant at low particle sizes making it ideal. This thesis was primarily interested in the dilution of solute contaminants, therefore a tracer particle that will behave in analogous way to a solute is desirable.

The model parameters discussed so far were applied in the same manner in both models in order to maintain comparability between the methods used.

5.4.4 Hardware Constraints

All of the model runs undertaken for this chapter were completed on one of two computers. On both machines 5 CPU cores clocked at around 2.5ghz, and 8Gb or RAM were available. It took between 12 and 24 hours for either machine to solve a single run of either of the two models designed. This is a long solution time. It was therefore imperative in the design to keep the models as simplistic as possible. These constraints impacted the mesh resolution, the length of the longer river reach model, and the decision not to use a random walk model.

5.4.5 5m Model Design Specification

The design of this model was intended to replicate the geometry of the flume tank experiment as described in chapter 4, within the abilities of the fluent software package and the hardware power available as described above.

5.4.5.1. Physical Dimensions

The model's dimensions were of a rectangular conduit 5m long, 0.15m wide and 0.3m high as shown in figure 5.1 below . At either end of the model the wave input and flow output were elevated 0.05m above the tank bottom creating a 0.05m step or weir at either end. These weirs were utilised in the flume tank described in chapter 4 to maintain a 0.05m water depth within the tank prior to wave release and they serve the same function in this model. The model had a time step of 0.5 seconds.

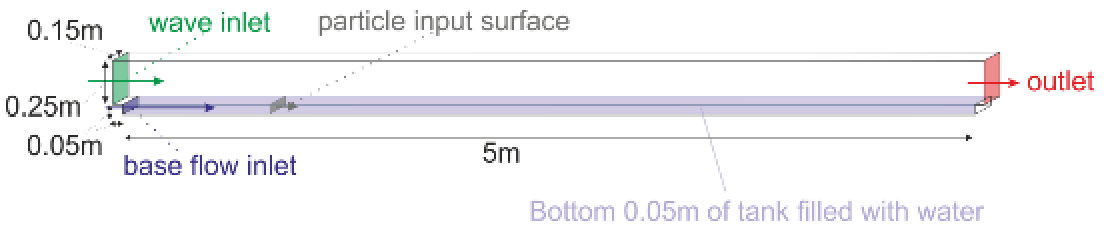


Figure 5-1. 5m Model design. The particle input surface shown in grey is located 1m down the tank. The input surface is shown in green and output in red. The blue area indicates the volume filled with water. The dark blue arrow and the dark blue surface indicate the baseflow inlet.

5.4.5.2 Inlets and Outlets and their Boundary Conditions

Two inlets were used in this model. First is the wave inlet that is situated at the start end. This inlet covers 0.25x0.15m square and sits above a step elevated 0.05m above the base of the tank. From this inlet the wave water is released at a rate of 12.5ls^{-1} between 3 and 4 seconds after the model has started running. The volume and rate of flow is the same as that released in the flume tank in chapter 4. Below this is a 0.05m step and then the vertical surface of the baseflow inlet. This flow inlet was assigned a constant inflow of 0.4ls^{-1} to provide the baseflow for the tank, this flow rate is the is rounded up from the 0.375ls^{-1} released in the flume experiment of chapter 4. The outlet was situated opposite the wave inlet at the other end of the tank above the 0.05m lip. Air was given a backflow fraction of 1 at the outlet to prevent the model from losing pressure.

5.4.5.3 Discrete Phase Particle Injection

Particles were injected from 2 until 3 seconds of the models runtime. The injection flow rate was 1gs^{-1}. Each particle had a diameter of 0.1mm and the physical properties of anthracite dust. Particles were injected across a flat surface perpendicular to the long axis of the tank at the 1m point down the tank. The surface had 10x10 grid of input points evenly distributed across it. This is a simplification from the manner in which dye was introduced to the flume tank. In the flume tank dye was injected with a pipette into the water column, producing a cloud with a bias in concentration for both the top and bottom of the tank. Here particles are introduced perfectly evenly. This is a necessary difference, since simulating a pipette injection would require a pressured input and the random walk model to produce a realistic cloud. Both of these would stretch the limited CPU power beyond capabilities.

5.4.5.4 Global Conditions and Wall Boundary Conditions

Gravity was defined as -9.81ms^{-1} on the vertical Y axis. The flume tank being simulated had a level base so a 90º gravity axis was desired. The walls were treated as a smooth surface that would reflect particles on impact.

5.4.5.5. Mesh Resolution

The whole model was uniformly meshed at 0.025m. Specifically, the model geometry was divided in to 0.025m cubes and the equations running the model were solved in each cube. This resolution is also therefore the accuracy resolution of the model. Any processes that occur at a scale lower than 0.025m would be generalised rather than detailed in the output. Mesh resolution is always a trade-off between accuracy and computational power.

5.4.5.6. Running Parameters

Prior to starting the model the bottom 0.05m of the tank was filled with water, in the same manner as the flume tank experiment. Particles were released 2 seconds after the start of the model run time and the wave was then released at 3 seconds. The model was run for 10 seconds with each time step being 0.01 seconds at a maximum of 20 iterations per time step. This very fine time resolution was necessary to prevent the solution from diverging due to too greater change occurring between cells in the time step. An additional benefit of this small time step is that the model has a higher temporal precision.

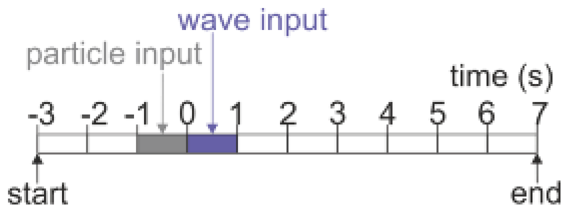


Figure 5-2. Timeline of the model running time. Time is counted forward and backward from the start of the wave release as this is how the data is presented in the results. The blue second shows the time in which the wave input was raised and the grey the particle input.

5.4.6 River Reach Model Design Specification

The 121m long model was designed to approximate the conditions in a river. Consequently the axis of gravity was altered to give the tank a downward slope, flow rate and wave magnitude were also scaled using the results reported from the field in chapter 3.

5.4.6.1 Physical Dimensions

The tank was 121 by 5 by 5m with a 1m deep and 0.5m high step at the bottom end as shown in figure 3 below. In the graphical results only 100m of the model are presented. This is due to the edge effects created by inlet and outlet conditions. The flow of water at the inlet produced a hydraulic jump (Chow 1959) characterised by a rapid fluctuation in water depth before settling out after 10m. This supercritical flow is a result of the input system available in fluent. Flow has to be input across an entire surface. If a smaller surface was used, flow would input under pressure, so the whole input 5m high end of the tank was used resulting in some water falling 4m.

Consequently a hydraulic jump is introduced. The outlet step weir raised the water depth over 1m as water backed up behind it before flowing over. These two variations in stage were not of interest in this study so the model was designed to give 100m of stable flowing water. The intention was to use this central 100m section in the results with the 10m either side are cropped out. After viewing the statistical output it was decided that 110m of the tank should be considered for the numerical results reported in the tables. The reason for this was that many particles had passed beyond 100m by the time frame that the range and mode statistics for particle distribution could be calculated. By using 110m a more accurate statistical portrayal is given than cutting off the data at 100m. The 10m at the inlet end of the tank is not presented in any of the results.

The rivers Holme, Ryburn, and Don are all wider than 5m through the majority of reaches studied in chapter 3. However, 5m is wide enough to demonstrate any cross channel lateral differentiation in flow velocities and consequently particle processes. A wider channel would vastly increase the computational requirements by expanding the tank volume and mesh size.

As detailed in the site descriptions in chapter 3, many of the Yorkshire regions rivers have an artificially elevated stage as weirs and flow control structures are common place. Therefore, as with the flume tank and the 5m model a small weir was built into the bottom end of this model to control stage in the form of an outlet step. In both models as with the flume, if the outlet step was removed stage would not be maintained as it is in the rivers with weirs.

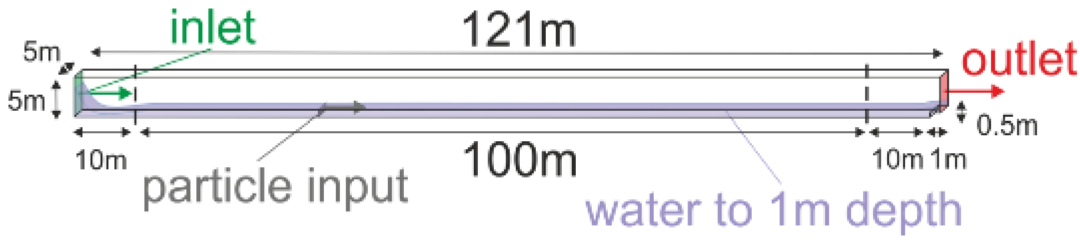


Figure 5-3. Reach Model design. The dashed lines represent the limits of the 100m section of the tank represented in the graphical results. The statistical results cover the 10m following this too. The blue shows the approximate volume occupied by water. The green surface and arrow show the inlet, the grey the particle input (35m from the inlet) and the red, the outlet.

5.4.6.2. Inlets and Outlets

The whole 5x5m area of the upstream end of the tank was defined as a velocity flow inlet. This inlet input 2.5 m^3s^{-1} for the first 14 seconds of the model run time, as this was sufficient for the simulation to settle to a steady state condition. At 15 seconds flow was raised to $10m^3s^{-1}$ for the base scenario (and then varied as described in table 1 in other scenarios) and then reduced back to $2.5m^3s^{-1}$ at 17 seconds to give the wave a 2 second duration (see figure 5.4). The waves released from reservoirs in chapter 3 involved the valves at the reservoir being opened for 2 hours or more. 2 seconds was used here so that the declining limb of the wave could be seen.

The outlet was defined as the down tank area above the weir of 5x4.5x5m. The outlet was defined as an exit for particles and fluids with a backflow of air at a rate of 1kgs^{-1} to prevent the model from losing pressure.

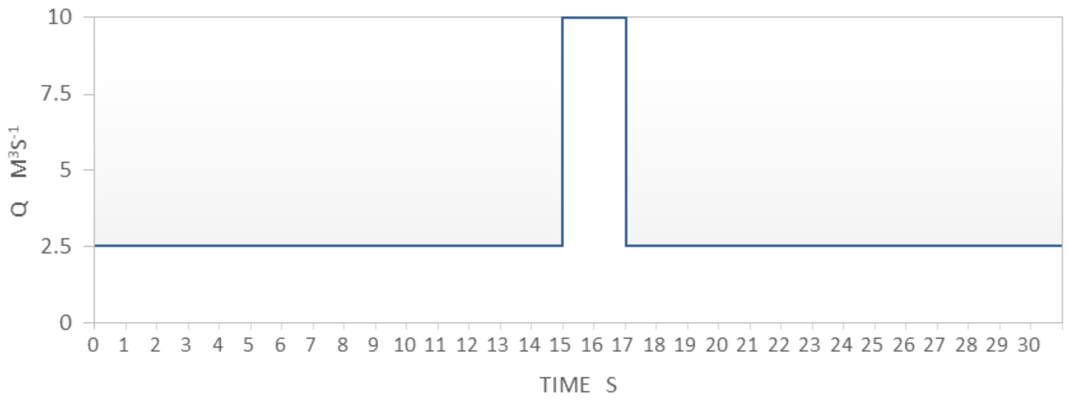


Figure 5-4. Discharge through time at the inlet of the Reach Model.

5.4.6.3. Experiment Sets

The magnitude and the duration of the wave was then varied from the parameters described above in order to meet objective 5.2.3. The parameters varied are detailed in table 1 below.

Table 5-1 the two parameters that were varied and their respective iterations that were tested.

parameter	Iterations
Wave magnitude	2.5m^{3}s^{-1}, 5m^{3}s^{-1}, 10m^{3}s^{-1}, 12.5m^{3}s^{-1}, 15m^{3}s^{-1}.
Wave duration	2, 3, 5, and 10 seconds

Wave magnitude has been shown to positively influence wave velocity in the literature (Gilvear 1989) and in the field results of chapter 3. In addition the greater kinetic energy transmitted by the wave has the potential to move the particles further down the tank. The five iterations used were designed to reflect comparable changes in flow during wave passage in the field experiments. On the River Holme flow rises from 1m^{3}s^{-1} to around 4m^{3}s^{-1}, on the Don, 1m^{3}s^{-1} to 9m^{3}s^{-1} is the flow change during a release. A baseflow of $2.5m^3s^{-1}$ was required to create 1m flow depth in the tank. A 1m flow depth was required to give a visible vertical profile to the results in line with objective 5.2.2 and to compensate for the mesh resolution described later. $5m^3s^{-1}$ was a 100% increase on this baseflow and $10m^3s^{-1}$ is a 300% increase.

A longer wave duration maintains the increase flow rate, and thus both dilution potential, and in stream velocities to move particles for longer. All of the wave lengths used here are low compared with the field. Releases from the reservoirs described in chapter 3 were all of 1 hour or longer. Not only is it unfeasible for the model to simulate a full hour in terms of processing power demands but at the 100m reach scale it is also not applicable. 10 seconds of $10m^3s^{-1}$ input was sufficient to show the prolonged effect of a higher flow rate on the particles. The 3 second duration wave was included because it inputs a total volume of $30m^3$, the same total volume as the $15m^3s^{-1}$ wave from the previous experiment set. This allows to a comparison between a higher magnitude shorter wave and a lower magnitude longer duration wave of the same total volume.

5.4.6.4. Discrete Phase Particle Injection

Particles were injected from a vertical surface area of 1 x 5m positioned 35m down the tank or 25m from the wave entry end of the tank as presented in the results section. This gave an even spread of injected particles across the flow field in an identical manner to the 5m model. The 25m down tank was an arbitrary point that in the same manner as the 1m injection point in the 5m model and the flume tank of chapter 4 gave the particles some time to move down tank before being caught by the wave. The injection occurred for 1 second starting 13 seconds after the model was started, or 2 seconds before the wave was released as shown on figure 4 below. The anthracite particles had a diameter of $1\text{^}10^{-6}$m and were injected at a flow rate of $2\text{^}10^{-4}kgs^{-1}$. This injection was sufficient to produce a visible particle distribution down the tank and 3081 particle tracks for statistical consideration.

5.4.6.6. Global Conditions

The angle of gravity was altered from the 90º angle by 2.3º to simulate a slope. This angle is the mean slope angle for the Holme catchment as described in chapter 3. Therefore gravity was set to $0.39ms^{-1}$ on the X axis and -9.81ms on the Y. This creates a gentle slope down tank.

5.4.6.7 Wall Boundary Conditions

The walls in addition to being reflectors of particles were given a roughness property to simulate the gravel and boulders substrate of the rivers described in chapter 3. A roughness height was set as 1m and a roughness constant was set at 0.8. The height condition only affects the immediately neighbouring cells, thus the true height effect is 0.25m as that was the mesh resolution, rather than 1m, and the constant is a number between 0 and 1, with 0 being a smooth surface and 0.5 being a rough pipe and 1 being maximum roughness. The fluent manual (Fluent 2014) describes this in the abstract, therefore 0.8 was selected as being rougher than a pipe. A 1m height was chosen as an approximation of the conditions in the River Holme, where boulders, gravel bars, weirs, and vegetation often have a greater vertical height than the river stage. Anecdotal measurements noted that bank vegetation and boulders never extended beyond 1m into the river, but were often greater than 0.5m. The roughness height of 1m was also used in an attempt to slow the flow as both early tests and the final result produced velocities in excess of $5ms^{-1}$.

5.4.6.8. Mesh Resolution

The geometry was meshed at 0.25x0.25m. This resolution was sufficient to make the 1m deep water of the baseflow conditions four cells deep. This provided four solutions of the fluid motion equations in baseflow conditions allowing for vertical stratification. Higher resolution was not feasible with the processing power available.

5.4.6.9. Running Parameters

The tank was filled to 1m deep prior to the running of the model. This model was then ran with just the $2.5m^3s^{-1}$ baseflow input for the first 13 seconds of run time to allow the model to settle to a steady state. A timeline with the wave release as the 0 point is given below to help visualise the temporal input changes within the model. The model was solved in 0.1 second time steps giving a high temporal resolution.

5.4.6.10 Time Line

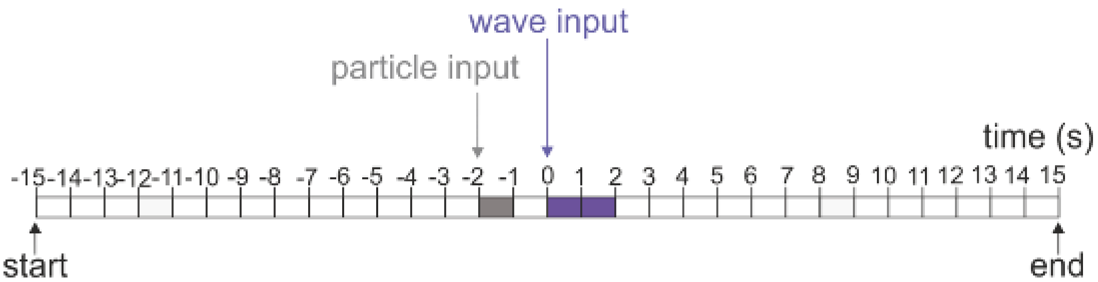


Figure 5-5. The timeline for the Reach Model. Time is centred at the point of wave release since this is how the results are presented. The blue seconds show the input of the wave and the grey the particles.

5.4.7 Interpretation of Results

Aim A5.2 of this chapter was to quantify advection and longitudinal dispersion. Whilst definitions of these process were given in chapter 4 it is necessary to expand upon them here to clarity how they will be quantified within the results. Gooseff *et al.* (2008) define advection as the progress of a peak concentration of a given contaminant within a flow. In this chapter this will be treated as the mode position of particles as this measure of central tendency gives the highest density of particles. Longitudinal dispersion is defined as spread of particles, or if considering a time series graph the total length of the curve. Here range is used as a measure of the spread of particles.

All particle tracks were exported as a CSV files with grid reference positions for each particle at each time step. Therefore summary statistics and percentages for particle distributions could be calculated.

5.4.8 Limitations of the Methodology

The key limitation imposed on both models is the lack of a random walk model for the secondary phase particle movements. In essence this removes the process of diffusion from the model and rather than the particles behaving as an expanding cloud they remain static if not under motion from advection. As stated earlier in the method a random walk model was not employed as it strained the computational resources beyond a workable limit. The Reach model has a number of other clear limitations worth addressing. Firstly the roughness of the boundary conditions. This was set to 1m in height and 0.8 in grading in an attempt to represent the increased roughness of a river bed. These settings are difficult to justify empirically. Fluent does not relate its roughness scale to a widely used metric such as Mannings N but rather uses a system of its own device, thus determining whether 0.8 was a realistic measure was not possible. Secondly a height of 1m, though in effect 0.25m could be considered excessive. This height was in part used to try and slow the velocities seen within the model. Despite these abstractions, in a non-depth averaged model boundary roughness only affects the immediate boundary cells (Lane and Ferguson., 2005), therefore the particle movements in such cells should be considered with caution. Another notable limitation became apparent within the results. The reach scale model was not long enough to give a fair comparison between different scenarios when addressing the differences in longitudinal dispersion achieved.

5.5 Results

Results are presented with the 5m model first and the output of the river reach model second.

5.5.1. 5m Model Results

The results in this subsection are related to Aim A5.1 and objectives O5.1.1 and O5.2.1.

Only a single scenario was run in the 5m tank as described in the parameters set out in the method.

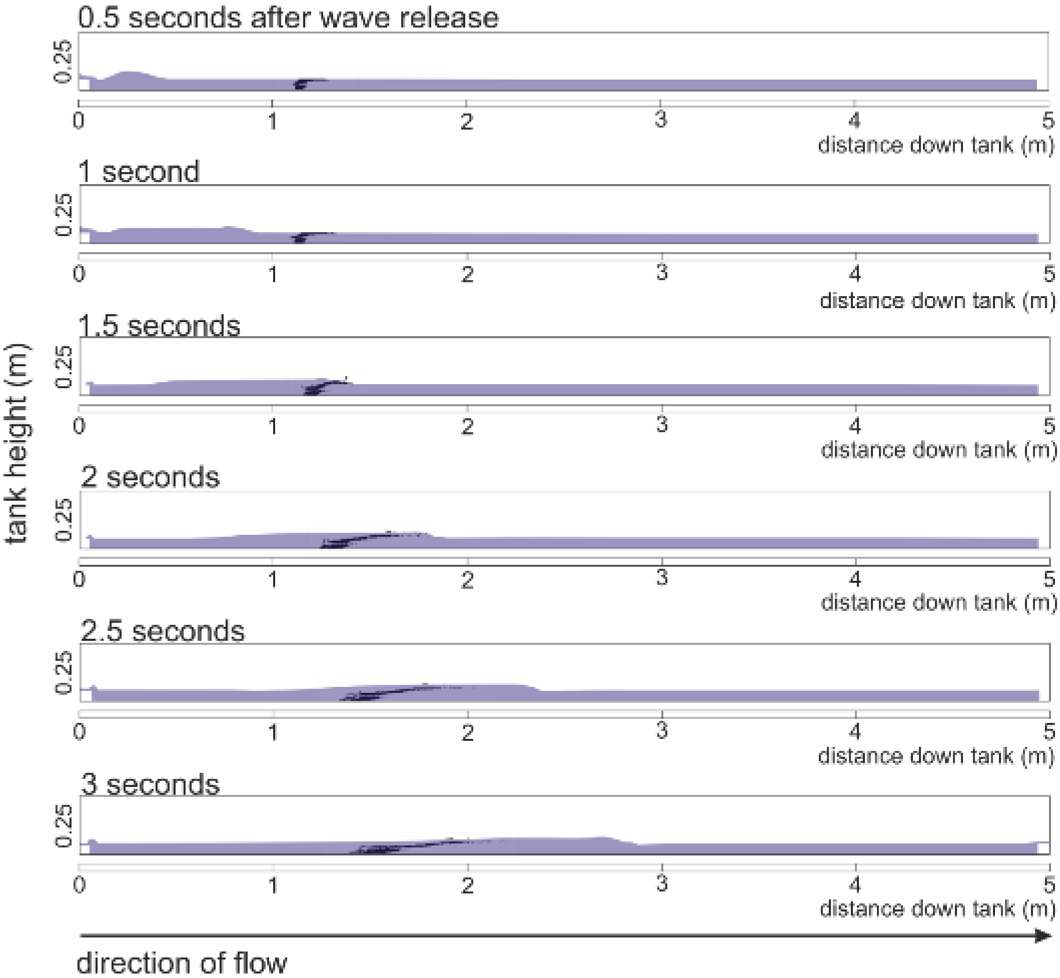


Figure 5-6. The progression of the wave (blue area) and the tracked particles (black points) down the tank at 0.5 second intervals over the first 3 seconds after the waves release.

The wave peak moved at an average 1.044 meters per second with little variation over the 5m distance. Particles were injected 2 seconds prior to the release of the wave as described in the method. Over these 2 seconds the particles became dispersed over 0.12m down the tank, largely in the top 0.01m of the water column. Once the wave had been released it took 1.4 seconds to reach the particles and begin to move them. 21% of the particles were moved above 0.05m in the water column and were then drawn down the tank with the wave. The other 79% of particles did move forward but in a far more limited manner moving at a mean of only 0.36m between 1.5 seconds after wave release and 3 seconds after. The upper 21% of particles moved a maximum of 0.87m and a mean of 0.6m. The bias for particles to move with the wave in the upper half of the water column is consistent with results reported in the flume tank in chapter 4. The wave front clearly moved ahead of the dye by 2.2 seconds after release, or at 1.9m down the tank.

5.5.2. River Reach Model Results

The following results are from the reach model. First a base scenario is presented in which $10m^3s^{-1}$ of water was released for 2 seconds into the tank. The results from this experiment are shown in graphical detail to give a visual demonstration of the processes occurring within the tank. Second results from varying scenarios detailed in table 1 are shown, first variations in wave magnitude, and then variations in wave duration in line with objective 5.2.3. The results are presented as a series of boxplots to high light the differences in data sets.

5.5.2.1 Base Scenario Results

The following graphical representation of 100m of the Reach Model relates to objectives O5.1.2, O5.2.2 and O5.2.3 and shows the wave moving down the tank and trans locating the anthracite particles down tank with it.

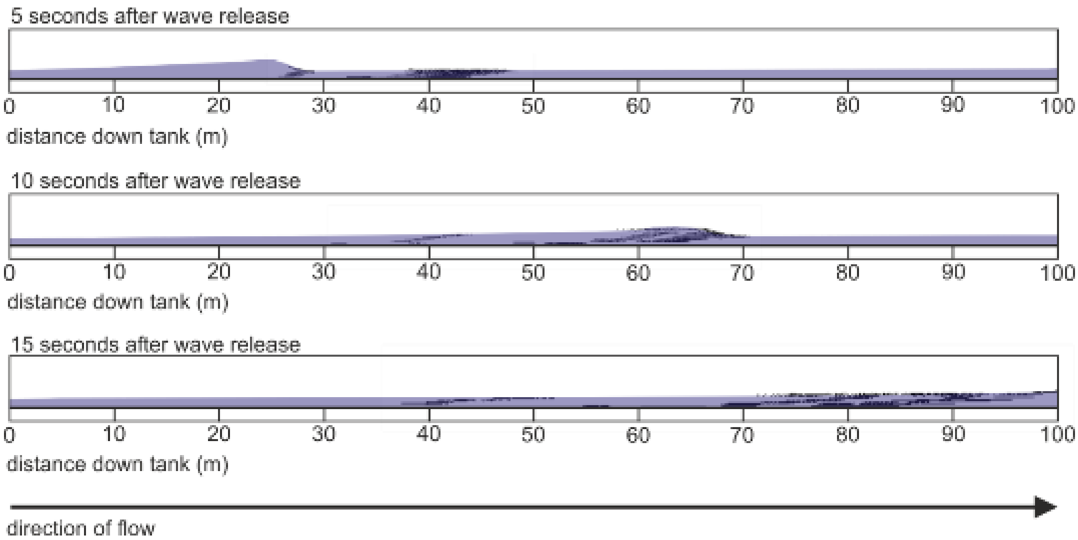


Figure 5-7. The base scenario results from the Reach Model. The blue shows $10m^3s^{-1}$ of wave water which is released from left to right down the tank. The pollution particles are represented in black.

The wave front moved down the tank at a velocity of $6.7ms^{-1}$ taking 14.8 seconds to cover 100m. Particles were released 2 seconds prior to the release of the wave at the 25m down tank point. The wave caught up with these particles after 5 seconds at the 25m point with the wave peak only outpacing the particles fractionally before it left the tank at 14.6 seconds or 98m down tank. The particle distributions from figure 5.7 above are presented in table 5-2 below. Vertical stratification of particle distribution is more limited than that seen in the 5m model. For instance at 10 seconds after wave release 39% of particles were in the upper half of the water column, that is above 0.8m. These particles had a mean down tank position of 63m, the 61% of particles in the lower water column had a mean position of 57m.

Table 5-2. Descriptive statistics for the particle distribution down the tank. Only the spread over the longitudinal down tank axis is detailed here.

Time (s)	No. particles	Minimum down tank position(m)	Mean down tank position (m)	StDev down tank position X	Maximum down tank position(m)	Range of down tank position(m)	IQR of down tank position (m)
5	3080	25.7	41.7	5.4	49.9	24.2	5.6
10	3080	31.2	60.1	8.9	71.1	39.9	8.2
15	3080	36	81.2	15.1	102	66	17.3

The range over which 3080 particles are distributed increases by more than 172% between 5 and 15 seconds after the waves release as it spreads the particles down the tank, the interquartile range also increases by 208%. The visual pattern shown in figure 6 demonstrates that the particles do split into two groups with an additional number of slower particles near the bottom of the tank.

5.5.2.2. Wave Velocity Results

In addition to particle output data and the water air boundary layer profile velocity data was output at a cellular level for the entire mesh. These results are presented for the base scenario below in figure 7.

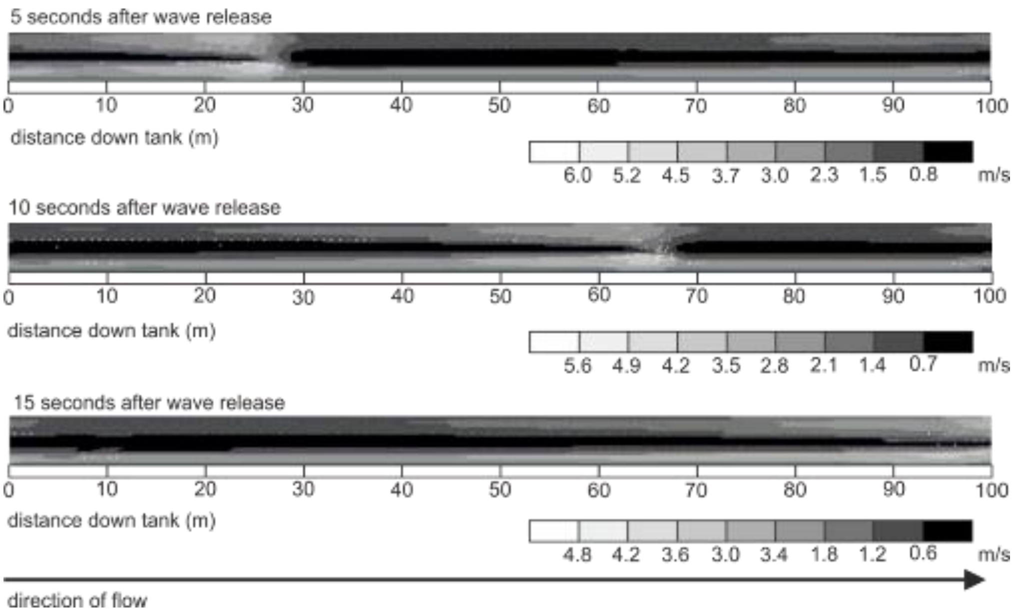


Figure 5-8. A side view of the velocity results from the base scenario in the Reach Model. White represents higher velocity magnitude and black lower. Both water and air are represented.

The highest velocities in figure 5.8 were concentrated at the surface of the wave front. Velocities decreased as the wave moved down the tank with peak cell velocity at 5 seconds being 7.38ms^{-1}, at 10 seconds 6.93ms^{-1}, and at 15 seconds 5.93ms^{-1}. Each of these high numbers represent a single cell on the wave front. The mean wave celerity velocity for the whole 100m distance was 6.76ms^{-1}. Given that two of the peak cell velocities noted here, 7.38 and 6.93ms^{-1} are higher than the celerity the flow can be stated to have met supercritical conditions. Velocity magnitude in the water down tank of the wave was generally lower than the wave at between 3.0 and 4.5ms^{-1} in the upper water column in both the 5 second and 10 second after release frames. Near the base of the tank this dropped to <2.3ms^{-1}. At the wave front, the higher velocity pixels were concentrated in a bevel shape around the surface of the rising limb and wave peak. Velocities of 5.2- 6ms^{-1} or greater at 5 seconds after release, and 4.9 – 5.6ms^{-1} or greater 10 seconds after release are shown in figure5.8. Velocity decreased with depth at the wave peak as with the rest of the tank but to a lesser degree with velocities within 0.2m of the base being as high as 3.5ms^{-1}.

5.5.2.3. Vertical and Transverse Particle Distribution

Figures 5.9 and 5.10 show in greater visual precision the locations of the particles within the tank and how they respond to wave passage. Figure 5.9 shows a side view of the tank at 2 second intervals from 5 seconds after the wave was released until 15 seconds after wave release and figure 5.10 shows a top down view at 10 and 15 seconds after wave release.

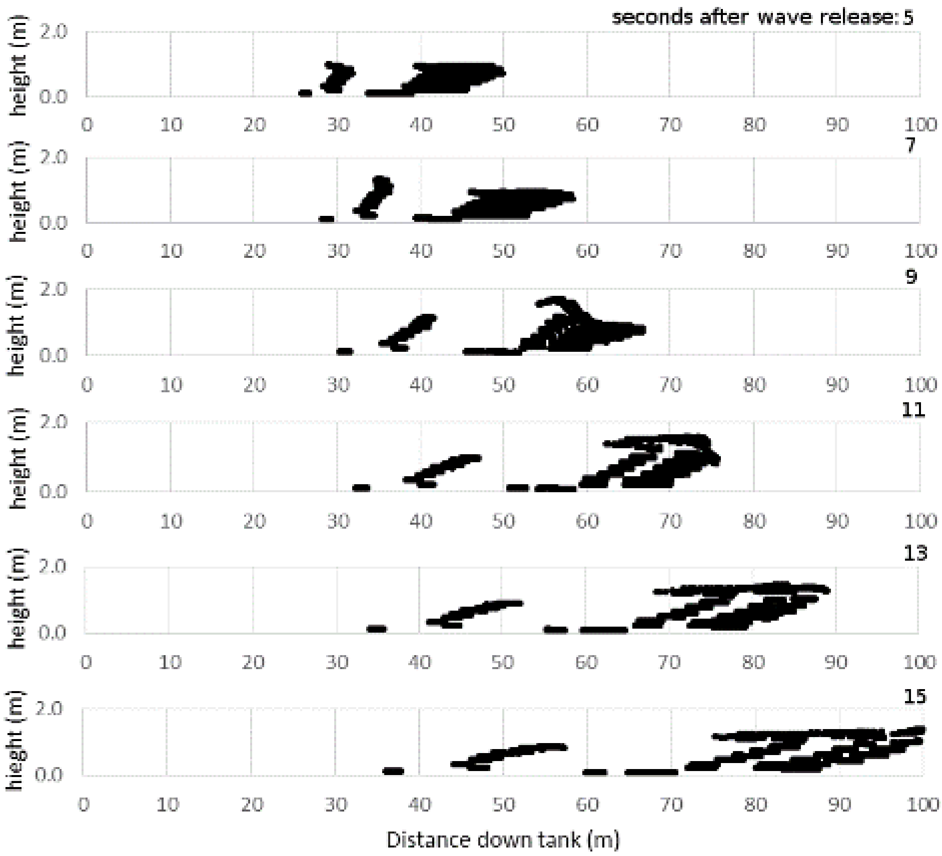
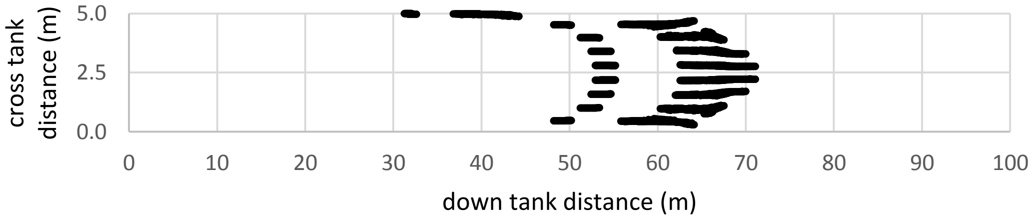


Figure 5-9. Particles are represented by the black points. Unlike figure 7 this figure is stretched on the y axis to show the position of individual particles in finer detail.

A clear pattern of vertical particle distribution through time can be seen in figure 5.9 above. Particles between 0.2 and 0.9m above the tank base maintained a close to linear profile during wave passage. Below 0.2m the particles came under a greater influence from the tank base boundary and progress down the tank at a much slower rate. Above 1m the particles are most affected by the surface and moved the fasted down the tank. At the baseflow of 2.5m^{3}s^{-1} the water is 1m deep. Therefore the vertical stratification of the particles can be split into three groups by height. Particles at the surface moved a mean distance 100% greater than those below 0.2m from the tank base. This is constitutes a clear result for objective O5.2.2.

10 seconds after wave release



15

15 seconds after wave release

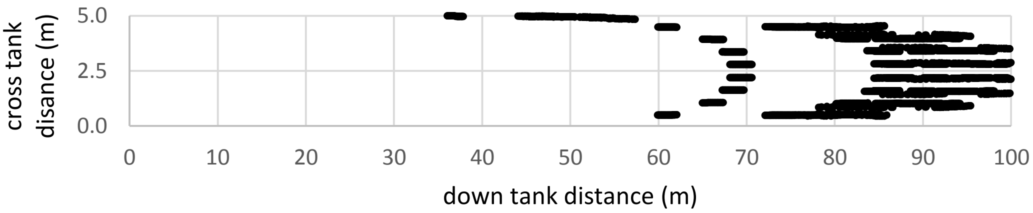


Figure 5-10. The plan view of the tank with the cross tank axis being stretched across the y axis 2.5m being the centre line and the x axis being the down tank distance. Particles are represented by black points and two points in time are depicted.

A plan view of the tank reveals that the particles dragging at the base of the tank were also influenced by the boundary condition at the far wall. An asymmetry between the near wall and the far wall was also apparent. The boundary conditions as defined in the method were identical for each wall. However when the particles were input across 10 x 10 points on the input surface, as described in the method, these points were not central in each cell resulting in particles near the 5m point cross tank being entrained by the wall to a greater extent than those near the 0m point. Particles had a higher velocity, and therefore down tank position at the centre of the water channel giving the particle distribution a curved delta shape in the cross channel dimension. When released the particles appeared in straight lines, and this can be seen in the particle group between 50 and 55m in the 10 second frame and 60-72m in the 15 second frame. The larger further forward particle mass however, at the 65-72m range in the 10 second, and 72-100m range in the 15 second frame breaks up from cleaner lines as the wave passed through. The straight lines of the particles is clear evidence of the lack of dispersion algorithm equally there is no evidence of a turbulent wake as seen in the flume experiments of chapter 4.

5.5.3 Particle Distribution Under Different Wave Scenarios

The following results detail the effects of the different wave scenarios presented in table 1, this relating specifically to objective O5.2.3.

The base scenario detailed through figures 6 – 9 involved a $10m^3s^{-1}$ wave release over 2 seconds. Two parameters of the wave release were then independently varied. First the magnitude of the wave was changed, and then the duration of its release. Results from these two data sets are presented below as a series of boxplots rather than detailed diagrams of the model output. This is because it is the difference in result from the base scenario rather than a detailed picture of each scenario that is of interest.

Wave magnitudes of 2.5, 5, 10, 12.5 and $15m^3s^{-1}$ were released down the tank, each with a 2 second release duration, the following two boxplots show the resulting particle distributions at 10 seconds and 15 seconds after release respectively.

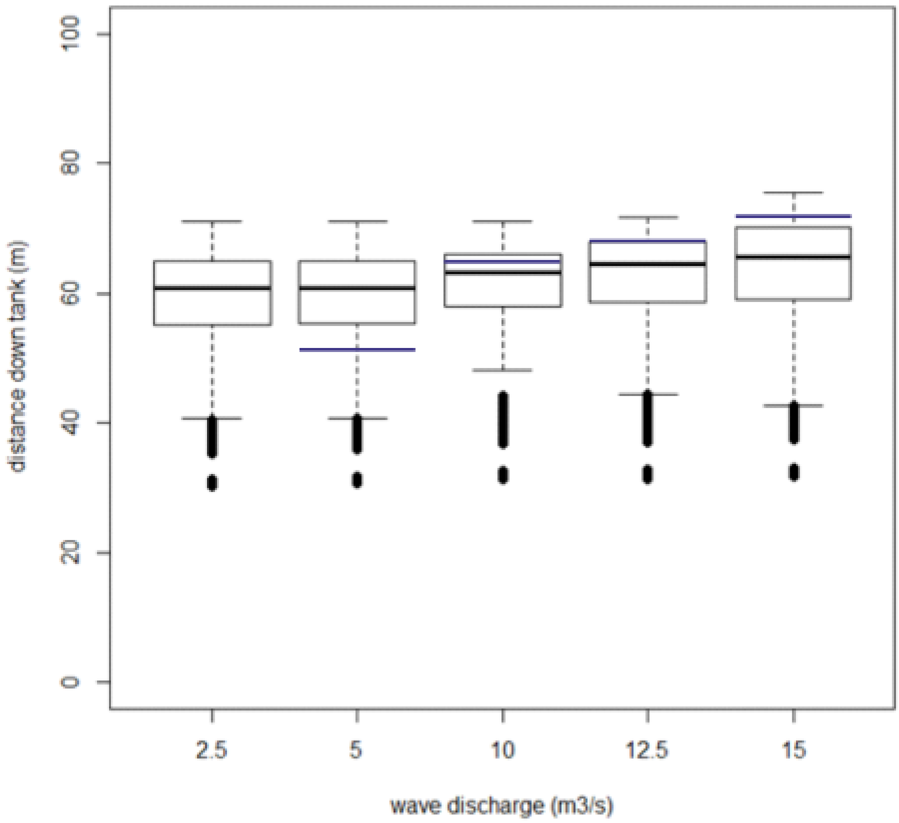


Figure 5-11. Particle distribution boxplots down the centre line of the tank (y axis) 10 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 4 scenarios with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

A blue line marking the down tank position of the wave peak has been included on figure 5.11 above because the position of the wave front is critical to understanding the particle distributions in the boxplots. At 10 seconds after the was wave released the $5m^3s^{-1}$ wave had progressed 54m down tank but had not caught up with the main body of particles so the interquartile range for both the 2. $5m^3s^{-1}$ and $5m^3s^{-1}$ scenarios was 9.8 and 9.6m respectively. The initial effect of wave passage was to reduce the interquartile range, and the range as a whole as the particles were entrained by the wave, so the $10m^3s^{-1}$ scenario showed a contraction of IQR at 8.1m compared with the two preceding scenarios. The $12.5m^3s^{-1}$ and $15m^3s^{-1}$ waves had a higher velocity and had IQRs of 9.4m and 11m as the wave had overtaken the main concentration of particles and was beginning to distribute them behind it.

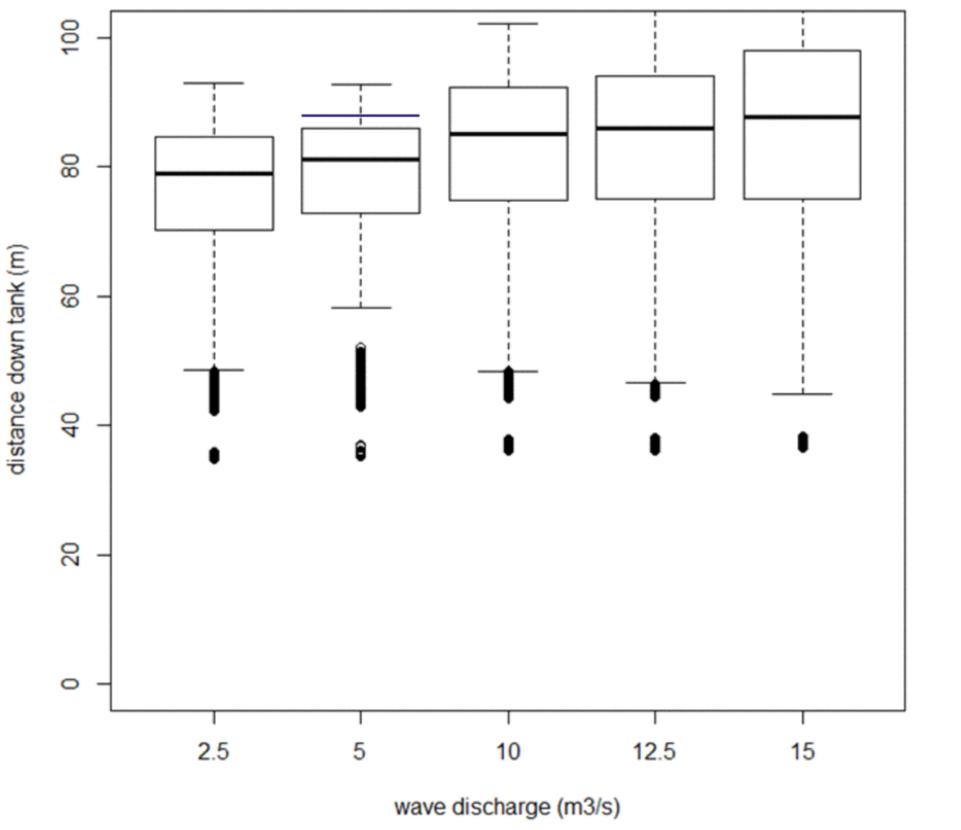


Figure 5-12. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 1 scenario with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

15 seconds after the wave was released the particles were distributed further down the tank (figure 5.12). The wave peak for the 10, 12.5 and 15m^{3}s^{-1} had passed 100m of the tank with the 5m^{3}s^{-1} wave being at the 87m mark. Some particles had passed the 100m point in the tank by 15 seconds, with 1.3% of particles in the 10m^{3}s^{-1}, 9% for 12.5m^{3}s^{-1} and 16.5% for the 15m^{3}s^{-1} scenarios. The 10, 12.5 and 15m^{3}s^{-1} scenarios all produced iteratively greater distribution of particles, as described with the ranges and IQR reported in table 5-3 below. They therefore had a greater dilution potential than the 2.5m^{3}s^{-1} baseflow scenario at the 15 seconds after release point in time. In the 5m^{3}s^{-1} scenario the wave front, (the blue line) had not progressed beyond 86m down tank. Consequently the range and IQR were smaller at 57.7 and 13.13 than the baseflow scenario.

Table 5-3. Summary statistics for particle distributions for the different wave magnitude scenarios 15 seconds after wave release.

Wave Magnitude M^{3}s^{-1}	IQR (m)	Range (m)	Mode (m)	% of particles past 100m	% increase in range
2.5	14.50	58.20	91	0.00	0
5	13.13	57.70	90	0.00	-0.9
10	17.30	66.00	92	1.30	13.4
12.5	19.00	69.80	103	9.03	19.9
15	22.90	>74	98	16.46	27.1

The mode and the range are both given in table 3 above as these are methods of quantifying both down tank advection and longitudinal dispersion as required by objective O5.2.3. Mode shows where the majority of particles were within the tank and represents the ability of the flow field to move the particles down tank. This is a description of advection. The effect of differing wave magnitudes on advection was complex. After 15 seconds of wave movement down the tank the 5m^{3}s^{-1} and 10m^{3}s^{-1} waves had a very minimal impact on advection with a difference of -1 and 1m from the 2.5m^{3}s^{-1} no wave scenario respectively. The 12.5m^{3}s^{-1} and 15 m^{3}s^{-1} showed an increased advection of 12 and 7m respectively. With the 15 m^{3}s^{-1} scenario having had some particles exit the tank this will be a slight under estimate. Even so the increase in advection is clearly not a linear relationship with flow magnitude. Excluding the 5 m^{3}s^{-1} wave, range or longitudinal dispersion does appear to increase at a closer to linear rate with magnitude. The 15m^{3}s^{-1} produced a range more than 15.8m greater than the no wave scenario, the 12.5m^{3}s^{-1}, 11.6m, the 10m^{3}s^{-1} 7.8m. 5m^{3}s^{-1} is a slight anomaly, but as mentioned before this is because the wave in this scenario had not yet caught up with some of the particles. Whilst the wave peak had not yet outpaced all the particles the effect is to reduce the range. Once the wave had moved beyond the particles the range rapidly increases.

The second variable that was altered was wave duration. Each of the following wave scenarios had a magnitude of $10m^3s^{-1}$ maintained over durations of 2, 3, 5 and 10 seconds.

Wave duration scenarios 10 seconds after wave release

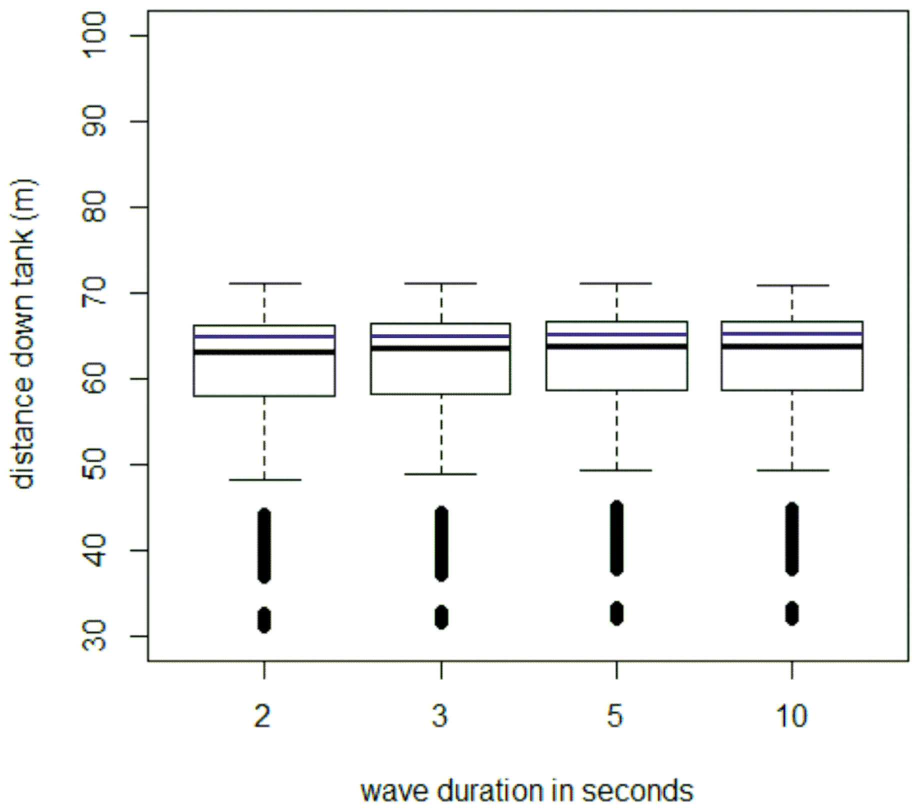


Figure 5-13. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers and the blue lines on the 4 scenarios with waves show the position of the wave peak. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

For the wave duration scenario set particle distribution 10 seconds after the wave has been released is close to identical for each scenario owing to the same peak magnitude of each wave and down tank progression at between 65 and 66m. At this point in time there was a minor difference in particle distributions between the four scenarios. This can quickly be illustrated by the median down tank position of the particles; 63.2m for the 2 second wave, 63.6m for the 3 second, 63.7m for the 5 second, and 63.8m for the 10 second duration wave.

Wave duration scenarios 15 seconds after wave release

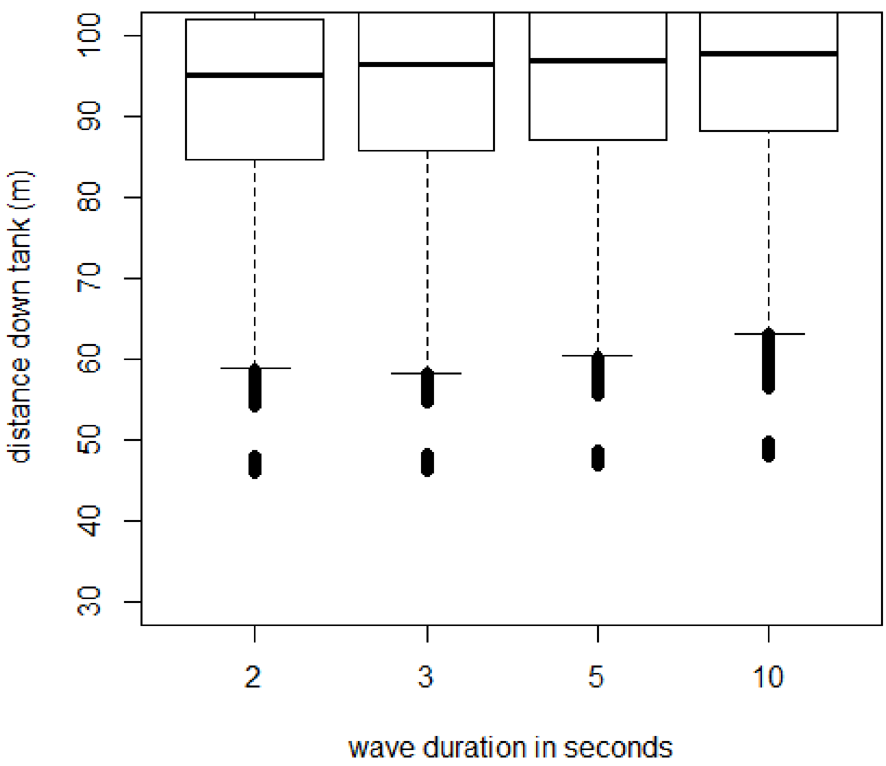


Figure 5-14. Particle distributions boxplots down the tank (y axis) 15 seconds after the wave was released for each of the five wave magnitude scenarios. The black dots represent outliers. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

15 seconds after the wave release the wave peak and a proportion of the particles had moved past the 100m point down tank (Figure 5.13, Table 5-6). The longer duration waves moved the particles further down the tank, both in number of particles that had left the tank, and overall distribution of particles.

Table 5-4. Percentage of particles that that have moved beyond 100m at 15 seconds after release for the differing wave durations.

Wave duration (s)	% of particles > 100m
2	32
3	34
5	36
10	38

The relationship between wave duration and particle movement is not linear. A 2 second duration wave moved 32% of particles beyond the 100m mark whilst a 10 second duration wave moved 38% as can been seen in table 5-6.

Comparison Between Higher Magnitude and Longer Duration Waves

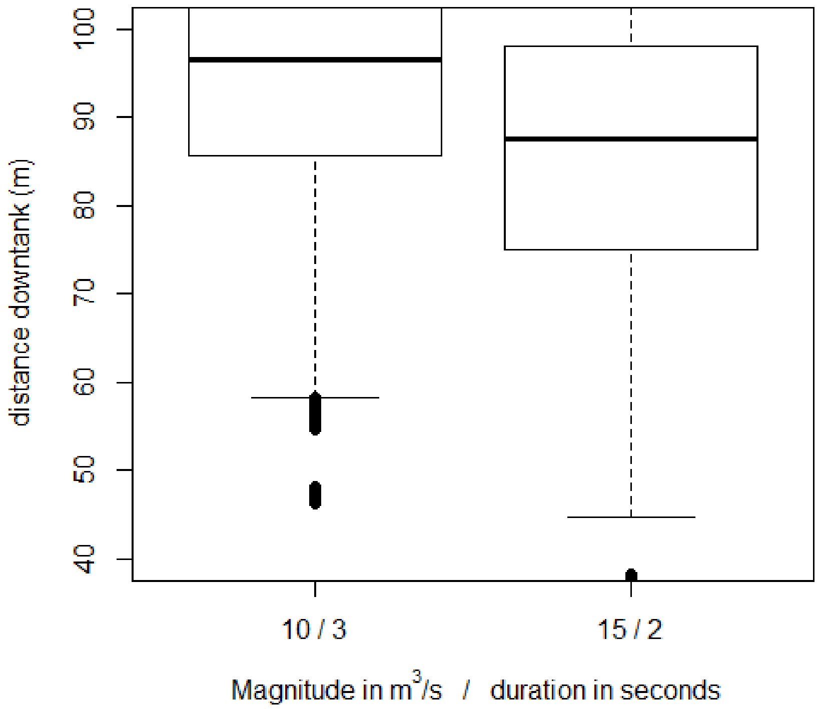


Figure 5-15. shows two boxplots on the Y axis representing the ditribution of particles down the tank at 15 seconds after wave release. The left hand boxplot shows the 3 second duration wave with a $10m^3s^{-1}$ magnitude and the right hand shows the 2 second duration $15m^3s^{-1}$ magnitude. Each boxplot displays the median, interquartile range, +/- 1.5 IQR and outliers.

Both scenarios presented in Figure 15 have a total wave volume of 30m^3. One entered the tank as 10m^{3}s^{-1} over 3 seconds, the other as 15m^{3}s^{-1} over 2 seconds. Comparing these two scenarios, a longer duration wave moved the particles further than a shorter duration wave with a higher magnitude. The longer duration wave moved 34% of the particles from the tank, and the higher magnitude moves 16.5. 100m is too short a tank to give an accurate measure of the total range over this time span so the difference between wave treatments on longitudinal dispersion is difficult to determine. Over the distance presented the 15m^{3}s^{-1} wave generates the longer range at 74m against the 58m for the 3 second wave.

5.6 Discussion

5.6.1 The Agreement between the CFDM and the Flume and River Results

The 5m model was designed to give a direct comparison between the flume results of chapter 4 and the CFDM method employed here. A comparison between the results for the experiments is given in table 5 below.

Table 5-5. Comparison between 5m model and flume tank results. Data is drawn from the 12.5l base scenario shown in figure 20 of chapter 4 for the flume. The CFDM data is identical to that shown in figure 1.

	CFDM model	Flume tank
Time for wave to transverse tank (s)	5.22	5
Wave rising limb length / height at 2m point (m)	0.12 / 0.04	0.1 / 0.05
Time for dye/particles to reach 2m mark (s)	2.34	1.2
Distance down tank at which the wave front outpaces the dye (m)	1.9	2.5

The wave progression results presented in table 2 show that the fluid dynamics generated within the model are a good approximation of that in the physical flume tank of chapter 4. The physical characteristics of the wave are comparable between both the CFDM and the flume tank. The two waves took 5.22 and 5 seconds to travel down the 5m tank respectively, the rising limb length was 0.12m and 0.1m, and the wave height was 0.04 and 0.05 in the CFDM and the flume tank.

There is a considerable variation in the effect of the wave upon the tracked particles in the CFDM and the dye in the flume tank. Two measures of this are presented in table 5 above. Firstly the CFDM tracked particles take 1.14 seconds longer to cross the 2m down tank point than the dye in the flume tank. This is a 49% increase in time. Secondly in the flume tank dye is entrained by the wave front and carried with the wave front until the 2.5m down tank point, in the CFDM the particles are out paced by the wave front at 1.9m. Whilst the pattern of particle distribution shown in figure 5.5 is similar that seen in figure 4.10 of chapter 4 the numbers given in table 2 do show a difference between the results.

The likely explanation for this result is the transfer of the wave as a movement of energy rather than particle motion. Low amplitude solitary waves in deeper water have been mathematically shown to have a transient upward or downward impact on particle motion during wave passage (Constantin and Escher 2007). The wave produced by the CFDM 5m model whilst still exhibiting a forward motion in the tracked particles is closer in behaviour to the shallow water energy waves originally detailed by (Longuet-Higgins 1953; Lighthill and Whitham 1955). A limitation of the CFDM method must therefore be acknowledged. Longitudinal particle process of advection is probably underestimated in the river reach model as they are clearly under estimated in the 5m model, the process of particle diffusion is entirely absent in both models.

A second possible factor is the omission of a random walk model to simulate diffusion. Whilst the purpose of this model was for the flow field itself to produce these processes at the cell by cell scale a random walk model could have been employed if more computing power were available. Within the range of particle specific results a random walk model could produce it is probable that some particles would have moved further down tank within an individual time step. Both Bocksell and Loth (2001) and MacInnes and Bracco (1992) suggest that random walk methods can over estimate dispersion if there is a high velocity gradient, with particles collecting in low velocity areas. Dehbi (2008) notes that particles tend to gather near walls. It is possible that wall affects would be amplified if random walk had been used and this in turn might increase longitudinal dispersion. Advection however would be reduced and the rate at which the wave out paces the dye would remain an underestimate compared with the flume tank results.

With the lack of a dispersion equation the model effectively only demonstrates the role of advection in moving particles and the impact of an extreme wall roughness on the cells immediately next to the walls in the reach model. Consequently the particle distributions seen cannot be related to a real river or given as a good validation of the flume. The process of faster advective flows near the surface of the wave front and the kinematic nature of the wave seen in the flume have been replicated at the larger scale but beyond this the CDFM approach as implemented has had limited success in achieving Aim A5.1.

The wave celerity velocity, at 6.76ms^{-1}, in the river reach model is considerably greater than that recorded at each of the field sites in chapter 3. The reason for this is firstly that the CFDM model is a straight idealised channel promoting faster flows (Fischer 1969) and secondly the water is 1m deep or greater in the model reducing the influence of friction generated by boundaries. In the field site rivers, 1m depth is often only achieved at the mid channel, and much of the channel is affected by a pool riffle system which acts as a control on flow velocities at lower depths (Beven 1979) exhibiting a stronger edge effect on the water (Gilvear 1989). Regardless of the cause of this result in the model it does pose a severe restriction to the models usefulness. The primary purpose of the model was to demonstrate that the particle mixing patterns seen in the flume tank of chapter 4 were replicable at a larger scale of 100m. A pattern of particles moving with the wave front in the upper water column has been seen. However, the secondary purpose (A5.2) of quantifying the mixing results generated by the wave has limited applicability to the real world situation, since the model is not a realistic representation of even a simplified river reach due to the wave celerity involved.

5.6.2 The Classification of Waves

The nature of the wave in the flume tank was discussed in chapter 4. Classifying a wave within the literature is not necessary for a discussion on dilution, but does allow the results to be set within comparable literature if a neat classification can be found. Finding a neat classification is not always the case however and papers dealing with this subject can be found back as far as Le Mahaute *et al.* (1968). The river reach model presented here shows awave that travels over 100m during 15 seconds and provides more information on the nature of the wave. In a similar fashion to the 5m model result, the wave has kinematic attributes in that energy is transferred forward to a greater extent than individual particles. In the literature such waves have been termed translatory waves (Elíasson *et al.* 2007). Like the 5m tank wave, the wave amplitude is greater than the depth making it a shallow water wave with a non-symmetrical profile. The rising limb is steep much like the wave profiles generated in the field chapter. It therefore cannot fit the model of a solitary wave, that moves at a consistent speed, and has a very limited effect on longitudinal particle motion as described by Whitman (1980) and Hsu *et al.* (2012) which itself is a subset of orbital waves. Whilst the wave has to an extent been categorised within the wave literature it is difficult to compare the form of the wave with other CFD studies as few examine single waves in confined channels but instead focus on deep water waves, that is waves where the amplitude or length is smaller than the water depth (Boo 2002; Ryu *et al.* 2003; Koo and Kim 2004; Zhang *et al.* 2006), or waves around man-made structures (Park *et al.* 2003). Waves generated in the field environment within chapter 3 have been shown to both decelerate and accelerate but this was largely due to changes in the river channel morphology and therefore direct comparisons cannot be drawn.

5.6.3 Particle Distribution in the River Reach Model

Vertical mixing in both the 5m and river reach models is limited. This is clearly visible in figure 5.8, but also discernible in figures 5.5 and 5.6. A particle in the top 0.1m of the water column remains there throughout the experiment. There are three reasons for this. The first is the lack of a diffusion model. As stated in the method a random walk model was not employed due to computational requirements. Secondly vertical mixing is a function of bed friction induced turbulence (Rutherford 1994). Whilst the bed and banks are defined with a roughness constant this does not produce a clear increase in turbulence through the water column as would be expected in a natural river bed. Thirdly however is the presence of the wave itself. Figure 5.7 shows that velocity increases with height in the water column in a non-linear fashion during wave passage. It therefore follows that the vertical distribution of particles in the water column increases in a non-linear fashion. This was a question raised in the results of the flume chapter that is now answered. Literature dealing with this topic typically does not consider rapid changes in flow or transient conditions and therefore does not comment on flow derived changes in vertical mixing (Rutherford 1994).

Transverse distribution of particles is also not linear. Particles in figure 5.9 are slowed near the banks. This pattern is well documented in the literature with recent papers from Boxall and Guymer (2003), Zhang and Zhu (Seo *et al.* 2006; 2011) and Dow *et al.* (2009) showing a greater movement of particles in the centre of the river channel. Because the 100m tank is a simple rectangle the particle curve is even, if the channel were to include meanders or flow structures this pattern would be far more chaotic. Whilst the distribution of particles down the tank reflects the flow velocities there is minimal evidence of particles moving laterally prior to wave passage. This is shows that the flow prior to wave passage in the tank is close to laminar. According to Rutherford (1994) turbulent transverse mixing usually takes 100 times the depth of the water column to complete. Given that the tank terminates within 30m by the point that the wave has passed the particles it is not surprising that limited transverse mixing is seen. Whilst distances of 10s or 100s of meters and timescales of seconds through minutes are of academic interest they are of limited importance when considering pollution management in one sense. However these changes in particle distribution are important when considering the length of river which they are distributed over by the wave. This is commented on further in the next subsection. Flume results in chapter 2 showed a significantly higher turbulence and resultant mixing activity in the wake of the wave, this result corroborates well with the CFDM results shown here.

5.6.4 The Effect of Wave Magnitude and Duration on Particle Distribution

Longitudinal dispersion and advection both increased with magnitude of the wave over a 15 second time frame. Longitudinal dispersion is the crucial measure for increasing dilution, as the greater the down river spread of particles, the lower the concentration at any given point. The results presented in this chapter indicate a close to linear relationship between magnitude and longitudinal dispersion. However, they are limited by the time duration and distance of the model. Higher magnitude waves move faster and so within the 15 second time the increase in longitudinal dispersion could be simply a function of wave speed. To obtain a true reflection of the impact of magnitude on dispersion the model would need to have a great enough length and run time to allow the particles to reach a true post wave interaction state. This same limitation applies to the comparison between a higher magnitude and longer duration wave of the same total volume. In the flume tank of chapter 4 this result was also limited by the measurement point and short time frame over which data was recorded. In the results presented in this chapter the longer duration wave has the greater impact on particle dispersion over the time and distance presented. With the particles still being under the direct influence of the wave 15 seconds after wave release and only 100m down the tank a fair assessment of the different wave profiles presented cannot be made, a longer model with a greater run time would be required.

Graf (1995) reported the opposite trend in longitudinal dispersion from dye tests during wave releases in the magnitude of hundreds of cumecs within Colorado river than that presented here. The explanation given is that in higher flows the role of bed friction reduces consequently less dye goes into temporary storage and dispersion is lower. In figures 5.8 and 5.9 particles near the bed and banks are clearly slowed by the homogenous boundary friction conditions applied in the model. Whilst the boundary roughness conditions are limited to the boundary cells they are very effective at trapping particles. If the boundary roughness was reduced particles would be slowed less effectively and consequently the dispersion would be reduced. However, if a dispersion model had been employed the random scattering would increase the incidence of particles coming into contact with the boundary layer and thus slowing more particles and increasing the longitudinal dispersion. Second, the 5m model had smooth surfaces but still showed an increase in longitudinal dispersion with particles being dragged forward by the wave, therefore it would seem simplistic to suggest that the difference between the result presented in this chapter and Graf (1995) is purely determined by boundary roughness conditions. In the Graf (1995) paper the stage river varies between 5 and 9m and flow changes are an order of magnitude greater than those in the reach model, or the field sites described in chapter 3 raising a question as to the comparability. It is possible that flow can be raised to such an extent that there is a step change in the relationship between the boundary conditions and particle dispersion. Whilst this chapter has improved upon the results of chapter 4 and shown that waves have increased longitudinal dispersion the lack of an ADE and complex bed topography limits the results application to a real river. The key question is whether the increase in advection and stratification of flow generated by the wave is outweighed by a change in movement of particles to temporary storage within the dead zone near the bed? This question would be a worthwhile topic for future studies.

5.6.5 The CFDM against an Advective Dispersive Equation Methodological Approach

In the introduction section of this chapter it was asserted that a 3D CFDM approach to modelling this problem was more suitable than using a 1D wave equation and an advective dispersive equation (ADE). Now that the results have been presented it is worth re-examining this claim. ADE methods commonly use average flow velocity as a function to both derive the dispersion coefficient and to solve the equation as a whole. The velocity data presented in this chapter shows a vertical stratification of flow velocities ranging from 6.93ms^{-1} at the wave peak surface down to 1.4ms^{-1} at the bed. This is a significant velocity range, an ADE method would mean these values for its input, assuming the whole river channel was treated as a single volume. Problems with ADE methods and high velocity differential situations have been noted by Baeumer *et al.* (2001) and Healy and Russell (1993). Whilst both authors propose mathematical amendments to common ADE formulations neither deal explicitly with variations in flow or super critical flows. Ideally a 1D model using an ADE would be built for comparison. Whilst an ADE that incorporates the findings of chapters 4 and 5 is beyond the resources available a simple 1D model that considers a field site from chapter 3 is presented in chapter 6. Despite the limitations of using a 1D flow model it is likely that such a model would have produced an accurate wave velocity as such models are regularly used for flow estimates. In many respects a 1D flow model might have produced a more accurate and meaningful out come from the perspective of informing management, even if such a model was potentially inaccurate in representing changes in longitudinal dispersion. A model scaled to the size of a catchment, or a longer reach of river with realistic wave celerities and flow velocities would produce a result that could be compared with the field. The model presented in this chapter has failed to do this. A second reason to use a 1D flow model is scale. An argument has been made that 121m was too short for the model presented in this chapter to meet its intended aim. The results comparing a high magnitude wave to a high duration wave were limited by the length of the tank and a 200m or even greater model would be desired. Such a model would be computationally feasible in 1D with scale commonly being a factor when choosing a modelling system (Hardy, 2006).

5.7 Conclusion

The CFDM was successful in so far as it replicated the particle distribution patterns seen in the flume tank in chapter 4. Over 5m the wave caught the particles and dragged them down the tank increasing longitudinal dispersion partially addressing aim A5.1. The vertical stratification of dye and flow seen in the flume was present in the 5m model as was the velocity of the wave, however the down tank particle movement was underestimated suggesting a flaw in the modelling of advection, additionally dispersion was not considered. In the longer Reach scale model the wave also moved down the tank caught the particles and dragged those in the upper water column toward the outlet. The wave moved considerably faster than those seen in either tank or field but the particle distribution pattern was consistent with what had been seen in both the 5m model and flume. Longitudinal dispersion was quantified as a wave moved down the Reach model showing an increase with both magnitude and duration of the wave fulfilling O5.2.1. A comparison between longer duration and higher magnitude waves was also presented, though the comparison was inconclusive as 110m of tank was insufficient to demonstrate the difference in results. The patterns seen in the flume tank of chapter 4 have been replicated within this chapter at a much greater scale. It is therefore reasonable to assert that the flume tank results are valid at higher scales and that waves would increase longitudinal dispersion in a river system in a comparable manner. The requirements of aim A5.3 have not been met, it is difficult to describe the Reach model as a sound approximation of even a simplified river channel as the wave velocities were too high, the wall conditions were questionable and diffusion was not modelled. In a similar manner to chapter 4, chapter 5 has provided lots of information on the impact of a wave on pollution dispersion and dilution but has not been a close enough approximation of a river system to make the results directly applicable to the industry focused questions directing this thesis.

Chapter 6. Discussion and Application

The purpose of this chapter is to draw together the findings and threads from the previous chapters and present a cohesive assessment of the key questions and aims laid down in chapter 1.

The question driving this thesis is; is it feasible to mitigate a transient water pollution incident by releasing a wave of water from a reservoir?

The general finding of this thesis is that it is feasible for a release wave to catch up with and dilute a pollution incident. Dilution is achieved through both an increase in local water volume and longitudinal dispersion. As a result this is a realistic mitigation strategy within the catchment scales and time frames studied.

To explain both this question and its answer further this chapter will deal with the three key questions laid down in chapter 1. This chapter is split into seven sections with the first three dealing with each question. Within each question initially the question itself is discussed, additions to the scientific literature are outlined, then alternative approaches and extensions to the study are considered. The fourth section details the design of a 1D flow model needed to overcome short comings in the three experimental chapters. The fifth section outlines the key lessons of this work with the object of giving a concise guidance to water managers. The sixth section is concerned with the challenges that a water manager wishing to implement reservoir releases as a mitigation system will have to consider and is additional to the fifth section. Finally an overall conclusion to the thesis and evaluation of the relative success of the three study approaches is given in the seventh section.

6.1 Key Question 1; How quickly can a wave of water released from a reservoir catch a slug of polluted water?

6.1.1 Summary of Wave Progression Results and Discussion

The data presented in the three data chapters clearly demonstrates that a wave can catch a pollution slug under the conditions tested. In chapter 3, the field study waves were recorded to move down a river at a mean velocity between 0.86ms^{-1} and 1.63ms^{-1}. Flow velocity control measurements taken outside of release experiments were consistently lower with a mean velocity of 0.35ms^{-1}. The dye tracer experiment showed that a wave would have little problem catching a slug of polluted water within the River Holme. The wave moved at a mean velocity of 1.09ms^{-1}. The dye in the control experiment, with the river flowing at 99.4% percentile conditions, had a mean velocity of 0.185ms^{-1}.

Waves released in the flume experiment (chapter 4) were found to be kinematic in nature and caught up with dye or substitute pollutant in every scenario tested. A 12.5l wave was found to traverse a 5m tank within 5 seconds. Rhodamine dye, oil, and kaolinite input 1m down the flume were found to move at the rate of the tank baseflow; $0.035ms^{-1}$ when not under the direct effect of the wave. In the longer Reach Model described in the CFDM (chapter 5) waves with a magnitude of between $5m^3s^{-1}$ and $15m^3s^{-1}$ were found to traverse 100m of the tank faster than the tracked particles injected 25m ahead of them which are caught by the wave within 5 seconds in each scenario tested. Aim A1 of this thesis was to measure wave speed and compare to this baseflow velocity; this aim has been fulfilled.

From a management perspective if a wave is expected to catch up with a pollution slug it is important to know how long this will take. The dye experiment in chapter 3 has shown that on the Holme catchment a wave released from Digley Reservoir would catch a pollution slug released at Neiley STW in 2 hours 15 minutes. Based on the dye velocity during the control experiment, a water manager has a maximum of 6 hours 30 minutes to dilute a pollution slug released from Neiley STW before it entered the River Colne at the confluence. Whether this is a meaningful timescale will be discussed in the later sections management.

6.1.2 Contribution to the Literature

The idea that waves have higher velocities than baseflows is generally accepted by hydrologists. Gilvear (1989) studies this topic in detail releasing 33 waves from reservoirs in multiple river systems. Furthermore, Gooseff *et al.* (2008) state in general terms that higher magnitude flows generate higher flow velocities. The nine release experiments conducted in this thesis provide additional examples of wave progression down river courses of varying scales. Only one dye experiment carried out during a reservoir release has been found within the literature which was conducted on the River Seine and did not have a control (Barillier *et al.* 1993). The dye experiment on the Holme both has a control and is on a smaller scale river. The management of a large river system like the Seine is complex and only applicable to catchments of a similar scale. There are many rivers, both in the UK and the wider world, of a comparable scale to the Holme.

6.1.3 Alternative Approaches: 1D Flood Route Modelling

A key limitation of the dye experiment was that it only considered one set of flow conditions in the control experiment, a flow of $0.51m^3s^{-1}$. To compensate for this and provide a wider evidence basis for making operational guidance recommendations a 1D model estimating flow velocities for various flow scenarios is detailed later in this chapter. The model produced, whilst at catchment scale, is limited to a steady state flow model intended to produce baseflow velocities for estimating pollution progression downstream. It does not model either a release wave or pollution, there is however a case for a model that simulates these processes. A 1D model could be validated against the field data provided in this thesis. Being able to run numerous flow scenarios would be a key benefit of such an approach, the progression of a wave on a river could be tested under differing antecedent and release conditions. Such results, if accurate, would produce a more comprehensive understanding of the relationships between wave celerity and factors such as magnitude or antecedent flow. This would assist managers in deriving accurate response times. Such models require high quality topological data (Bates *et al.* 2003; Di Baldassarre and Uhlenbrook 2012). There are contemporary papers that report good agreement between simulation of a flood wave and discharges measured in the river for both controlled releases (Xia *et al.* 2012) and dam-break scenarios (Xu *et al.* 2012). However there are also other papers that question the accuracy of predictive methods such as the Muskingum-Cunge (Fenton 2011).

6.2Key Question 2; How much dilution can be achieved?

6.2.1 Evidence for Dilution

Dilution of the two primary water quality parameters, NH_4 and conductivity was achieved in the majority of field experiments. The peak quantity of dilution achieved for both these parameters ranged between 30-59% on the River Holme with similar results on the River Ryburn and on the Don at Stocksbridge. On the Don at Blackburn Meadows, 25km downstream of the release reservoir, no dilution was detected in the first event and limited dilution of NH_4 during the second. The lack of dilution at the Blackburn Meadows site suggests two conclusions; first, the low NH_4 water from Underbank reservoir was not transmitted 25km down river within the time period studied. Or second, the water was transmitted but mixing with high in river NH_4 concentrations resulted in negligible dilution despite the rise in discharge. A water manager should therefore be aware that a dilution system may only work within a set distance downstream of a given reservoir.

Due to the saturation of the river system and the released water, reservoir releases were found to have a minimal impact on DO levels. These results were compared to similar reports by Bariller *et al.* (1993) and Malatre and Gosse (1995) in the discussion section of chapter 3. During the dye experiment concentrations of Rhodamine WT were significantly lower than those measured during the control. In both the control and wave release experiment sampling for the dye test was stopped prematurely, consequently a complete chemograph was not presented. Whilst this is a limitation, over the four hours of dye measurements during the release experiment concentrations rose far slower than they did over the two hours of the control, with the recorded peak being only 20% of that recorded in the control. Dilution within both the CFDM experiments was quantified as an increase in longitudinal dispersion and so will be dealt with under the third key question.

6.2.2 Can Dilution be Predicted?

Witnessing a dilution of in river concentrations in response to flow is of no surprise. Dilution factors have been used in pollution mitigation studies before with a recent example being Ort and Siegrist (2009) who used conductivity probes to measure sewage effluent dilution. Xu (2014) calculated downstream dilution of sedimentation rates on the Yellow River and Farhadian *et al.* (2014) described rivers as having an assimilation capacity in their ability to dilute pollution. The real question however is; for a given set of input conditions, how much dilution will be achieved? R^2 values for the relationship between flow and NH_4 and conductivity, reported in chapter 3 and the polynomial models that produced them were variable. A consistent relationship between flow and dilution cannot, thus, be given. Rather, other factors including the quality of the diluting water and the degradation of the pollutant, must be quantified. A water manager would have to implement a water quality monitoring program to gather the necessary data if an accurate prediction of dilution for a given wave was desired. A simple mass balance approach can be used to estimate dilution, an example of this is given in key lessons of the work section of this chapter.

6.2.3 Contribution to the Literature

In chapter 3 of this thesis it was noted that only two papers replicated their results and report multiple reservoir releases; these are Krein and De Sutter (2001) who examined two, and Kurtenbach *et al.* (2006) who examined five. In this thesis nine releases and the dilution they generate are reported. These cover three catchments of different scales, four seasons, and different forms of release wave profile included multi-peaked scour tests. This thesis has both delivered replication of methods and considered a variety of scenarios to a greater degree than any paper yet published in the field. This increased breadth and depth of study has allowed subjects such as the reliability of dilution and the impact of variables such as seasonal change to be commented upon.

6.2.4 Alternative Approaches; Dilution Modelling, Other Forms of Pollution

The data presented in this thesis generally depict good water quality with the majority of experiments involving pollutant levels below EQS. In a real pollution incident, water quality would be significantly worse and different dilution factors could be achieved. As has been discussed in chapter 5, it is unclear as to whether an ADE based method for modelling dilution could account for rapid changes in discharge and flow velocity. Despite these limitations there are a few papers that make a case for an non-steady state ADE based study at the catchment or reach scale. Chapra and Whitehead (2009) provide a catchment scale model that modelled longitudinal dispersion and hourly flow changes in response to mine contaminated dam releases or break scenarios. Minimal catchment water quality data was provided and the model was not validated. Furthermore the model was built on the Fischer (1973) equations for dispersion, which includes averaging velocity profiles. At a higher temporal scale, 10 minute steps, but limited to a river reach, Cristea *et al.* (2010) reported an ADE based model dealing with nitrate pollution spills within the River Swale. Whilst this paper did report field validation, a depth average velocity equation was used and no rapid fluctuations in flow occurred. Progressively a paper by Mannina and Viviani (2010) concerned with CSO discharges during flashy storm events did solve an ADE and dispersion based equation set at the grid level, with a rapid change in flow at a temporal scale of tens of minutes. The hydrograph modelled and the decline in DO and spikes in BOD and NH_4 concentration simulated agree to an extent with the field data shown. Whilst the data input requirements are very intensive these papers do make a case for a further study examining an ADE based method. Such a study would require a number of problems to be overcome. The results from the Don suggest that releases from Underbank reservoir can achieve very high wave celerities. Such extreme conditions may invalidate any depth averaged dispersion based equation. This problem would need to be considered at a mathematical level against the equations presented by Fischer (1973) and then tested empirically by constructing ADE based models at the scale of the flume, the reach model presented in chapter 5 and the field sites presented in chapter 3. The waves generated on the Holme and the Ryburn catchments are more comparable in terms of rising limb duration, 30 to 60 minutes, to that presented by Mannina and Viviani (2010), although of much higher magnitude, up to $4m^3s^{-1}$ on the Holme and only up to $0.4m^3s^{-1}$ in the paper under discussion. It therefore seems likely that an ADE based method could be used to model the Holme and Ryburn catchments if the resources were available.

The field experiments only considered NH_4, DO and conductivity. Water quality parameters such as phosphorus, coliforms, metals, and complex compounds such as medical products could all be measured. This thesis has assumed that conductivity and NH_4 are broadly representative of water quality as a whole however this assertion could be tested. A future program of releases could be carried out examining the dilution of a wider range of water quality parameters.

6.3 Key Question 3, What Mixing Processes Occur When a Wave Catches Polluted Water?

6.3.1 Longitudinal Dispersion and Vertical Stratification

The mixing process of greatest relevance to dilution is longitudinal dispersion. The more dispersed down a river course a pollutant is the lower the local concentration will be. In both the flume tank of chapter 4 and the CFDM of chapter 5 waves released from one end of a rectangular conduit flowed down a channel, caught up with, and passed through, a concentration of dye, or tracked particles. In the CFDM there was a clear increase in longitudinal dispersion with waves of increasing magnitude or duration. In the flume longitudinal dispersion decreased, though this is largely a function of the very short down tank distance and time scale the experiment was conducted over. In both the flume and CFDM water velocities, and dye, or tracked particle motion, were found to be of a much higher magnitude in the upper water column. For instance, in the flume tank velocities exceed 0.5ms^{-1} above 4cm in the water column and were below 0.2ms^{-1} within 1cm of the tank bottom, the effect of which upon the dye was to carry it 2m down the tank but with a strong vertical stratification. This result was consistent with the dead zone model detailed by Beer *et al*. (1982) which argues that velocities near the bed can be slow enough to be considered temporary storage. Over the 110m length of the CFDM tank waves of 10m^{3}s^{-1} and 15m^{3}s^{-1} increased longitudinal dispersion by 13.4% and 27.1% respectively over that of the particles in a steady 2.5m^{3}s^{-1} flow. Whilst this is likely an underestimate due to the simplicity of the model it is a consistent pattern within the flume experiments.

6.3.2 Model Results and their Application to the Field Environment

For the results of the flume and the CFDM to be important they need to be an accurate portrayal of what occurs in the river. It is necessary then to compare the results from chapters 4 and 5 with the results from chapter 3. The longitudinal dispersion based results from the field experiment are limited due to the point based method for collecting water quality samples. The dye results reported during the control test produced a chemograph with a far steeper rising limb and kurtosis than that from the wave experiment. Waves can therefore be said to increase longitudinal dispersion in the field. If the waves produced in the tank experiments are similar to those in the field it is reasonable to expect them to produce similar mixing patterns. The magnitude, rising limbs and velocities of the waves in the flume and CFDM can be compared with those from the field.

In the CFDM, the waves that achieved a positive increase in dispersion were of a magnitude of 10m^{3}s^{-1} or greater. Whilst the 5m^{3}s^{-1} wave would achieve this in a longer tank it clearly has a lower impact. Of the field flows tested only one on the Don catchment approached these flow magnitudes with a downstream peak of $9m^3s^{-1}$. Other than magnitude, the second feature of the waves in both the flume tank and the CFDM was the presence of high supercritical velocities within the wave front. Froude numbers calculated for the flows in chapter 3 suggest that supercritical conditions would only occur in shallow riffle reaches. Coupled with this the wave front velocities seen in the flume and field were close to $1ms^{-1}$. In the CFDM velocities of $6.7ms^{-1}$ were recorded showing that the model over estimates celerity. On this basis it is difficult to consider the waves from both the flume and the CFDM accurate representations of those in the field. It is not possible to examine the role of supercritical flows in longitudinal dispersion since no papers on this subject have been found.

Both the flume and CFDM are simplifications of a real river channel, neither featured meanders, or rough channel topography at an above grid level. Roughness was only modelled at an abstract level in the CFDM. As such no secondary flows were generated. As discussed in chapters 4 and 5, this limitation makes it difficult to apply the dispersion results to the field situation.

Finally the rising limbs of the waves in both the flume and CFDM are very steep. If the ability of these wave fronts to entrain particles can be realistically achieved within a real river a similar wave front should be seen. Data presented in chapter 3 from the field sites is at a 15 minute resolution, in chapters 4 and 5 the rising limbs are of less than 1 second long. Anecdotal records from the 05/06/13 release on the Don catchment at Stocksbridge note that the rise in stage with wave arrival here was very rapid, with a climb in stage of >30cm occurring within 5 minutes. The rising limbs seen in flume and CFDM can therefore be considered comparable to the wave seen during one of the releases from Underbank reservoir, it is unclear whether they accurately represent the other release experiments.

6.3.3 Contribution to the Literature

In both the flume tank and CFDM experimental results longitudinal dispersion was quantified and visually displayed in detail for waves of differing magnitudes and durations. Such a data set has not been found within the literature surveyed. A paper by Mannina and Viviani (2010) is the closest comparison as the increase in longitudinal dispersion during wave passage is demonstrated in a 1D flow model. This result is however only described with a mathematical function and time series data for a single wave scenario. Both chapter 4 and 5 have identified the wave front as an area of super critical flow that is responsible for entraining dye, tracked particles, and other pollution substitutes. This observation has not been reported in the wider literature.

6.3.4 Alternative Approaches and Extensions to Modelling Dispersion

A 1D flow model with an ADE water quality component could be compared to the results from the flume and CFDM chapters with a similar justification for that given in the Key Question 2 section of this chapter. Replicating the dispersion increases recorded in the CFDM in a 1D model would provide a valuable validation of the method. Another worthwhile extension would be a CFDM ran over a 200m reach, or with an uneven channel closer to that seen in the field environment. The extra distance of a 200m model would allow the waves to exit the model some time before the particles and consequently give a better representation of particle distribution in after wave passage. This would be particularly useful for examining waves of a longer duration.

6.4 The 1D model

1D steady state flow model was constructed with Hec-RAS and Hec GEORAS GIS software in order to determine pollution progression down catchment under different flow scenarios. This was needed in order to address Key Question Q1 as only one baseflow scenario was tested in the dye experiment of chapter 3, a low flow of $0.51ms^{-1}$.

6.4.1 Methodology

The model was based on a 0.25m LIDAR raster grid of the Holme catchment supplied by the EA. Cross sections of the channel were taken at 25m intervals for the length of catchment between 200m downstream of each reservoir and the confluence with the Colne. A distance of 3.6km of the Mag Brook, 3.2km of the New Mill Dye, and 2.7km of the Holme Styes tributaries were modelled. An additional 53 cross sections were added at the major weirs and flow control structures in the river, or at any point in the lower reaches where rapid changes flow velocity occurred. At various points cross sections had to be shifted slightly up or down stream to avoid bridges.

A case could be made for varying Manning's N down the catchment, the sinuosity of the channel and the grading of bed material do change from reach to reach down catchment. However a detailed field study would be required to produce accurate reach by reach Manning's N values and this was not possible given the time constraints. Consequently Manning's N was estimated for the whole catchment using the method presented in Chow (1959). For categories including; the bed material, the degree of irregularity, the variation in cross sectional profile, the importance of in-channel obstructions, vegetation, and meander curvature and angle, a Manning's N estimate is given. These are then totalled to produce a final number. For this model, coarse gravel, a rough irregular channel, with appreciable obstructions, low vegetation and significant meanders produced a Manning's N of 0.0844. This setting for Manning's N produced a flow time comparable to the dye test result (9 hours 35 minutes compared with 9.45 taken by the dye in chapter 3). In addition to this setting a sensitivity analysis was carried out. Manning's N was varied from 0.04 by steps of 0.01 through to 0.12, the results are presented in the following graph.

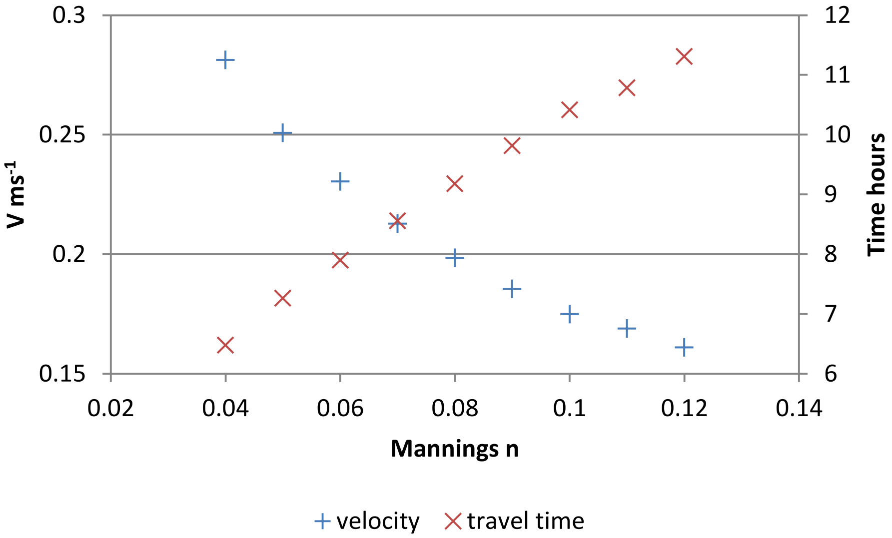


Figure 6-1 the trend in velocity and travel time of a pollution slug moving between the STW input and the confluence with the River Colne 6.5km down river.

A Manning's N higher than 0.0844 would produce a result closer to that seen in the field. The 09.45 hours figure given for dye travel time in chapter 3 was for the arrival of the dye rather than the peak concentration and is therefore likely to be an over estimate for mean velocity. However it is difficult to justify a significantly higher Manning's N based on the substrate encountered in the field. Shaw (1988) recommends a value of 0.05 for rocky streams, so 0.844 could already be considered high. Figure 6.1 shows that the model is sensitive to large changes in Manning's N. Selection of a Manning's N number is to an extent a subjective decision, a different value, or a value that varied over the catchment might produce a significantly different result. The argument for the models validity is that it produced a flow velocity comparable to the field test result.

Three flow scenarios were tested; a flow of $0.5m^3s^{-1}$, $1m^3s^{-1}$ and $3m^3s^{-1}$ as measured at the confluence with the River Colne. $0.5m^3s^{-1}$ was selected both because it could be validated against the dye experiment of chapter 3, but also because as a low flow it is unlikely a release would be carried out below this discharge, as the reservoir stocks would likely be drawn down. $3m^3s^{-1}$ was chosen as a high flow scenario. This flow level puts some areas of the river at bank full and would likely only be generated if the reservoir spill ways were over flowing, reducing the probability that a water manager would carry out a release. $1m^3s^{-1}$ was chosen as it is a regular flow on the River Holme at 76.01% exceedance and a round number that is easy for a water manager to internalise.

Table 6-1 Flow frequencies for each scenario and the corresponding discharges at the three gauged points down the river. All flows are derived from the 2 year data set described in chapter 3.

Frequency of exceedance (%)	Queens Bridge (m^3s^{-1})	STW reach (m^3s^{-1})	Digley Reservoir outflow (m^3s^{-1})
99.38	0.5	0.44	0.09
76.01	1	0.78	0.18
26.38	3	2.26	0.45

The three gauged flows presented in table 6-1 were used to define the relationship between the different reaches of the river. The relative contributions of Holme Styes reservoir, or the Mag Brook tributary, or through flow into the river were not recorded in the data presented in chapter 3. The relative contributions of these sources therefore had to be estimated. The model was primarily concerned with the flow velocities between the STW and the confluence with the Colne. The change in flow between the STW and the Queens Bridge gauge under 76% frequency was $0.24m^3s^{-1}$, therefore $0.2m^3s^{-1}$ was assigned to Mag Brook and $0.04m^3s^{-1}$ to direct inputs. This division was arbitrary and not based on observations. If more time were available rainfall records could be analysed with the terrain model to estimate the relative contributions of each area of the catchment. The main channel of the River Holme was divided into four reaches, each bounded by a confluence with a tributary or the Colne at the bottom. The STW reach flow increased to the level of the Queens Bridge flow after the confluence with Mag Brook. The relative contribution between Mag Brook and direct inputs therefore has no impact on the flow velocities seen between the STW and the Confluence.

6.4.2 Results

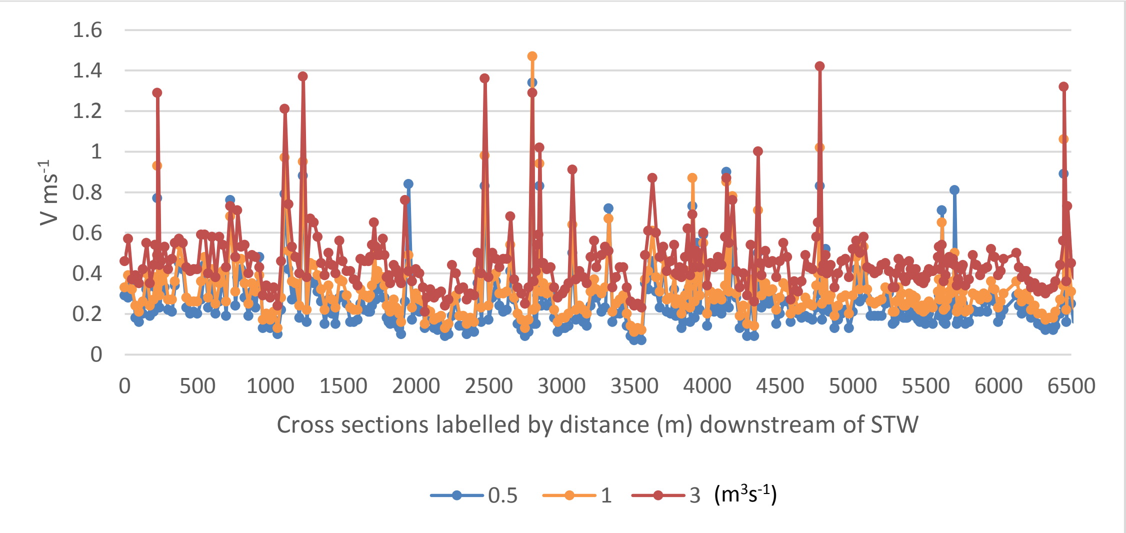


Figure 6-2 velocity by model cross section for the three flow scenarios tested, with each cross section labelled by distance downstream of the STW.

The velocities in figure 6.2 are displayed for each station over the river reach between the STW and the confluence. The velocities were multiplied by distance for each given station and time averaged over the total distance between the STW and confluence to produce the mean velocities given in table 6-2. Whilst station by station velocities could be used to generate a more precise estimate of downstream pollution movement an average is more practical for a water manager who is already using a simplistic estimate of wave velocity.

Table 6-2 Mean velocities between the STW outflow and the confluence for each flow scenario.

Flow scenario	mean V ms^{-1}
$0.5m^3s^{-1}$	0.189
$1m^3s^{-1}$	0.253
$3m^3s^{-1}$	0.396

6.4.3 Discussion

Flow velocities generated within the model were generally higher than those seen in the field despite a high Manning's N. The $0.5m^3s^{-1}$ scenario produced a mean velocity of $0.189ms^{-1}$ compared with the $0.185ms^{-1}$ of the dye experiment. A terrain mesh of 0.25m and a density of cross sections at 25m is high, unless the terrain data is erroneous this is unlikely to be the source of error. A possible cause of this difference is the effect of the weirs. In the model weirs create a pool of slow flowing water followed by a drop in elevation resulting in rapid flow. Despite the additional cross sections included to reduce incorrect modelling of weirs it is likely that velocity is over estimated at these sites.

Velocity increases with discharge unsurprisingly, though not in a linear fashion. At a high flow of $3m^3s^{-1}$ mean velocity remains significantly slower than the waves reported in chapter 3 despite having a higher flow magnitude through the lower river reaches. A map demonstrating the relative movements of a wave release and a pollutions slug moving at the velocity of the baseflow for each scenario is included in the next section.

6.5 Key Lessons of the Work

This section will outline the key findings of the work that are of importance to water managers. Three topics will be covered, each relating to one of the key questions established in chapter 1 of the thesis and evidence presented in the three experimental chapters, they are; response times, the role of the kinematic wave and longitudinal dispersions, and dilution estimates.

6.5.1 Response Times

A key question for a water manager is how long do I have to respond to a pollution incident? This question is related to key question Q1 of this thesis, which was concerned with how quickly a wave would catch a pollution slug. If a compliance point or time, that is a point in time or distance down river by which a pollution incident must have been diluted, exists a manager needs to know how greater time window they have to act in.

The following equation can be used to determine the downriver distance that dilution would occur at for a wave released from a reservoir and a pollution slug at a known point in the river.

$$x = \frac{V_r S_p - V_p S_r}{(V_r - V_p)}$$

Equation 6.1

Where x is the distance from the reservoir where the pollution and the wave meet, V_r is reservoir wave velocity (ms^{-1}), S_p is the pollution start point (m), or current position, V_p is the pollution slug velocity (ms^{-1}), or the mean velocity of the river baseflow, and S_r is the reservoirs position (m) (0, if all other distances are measured from the reservoir).

To use equation 6.1 the current location of the pollution slug is needed. Additionally velocities for both the release wave and the pollution slug have to be estimated. Evidence for both wave velocities and baseflow velocities was reported in the field experiments of chapter 3, and baseflow velocities were modelled and reported in this chapter. Wave velocities reported in chapter 3 ranged from 0.86ms^{-1} to 1.63ms^{-1} with a mean of 1.25ms^{-1} for the six experiments on the River Holme. In the absence of a flood model a water manager needs a rule of thumb number for wave velocity, 1ms^{-1} is proposed. This would account for the slowest waves on the Holme and may approximate results in similar catchments. For the pollution velocity, the mean velocity of the reach of the polluted river must be determined. The dye experiment in chapter 3 yielded a mean velocity of 0.185ms^{-1} for a low flow discharge of 0.51m^3s^{-1}, this is an isolated result however. Point measurements of flow velocity were measured but not set in sufficient context to be meaningful. Flow velocity is influenced by slope, roughness, discharge, and channel topography (Shaw 1988). These factors were considered in the 1D flow model reported in this chapter.

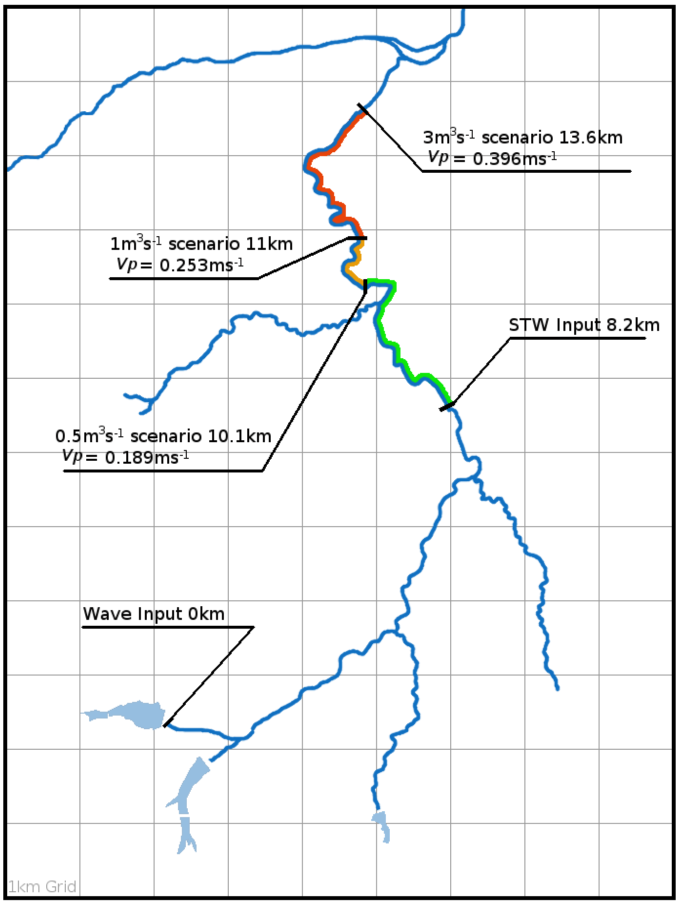


Figure 6-3 The 1km grid Map shows the three scenarios from the 1D model reported earlier in this chapter. A 1ms^{-1} wave was released to catch the pollution slugs. The green line represents the distance travelled by the pollutant traveling at 0.189ms^{-1} before being caught, the orange the 0.253ms^{-1} and red 0.96ms^{-1}.

As figure 6.3 demonstrates, a wave traveling at 1ms^{-1} will catch up with a slug of polluted water moving at baseflow velocities for discharges between 0.5m^{3}s^{-1} and 3m^{3}s^{-1} before it reaches the confluence with the Colne. If a wave is released from the reservoir at the same time as a pollution slug into the river flowing at 3m^{3}s^{-1} the wave will catch the pollution 13.6km from the reservoir after 3 hours and 46 minutes. A mean velocity of 0.396ms^{-1} for a flow of 3m^{3}s^{-1} can be used for V_p in equation 6.1. A water manager would need to have velocity estimates for a range of flow conditions in a river where a pollution incident might be expected to occur.

To estimate the response time, that is the time period a water manager would have to carry out a release before the pollution slug reaches a predetermined compliance point, the following equation can be used;

$$T = \frac{D_r}{V_r} - \frac{D_p}{V_p}$$

Equation 6.2

Where D_r and D_p are the distances to the compliance point (m), that is point x from equation 6.1. Divided by the velocities (ms^{-1}), V_r and V_p for the release wave and the pollutant respectively to give the travel times. The difference between the travel times is response time.

For a scenario where the pollution is traveling at a mean velocity of 0.396ms^{-1} on the River Holme over the 6.5km between the STW and the Colne confluence, the travel time is 4 hours and 33 minutes. For the wave traveling 14.7km at 1ms^{-1} the travel time is 4 hours and 5 minutes giving a response time of 28minutes.

This subsection has provided a workable set of guidance for a water manager detailing the calculation of response times and the dilution point on a river for a given velocity of baseflow and release wave. The evidence underpinning this guidance is drawn entirely from the dye test and wave velocities recorded in chapter 3 and the 1D flow model presented in this chapter. The key limitations of this guidance are; first, wave velocities may vary considerably with catchment characteristics and flow magnitudes, and second using mean velocity as a predictor for pollution motion only accounts for advective transport, ignoring longitudinal dispersion. A catchment scale 2D or 3D model would be required to estimate dispersion.

6.5.2 The Kinematic Wave Effect and Longitudinal Dispersion.

Key Question Q3 was concerned with the mixing processes at work in the river during wave passage. This question can be considered in terms of two processes, the kinematic motion of the wave, and longitudinal dispersion.

The results from the dyed bulk release experiments within the flume of chapter 4 and the differential between the wave celerity and particle velocity in both the flume and CFDM results suggest that waves in channels generated by a rapid rise in discharge are kinematic in nature and propagate primarily as a motion of energy rather than material. Papers such as Glover and Johnson (1974) and Malatre and Gosse (1995) have demonstrated that a wave can move faster than the water quality associated with it upon release. Only limited evidence for this was detected in the field results of chapter 3 with lags in temperature change after the arrival of the wave front only being found on the River Don and one experiment on the Holme. However lags between peak flow and peak dilution of both NH_4 and conductivity were common. The water quality of the wave will affect dilution. The relative contributions of the reservoir and river water quality to the quality of the wave at the point it catches the pollution is therefore important. Over 8.2km of the River Holme the poor NH_4 quality at the reservoir during a summer release was able to transfer downstream to produce reduced dilution at the STW. 26km downstream on the River Don no NH_4 dilution was witnessed in one experiment and delayed dilution during another with the kinematic nature of the wave being a potential explanation as discussed within that chapter. The relative contribution of the river and the reservoir to in wave water quality will be a function of several factors including distance from the reservoir, the velocity of the wave and the role of tributaries. An Insufficient number of water quality probes were deployed down catchment to study this process. It can therefore be stated that the kinematic effect of the wave will influence dilution but to an unknown degree.

Longitudinal dispersion was studied both the chapters 4 and 5 with dye or particle dispersion being quantified during the passage of various wave treatment scenarios. The results from the flume in chapter 4 suggested a reduction in longitudinal dispersion with increasing wave magnitude over a very short distance and time scale. Over a greater distance, longitudinal dispersion was seen to increase with both wave magnitude and duration in the CFDM of chapter 5. Neither experiment set replicated the complex bed topography or roughness of a river, or the catchment scale. Secondly the waves produced in both experiments were supercritical, there is no evidence that the waves in the river catchments studied were supercritical outside a few riffle systems. Therefore, it can be stated that a wave will have an effect on longitudinal dispersion, but insufficient evidence has been collected to quantify this effect at the catchment scale. The recommendations in the preceding and following sections do not account for either kinematic waves or longitudinal dispersion as their effects have not been quantified at the catchment scale.

6.5.3 Dilution Estimates

Estimating dilution for a given wave release is a critical task for a water manager wishing to mitigate dilution. This section can be considered an answer to question Q2; how much dilution can be achieved?

In chapter 2 a conservation of mass equation employed in flow gauging tracer studies was presented;

$$C_f(Q + Q_t) = C_t Q_t + C_b Q$$

Equation 2.1

For a water manager, the final concentration, C_f is of interest. To reflect this, the equation can be transposed too;

$$C_f = \frac{C_p Q_p + C_r Q_r}{Q_p + Q_r}$$

Equation 6.3

Where C_f is the final concentration, C_p is the concentration, in mgl^{-1}, of the polluted in river water, Q_p is the discharge of the polluted water in m^{3}s^{-1}, C_r is the concentration of the diluting water wave in mgl^{-1}, and Q_r is the discharge of the wave flow in m^{3}s^{-1}. Q_p + Q_r gives total flow at the point of dilution.

The two inputs, the polluted baseflow and the wave flow are multiplied by their concentration and then divided by the total flow to give a final concentration. This is a repurposing of an equation designed to estimate the concentration of a continuous tracer input, due to its simplicity it is a suitable management tool. A water manager would need an estimate of the relative contribution of the baseflow and wave to the peak wave discharge at the point of dilution, and they would need an estimate of the concentration of the pollutant under concern for both flow components.

In chapter 3, the concentration of the wave immediately upstream of the STW input was not recorded due to a lack of equipment, as such C_r remains an unknown for that data set. In tables 6-3 and 6-4 below reservoir concentrations of conductivity and NH_4 were used as an estimate for C_r. Equally due to there being not available records of STW output NH_4 and conductivity were only measured in river. In chapter 3 STW concentrations during release experiments were estimated from the mean of samples recorded from 6 other days of the experiment week, these numbers constitute C_p. The limitations of this approach are described in that chapter.

Two hypothetical scenarios are included in the following tables with pollution inputs of 4mgl^{-1} NH_4 and 5000µscm for C_r. One limitation of study presented in chapter 3 was that no major pollution incidents were recorded. Using two higher pollution inputs for equation 6.3 provides an estimate of dilution under a more realistic incident scenario. The flow and reservoir water quality inputs from the 16/11/11 experiment and 29/05/13 experiments were used for scenarios 1 and 2 respectively. 16/11/11 provides a clean reservoir input, 29/05/13 an input with a relatively high concentration of NH_4.

Table 6-3 The components for equation 6.3 for NH_4 data from the River Holme experiments and two hypothetical scenarios.

	Eqn	26/10/11	16/11/11	14/03/12	29/05/13	13/03/13	09/05/13	Sc 1	Sc 2
Reservoir NH_4 mgl^{-1}	Cr	0.122	0.069	0.167	0.351	0.077	0.688	0.07	0.351
STW NH_4 mgl^{-1}	Cp	0.46	0.25	0.41	0.53	0.32	0.46	4	4
Peak discharge m^{3}s^{-1}	Qp+Qr	2.43	2.43	2.58	3.01	2.51	2.43	2.43	3.01
Baseflow m^{3}s^{-1}	Qp	0.58	0.73	0.68	0.86	0.86	0.96	0.73	0.86
Stormflow m^{3}s^{-1}	Qr	1.85	1.70	1.90	2.15	1.65	1.47	1.70	2.15
Final estimate NH^4 mgl^{-1}	Cf	0.20	0.12	0.23	0.40	0.16	0.60	1.25	1.40
Measured peak dilution NH_4 mgl^{-1}		0.26	0.14	0.22	0.22	0.19	0.31	n/a	n/a

Table 6-4 The components for equation 6.3 for conductivity data from the River Holme experiments and two hypothetical scenarios.

	Eqn	26/10/11	16/11/11	14/03/12	29/05/13	13/03/13	09/05/13	Sc 1	Sc 2
Reservoir conductivity uscm	Cr	68	65	73	70	63	75	65	70
STW conductivity	Cp	321.74	367.39	346.94	380.95	390.48	415.71	5000	5000
Peak discharge m^3s^{-1}	Qp+Qr	2.43	2.43	2.58	3.01	2.51	2.43	2.43	3.01
Baseflow m^3s^{-1}	Qp	0.58	0.73	0.68	0.86	0.86	0.96	0.73	0.86
Stormflow m^3s^{-1}	Qr	1.85	1.70	1.90	2.15	1.65	1.47	1.70	2.15
Final estimate conductivity µscm	Cf	129	156	145	159	175	210	1551	1483
Measured peak dilution conductivity µscm		148	169	170	160	164	291	n/a	n/a

The estimated peak dilution concentrations (C_f) for both NH_4 and conductivity are not identical to those measured in the field during experiments. For NH_4 dilution was over estimated by between 0.06 and 0.01mgl^{-1} for 4 of the 6 experiments and under estimated for the two experiments with high NH_4 concentrations at the reservoir by between 0.29 and 0.18mgl^{-1}. The kinematic effect of the wave could be a factor here as discussed in the previous section and chapter 3. The difference in result could also be reflection of tributary inputs between the reservoir and the STW outflow, fluctuations in STW output or non-conservative behaviour of NH_4. Conductivity was estimated with greater accuracy, with errors ranging between 25 and 1µscm.

The two hypothetical scenarios demonstrate that an appreciable degree of dilution can be achieved with 4mgl^{-1} NH_4 dropping to 1.25 mgl^{-1} and 1.40 mgl^{-1} for the two respective waves. This did not bring NH_4 below the 1mgl^{-1} stipulated by the EU 2006/44/EC Fisheries Directive. Conductivity concentrations were reduced to a peak dilution of 1551 and 1483µscm for the two scenarios. The higher reservoir NH_4 in scenario 2 resulted in a peak dilution 0.15mgl^{-1} greater than that of scenario 1, suggesting that in a situation involving a high magnitude pollution incident diffuse sources of NH_4 from the reservoir tributaries are likely to less significant.

Equation 6.3 was for a continuous input and does not consider longitudinal dispersion. Moore (2005) details a series of equations for gulp injection gauging which could be transposed to solve for pollution concentration at a given point in time. Whilst this would be desirable the results from the three data chapters, and chapters 4 and 5 specifically do not provide a sound evidence basis for doing so. The failure of chapters 4 and 5 to provide a governing rule for the effect of a wave on longitudinal dispersion applicable at the catchment scale makes it difficult to modify any dispersion equations in a meaningful manner.

This section has sought to demonstrate dilution estimates using a simple mass balance equation. The evidence used is drawn entirely from water quality and flow data reported in chapter 3 of this thesis and referenced equations. In general it can be noted that the results in chapters 4 and 5 have contributed little to the operational recommendations in this chapter. This is due to the failure of these chapters to produce results that were at an appropriate catchment scale or waves and bed topographies that were demonstrably similar to those seen in the field catchments studies.

6.6 Challenges to using reservoirs as a mitigation tool

The section preceding this one outlined the operational recommendations that can be drawn from the results presented in this thesis. This section will cover the key questions a water manager would have to answer in addition to considering the results and equations presented above.

The following problems are presented in the order they should be considered prior to carrying out a release;

1. Will the release threaten water supply?
2. Will the release cause environmental harm?
3. Is a release necessary?
4. How much water should be released?
5. Is there a preferable wave profile for the release?
6. Is dilution feasible?

6.6.1 Water supply

The release experiments carried out in chapter 3 released a total volumes of water amounting to between 0.24 and 3% of the total reservoir capacity. The Holme reservoir has a capacity of 3443.2ML (Tinsdeall pers comms 2014), releases from the reservoir totalled between 8.6ML and 32ML as calculated from the hydrographs in chapter 3. These releases therefore account for a loss of between 0.24 and 0.93% of capacity over periods ranging between 2 and 5 hours. Underbank reservoir has a total capacity of 2867.5ML. Therefore a release of 87.3ML over 2.5 hours (estimated based on peak discharge measurements pers comms Tindeall 2014) compromises 3% of capacity. For Ryburn, the total capacity is 995.4ML with the release being 8.4ML over 4 hours or 0.84% of this. Whilst these percentages are fractional, an increase in the duration of the release to mitigate a longer pollution incident would multiply this. The economic value of untreated potable water is volatile to use an economic term. Water has a very high value in times of shortage, and a very low value in times of excess

(Gibbons 1986). This value being augmented by both supply and demand fluctuations. Should a water service provider fail to meet demand hose pipe bans and other measures may have to be implemented both damaging public perception of the service provider and potentially lowering profit margins. The risks here are tangible but hard to produce a specific price value for. In systems where monetisation of differing economic outcomes of water resource usage is available, cost benefit analysis and economic assessments of water use can be carried out (Qureshi *et al.* 2007). However without accurate prediction of future supply and demand of water resources such monetisation cannot be achieved. If decisions on whether water should be released from reservoirs are to be made from a water resource conservation stand point this new outflow needs to be considered in the reservoir management model.

Supply and stock management for reservoirs can be based on either forecasts (Zhao *et al.* 2011), or historical data sets (Alaya *et al.* 2003) but in either case can be subject to uncertainties (Zhao *et al.* 2014). There is a risk in spending water and compromising future supply, therefore a reservoir management system needs to categorise risk and set as side water that optionally can be released for dilution.

6.6.2 Reservoirs as Sources of Environmental Harm

Reservoir releases have the potential to cause pollution incidents themselves, either by releasing polluted water, or by suspending polluted sediments from within the river. Both have been documented within the literature with releases on the Yellow river being associated with reservoir derived fluxes of PCPs (Xu *et al.* 2010), and releases on the Seine being attributed to drops of DO with the re-suspension of sediments (Barillier *et al.* 1993). These problems are clearly contextual. In chapter 3 it was observed that during the summer season NH_4 concentrations can increase within the Holme reservoir and affect the quality of a release. It was also clear that the majority of the SSC, as defined by turbidity, within the water release wave was derived from the river rather than the reservoir. Within the River Holme then a manager must consider any summer release of water a nutrient pollution risk, and the potential for any release wave to raise turbidity, having a visual impact on the river. A water manager would have to understand the conditions in each catchment to determine the risk of an adverse impact from releasing a wave from a reservoir and institute operating rules relevant to that site. For instance in the Holme, water quality sampling of the reservoir would be required before a summer release. Furthermore if the visual impact of the increase in turbidity was considered an issue, releases at weekends, or during the summer holidays should be avoided.

A second key potential consequence of releasing water from a reservoir is flooding. Increasing the flow in a river has the potential to overtop the banks and cause flooding if the magnitude of the release and the flow within the river are great enough. The decision to make any release would have to qualified by a flooding assessment based on contemporary data or forecasts for the planned time of release.

6.6.3 The Necessity of a Release

Whether or not a release should be carried out to dilute a pollution incident is a question of exposure time. Given enough time, most pollution incidents will dilute and the effect of longitudinal dispersion and increasing downstream discharge take effect. The purpose of a release is to speed this process and reduce the time a reach of river is exposed to high concentrations of a pollutant. For example on the River Holme flowing at a mean $0.51ms^{-1}$ an incident sourced from Neiley STW would take 9 hours and 45 minutes to reach the confluence with the River Colne and the higher discharge available there. A wave traveling at $1ms^{-1}$ would reach the confluence in 4 hours and 5 minutes reducing the exposure time by 5 hours and 40 minutes assuming significant dilution were to occur. Secondly the wave, by increasing flow velocities, would increase the removal rate of the pollutant from the river reaches affected. Whether this is desirable or not depends on the effect of a pollutant on the environment. If reducing exposure by 5 hours in a reach of river will reduce the likelihood of a fish kill then a release is a viable option.

6.6.4 How much water should be released?

The recommendations in section 6.5 provide a method for estimating peak dilution. Estimating dilution for a longer duration and determining how much water to release is a complex task. A waves peak magnitude will reduce and duration extend as it moves down catchment (Shaw 1988). Determining how rapidly this will occur for a given catchment would require a computational flow model. In the absence of a catchment specific model field observations can be used. On the Holme catchment peak discharge achieved at the STW and Queens Bridge is a mean $1.6m^3s^{-1}$ above baseflow. This varies over a range of $0.8m^3s^{-1}$ as determined by the peak discharge of the reservoir release which itself is affected by the hydraulic head.

Estimating flow duration for the Holme can be done using the following method and observations. With the exception of the 09/05/13 event NH_4 always took at least one hour to increase $0.1mgl^{-1}$ after peak dilution; this pattern was also seen on the Don and Ryburn. With the exception of the first peak during the 13/03/13 experiment the same can be said for conductivity. It took at least one hour for conductivity to recover $100\mu scm^{-1}$. In all of the release regimes tested peak flow at the reservoir valve was never maintained more than 2 hours and this reflects in the hydrographs produced down river. They have long recession limbs ranging from 4 to > 6 hours, but do not maintain their peaks more than 15 minutes. Should the valves be opened longer it is likely that peak flows and dilution rates closer to peak would be maintained longer. Taking into account that partial recovery from dilution takes longer than an hour a release should be maintained for 1 hour less than the duration of the pollution incident being considered.

$$r_t = p_t - 1$$

Equation 6.4

Where r_t = release time in hours, and p_t = pollution incident duration in hours.

6.6.5 Wave release profile

Aim 4 of this thesis was to examine varied scenarios including release hydrograph profiles. A key management motivation behind this was to determine whether a long duration wave was more effective than a higher magnitude wave of the same total volume at diluting a pollutant. The wave profile sets designed for both the flume tank and CFDM of chapters 4 and 5 addressed this question specifically. Due to the relatively short lengths of both tank experiments the results were inconclusive. In the flume a visual inspection showed higher magnitude waves moving more dye down tank. However a quantification of dye concentrations through time at the 1.4m down tank point showed that the lower magnitude longer duration waves increased longitudinal dispersion more. In the CFDM the same trend was observed when the particle distributions generated by two waves totalling 30m^3 were compared. These results are described as inconclusive because of the distances involved. In both cases the higher magnitude waves entrained a greater quantity of tracer within the wave front. At the point of measurement, 1.4m and 110m respectively insufficient time had passed for the tracer to disperse out from the wave front or be left behind by the wave. Over a longer distance of several hundreds of meters it is possible a higher magnitude wave could result in greater dispersion. What is evident is that immediate effect is for a higher magnitude wave to restrict molecular and turbulent diffusion as particles are propelled down the channel.

Wave profile also affects the velocity of the wave as has been discussed. Higher magnitude waves have a higher velocity, therefore maybe desired to reduce catch up times.

6.6.6. Is dilution feasible?

This question has already been discussed in section 6.5 of this chapter. It is worth noting that the recommendations of this thesis have only been validated for the catchments and conditions studied. Whether dilution can be achieved will depend on the nature of the pollution, the magnitude and velocity of the wave, the scale of the catchment and the water quality of the river between the reservoir and pollution incident. All of these variables can vary to a large degree. The mass balance equation (6.3) provided can be used to derive estimates for any given input, but unless reliable data for all the inputs exists results may vary. For instance, the results from the River Don at Blackburn Meadows indicate that a 9m^{3}s^{-1} wave has a very limited impact on dilution. Without detailed numbers for the reservoir wave water quality as it arrives at Blackburn Meadows the reasons for this remain unknown but it is possible that reservoir releases may have a limited impact at distances as long as 25km in heavily urbanised catchments. All systems have their limitations, a water manager should monitor the results of using such as system and learn from experiences.

6.7 Conclusion and Evaluation

A mitigation system for in river pollution incident is necessary even with the ongoing development of source control and incident reduction strategies, as costly accidents still happen. Currently there are no such mitigation systems, and pollution incidents are largely dealt with through financial penalties. Such penalties whilst an effective deterrent do not alleviate the conditions in a river after a spill. A system that can mitigate river pollution, has a quick response time, minimal adverse side effects, and is economically and practically feasible is needed. Water released from a reservoir to dilute pollution has the potential to meet all these criteria. The object of this thesis has been to establish the feasibility of a reservoir release system for diluting pollution incidents within rivers. It has been shown, that under the conditions tested in field, flume tank and a CFDM model experiments, that a wave of water released down a river can catch up with and dilute polluted waters with in a management practical time period.

In chapter 3 the system was field tested. Waves were released to dilute the outflows of STW and a dye slug. Waves were found to move down river at approximately 1ms^{-1} in the majority of experiments. By contrast measured the dye slug in the control experiment had a mean velocity of 0.185ms^{-1} and 3m^{3}s^{-1} flow in the 1D model had a velocity of 396ms^{-1}. Dilution of both NH_4 and conductivity ranged between 30-59% of control values in response to wave arrival. These two results established the feasibility of using reservoir releases as a mitigation tool at an empirical level. These results were drawn upon to inform the operational recommendations in chapter 6, additionally the scope of a 9 release experiment data set across three catchments and four seasons exceeds anything published in the literature to date. Whilst short comings such as a lack of water quality data immediately upstream of the STW input most be acknowledged the approach taken in chapter 3 can be largely considered to have fulfilled the aims of this thesis.

In chapter 4 waves were released down a 5m flume tank in two experiment sets. In the first set the wave water was dyed, this showed that as the wave was released into the tank , the dyed water was left behind. The wave was transferred down tank as an energy wave. This suggested that as the wave moved down river a progressively lower proportion of the water from the reservoir would remain. Therefore the water quality of the reservoir water will become less important as downstream distance increases. Second the dyed wave water was found to move over the top of the baseflow water. This vertical stratification of flow within the wave front was seen in the second set of experiments, the drop tests. A volume of dye, or pollution substitute such as oil, was dropped into the tank and then a wave was released to dilute it. The results from these experiments indicated that the upper water column can move at supercritical velocities during wave passage and this greatly increases down tank advection. Longitudinal dispersion was quantified at 1.4m down tank and showed, that over a short time frame a wave reduced longitudinal dispersion as dye was moved to rapidly for diffusion to take place. The results from this chapter were to clearly establish the kinematic effect of the wave in an idealised environment. However, due to the scale, lack of roughness elements in the channel and super critical nature of the waves tested it is difficult to translate any of the findings directly to the river channel. Equally, whilst longitudinal dispersion was measured, the methodological design of this experiment failed to account for dilution in a thorough manner. Had a second camera been placed at bottom end of the tank and the refraction wave reduced or eliminated a more comprehensive dispersion result might have been gained. This chapter has provided some insights into mixing in the water column, and was novel in its approach, but given the aims of this thesis it can be considered a failure as none of the results were directly related to the operational recommendations made in chapter 6.

In chapter 5 two CFDM were constructed. The first was a 5m model designed to replicate the flume tank as closely as possible, the second, referred to as the reach model was 121m long and intended to be closer to the scale of a river reach. In both models a volume of water was released into the tank from one end to catch a volume of particles injected at a down tank point. A similar pattern to that seen in the flume tank was produced. Flow within the wave front was vertically stratified, with the higher supercritical velocities near the surface. The anthracite particles released into the tank were caught and dragged over the top of the water column in a comparable manner to the dye in the flume tank, though with a measured increase in longitudinal dispersion. Whilst the patterns were similar the wave velocities and particle distributions were not numerically identical to those seen in the flume. In the 5m model particle progression down tank was far more limited than that seen in the flume. In the reach scale model the wave speed exceeded $6ms^{-1}$. Longitudinal dispersion increased with both magnitude and wave duration, a comparison between waves of equal volume but differing magnitude and duration was inconclusive. Whilst the scale of the CFDM was an improvement on the flume tank the 110m over which longitudinal dispersion was measured was still insufficient for comparing different wave profiles. Additionally the lack of a diffusion model, the supercritical nature of the wave and the simplification of the channel topography and roughness all limited the applicability of the results to a natural river system. As with the flume experiments of the previous chapter some valuable information was obtained such as the positive relationship between wave magnitude or duration and longitudinal dispersion, but the limited contribution to the operational recommendations of chapter 6 mean that the approach must be considered a failure.

As a whole work this thesis has assessed the feasibility of using reservoir releases to dilute pollution and provided operational guidance for water managers as detailed in chapter 6.

Appendix A
List of Publications Existing and Planned.

Existing publications;

Gillespie, B.R. DeSmet, S. Kay, P. Tillotson, M.R. Brown, L.E. 2014 A critical analysis of regulated river ecosystem responses to managed environmental flows from reservoirs. Freshwater Biology.

Planned titles;

'The response of NH_4 and conductivity during nine release experiments across three different catchments' will be produced from chapter 3.

'mixing within the wave column and its effects on longitudinal dispersion within a flume tank' will be produced from chapter 4

'longitudinal dispersion of particles under the influence of a monoclinal wave in a computer fluid dynamics model' will be produced from chapter 5.

A second paper concerned with the secondary water quality data of chapter 3 could also be produced.

References

ACREMAN, M. C. and A. J. D. FERGUSON. 2010. Environmental flows and the European Water Framework Directive. *Freshwater Biology,* **55**(1), pp.32-48.

ADAM, P. 1994. *Australian Rainforests.* Oxford Biogeography Series. New York: Oxford University Press.

ADAMS, L. K., D. Y. LYON and P. J. J. ALVAREZ. 2006. Comparative eco-toxicity of nanoscale TiO2, SiO2, and ZnO water suspensions. *Water Research,* **40**(19), pp.3527-3532.

ADAMUS, C. L. and M. J. BERGMAN. 1995. Estimating nonpoint source pollution loads with a GIS screening model. *Journal of the American Water Resources Association,* **31**(4), pp.647-655.

AGUIRRE-PE, J., M. OLIVERO and A. MONCADA. 2003. Particle Densimetric Froude Number for Estimating Sediment Transport. *Journal of Hydraulic Engineering,* **129**(6), pp.428-437.

AHIABLAME, L. M., I. CHAUBEY, D. R. SMITH and B. A. ENGEL. 2011. Effect of tile effluent on nutrient concentration and retention efficiency in agricultural drainage ditches. *Agricultural Water Management,* **98**(8), pp.1271-1279.

AL-ATRESH, S., A. SHARIFIAN and B. KUEGER. 2012. Using a split chimney for dilution of exhaust pollution: a CFD approach. *In: International Conference on Fluid Dynamics and Thermodynamics Technologies: FDTT,* Singapore. IPCSIT, pp.156-161.

ALAYA, A. B., A. SOUISSI, J. TARHOUNI and K. NCIB. 2003. Optimization of Nebhana reservoir water allocation by stochastic dynamic programming. *Water Resources Management,* **17**(4), pp.259-272.

ALIPOUR, S. S., A. KARBASSI, S. M. MONAVARI, N. MEHRDADI and M. J. MOHAMMADIZADEH. 2012. Removal of metals during estuarine mixing. *Fresenius Environmental Bulletin,* **21**(12 A), pp.3881-3885.

ALMEIDA, M. C., D. BUTLER and E. FRIEDLER. 1999. At-source domestic wastewater quality. *Urban Water,* **1**(1), pp.49-55.

ALOFS, D. J. and R. L. REISBIG. 1972. An experimental evaluation of oil slick movement caused by waves. *Journal of Physical Oceanography,* **2**(4), pp.439-443.

AMERICAN-HERITAGE. 2003. *dilution*. *American Heritage Dictionary.* 4th ed.

ANDERSEN, M. S. 1994. *Governance by green taxes: Making pollution prevention pay.* Issues in Environmental Politics. Manchester: Manchester University Press.

ANDERSON, D., M, P. GLIBERT, M and J. BURKHOLDER, M. 2002. Harmful algal blooms and eutrophication: Nutrient sources, composition, and consequences. *Estuaries,* **25**(4), pp.704-726.

ANI, E.-C., M. G. HUTCHINS, A. KRASLAWSKI and P. S. AGACHI. 2010. Assessment of pollutant transport and river water quality using mathematical models. *Revue Roumaine de Chimie,* **55**(4), pp.285-291.

ARINI, A., A. FEURTET-MAZEL, S. MORIN, R. MAURY-BRACHET, M. COSTE and F. DELMAS. 2012. Remediation of a watershed contaminated by heavy metals: A 2-year field biomonitoring of periphytic biofilms. *Science of The Total Environment,* **425**, pp.242-253.

ASTARAIE-IMANI, M., Z. KAPELAN, G. FU and D. BUTLER. 2012. Assessing the combined effects of urbanisation and climate change on the river water quality in an integrated urban wastewater system in the UK. *Journal of Environmental Management,* **112**, pp.1-9.

ATHERTON, W., J. W. ASH and R. M. ALKHADDAR. 2008. *The modelling of spills resulting from catastrophic failure of above ground storage tanks and the development of mitigation -=. International Oil Spill Conference Proceedings.* American Petroleum Institute.

BACH, P. M., D. T. MCCARTHY and A. DELETIC. 2010. Redefining the stormwater first flush phenomenon. *Water Research,* **44**(8), pp.2487-2498.

BAEUMER, B., D. A. BENSON, M. M. MEERSCHAERT and S. W. WHEATCRAFT. 2001. Subordinated advection-dispersion equation for contaminant transport. *Water Resources Research,* **37**(6), pp.1543-1550.

BAKER, L. A. 1992. Introduction to nonpoint source pollution in the United States and prospects for wetland use. *Ecological Engineering,* **1**(1–2), pp.1-26.

BALMFORTH, D. J. 1990. The pollution aspects of storm-sewage overflows. *Water and Environment Journal,* **4**(3), pp.219-226.

BARILLIER, A., J. GARNIER and M. COSTE. 1993. Experimental reservoir water release: Impact on the water quality on a river 60km downstream (upper Seine River, France). *Water Research,* **27**(4), pp.635-643.

BARRON, M. G. 1990. Bioconcentration. Will water-borne organic chemicals accumulate in aquatic animals? *Environmental Science & Technology,* **24**(11), pp.1612-1618.

BATES, P. D., K. J. MARKS and M. S. HORRITT. 2003. Optimal use of high-resolution topographic data in flood inundation models. *Hydrological Processes,* **17**(3), pp.537-557.

BAXTER, L. L. and P. J. SMITH. 1993. Turbulent dispersion of particles: the STP model. *Energy & Fuels,* **7**(6), pp.852-859.

BBC. 2010. *Major fire at Huddersfield chemical factory* [online]. [Accessed 09/05/2014]. Available from: http://news.bbc.co.uk/1/hi/england/bradford/8699956.stm.

BEER, T. and P. C. YOUNG. 1983. Longitudinal dispersion in natural streams. *Journal of Environmental Engineering,* **109**(5), pp.1049-1067.

BELL, J. B., W. R. MACRAE and G. E. ELLIOTT. 1980. Incidence of R factors in coliform, fecal coliform, and Salmonella populations of the Red River in Canada. *Applied and Environmental Microbiology,* **40**(3), pp.486-491.

BERLAND, G., T. NICKELSEN, J. HEGGENES, F. OKLAND, E. THORSTAD, B. and J. HALLERAKER. 2004. Movements of wild atlantic salmon parr in relation to peaking flows below a hydropower station. *River Research and Applications,* **20**(8), pp.957-966.

BERMAN, T., B. SHERR, E. SHERR, D. WYNNE and J. MCCARTHY. 1984. The characteristics of ammonium and nitrate uptake by phytoplankton in Lake Kinneret. *Limnology and Oceanography,* **29**(2), pp.287-297.

BESIO, G., A. STOCCHINO, S. ANGIOLANI and M. BROCCHINI. 2012. Transversal and longitudinal mixing in compound channels. *Water Resources Research,* **48**(12), pW12517.

BEVEN, K. 1979. On the generalised kinematic wave routing method. *Water Resources Research,* **15**(5), pp.1238-1242.

BILOTTA, G. S. and R. E. BRAZIER. 2008. Understanding the influence of suspended solids on water quality and aquatic biota. *Water Research,* **42**(12), pp.2849-2861.

BLACKBURN, M. A. and M. J. WALDOCK. 1995. Concentrations of alkylphenols in rivers and estuaries in England and Wales. *Water Research,* **29**(7), pp.1623-1629.

BLOIS, G., J. L. BEST, G. H. SAMBROOK SMITH and R. J. HARDY. 2014. Effect of bed permeability and hyporheic flow on turbulent flow over bed forms. *Geophysical Research Letters,* **41**(18), pp.6435-6442.

BLONDEAUX, P. and G. VITTORI. 2014. The flow over bedload sheets and sorted bedforms. *Continental Shelf Research,* **85**, pp.9-20.

BOCKSELL, T. L. and E. LOTH. 2001. Random walk models for particle diffusion in free-shear flows. *American Institute of Aeronautics and Astronautics Journal,* **39**(6), pp.1086-1096.

BOO, S. Y. 2002. Linear and nonlinear irregular waves and forces in a numerical wave tank. *Ocean Engineering,* **29**(5), pp.475-493.

BOOKER, D. J. 2003. Hydraulic modelling of fish habitat in urban rivers during high flows. *Hydrological Processes,* **17**(3), pp.577-599.

BORJA, A. and M. ELLIOTT. 2007. What does 'good ecological potential' mean, within the European Water Framework Directive? *Marine Pollution Bulletin,* **54**(10), pp.1559-1564.

BOSCH, N. S., M. A. EVANS, D. SCAVIA and J. D. ALLAN. 2014. Interacting effects of climate change and agricultural BMPs on nutrient runoff entering Lake Erie. *Journal of Great Lakes Research,* **40**(3), pp.581-589.

BOTTRELL, S. H., D. HATFIELD, R. BARTLETT, M. J. SPENCE, K. D. BARTLE and R. J. G. MORTIMER. 2010. Concentrations, sulfur isotopic compositions and origin of organosulfur compounds in pore waters of a highly polluted raised peatland. *Organic Geochemistry,* **41**(1), pp.55-62.

BOUFADEL, M. C., K. DU, V. KAKU and J. WEAVER. 2007. Lagrangian simulation of oil droplets transport due to regular waves. *Environmental Modelling & Software,* **22**(7), pp.978-986.

BOURG, A. C. M. and C. BERTIN. 1996. Diurnal variations in the water chemistry of a river contaminated by heavy metals: natural biological cycling and anthropic influence. *Water, Air, & Soil Pollution,* **86**(1), pp.101-116.

BOWES, M. J., C. NEAL, H. P. JARVIE, J. T. SMITH and H. N. DAVIES. 2010. Predicting phosphorus concentrations in British rivers resulting from the introduction of improved phosphorus removal from sewage effluent. *Science of The Total Environment,* **408**(19), pp.4239-4250.

BOXALL, J. B. and I. GUYMER. 2003. Analysis and prediction of transverse mixing coefficients in natural channels. *Journal of Hydraulic Engineering,* **129**(2), pp.129-139.

BRADFORD, M. J., P. S. HIGGINS, J. KORMAN and J. SNEEP. 2011. Test of an environmental flow release in a British Columbia river; does more water mean more fish? *Freshwater Biology,* **56**(10), pp.2119-2134.

BROWN, L. E., G. MITCHELL, J. HOLDEN, A. FOLKARD, N. WRIGHT, N. BEHARRY-BORG, G. BERRY, B. BRIERLEY, P. CHAPMAN, S. J. CLARKE, L. COTTON, M. DOBSON, M. FLETCHER, J. FOSTER, A. HANLON, S. HILDON, P. HILEY, J. HOSEASON, K. JOHNSTON, P. KAY, A. MCDONALD, A. PARROTT, A. POWELL, R. J. SLACK, A. SLEIGH, C. SPRAY, K. TAPLEY, R. UNDERHILL and C. WOULDS. 2010. Priority water research questions as determined by UK practitioners and policy makers. *Science of The Total Environment,* **409**(2), pp.256-266.

BROWN, V. M. 1968. The calculation of the acute toxicity of mixtures of poisons to rainbow trout. *Water Research,* **2**(10), pp.723-733.

BULL, L. J. 1997. Relative velocities of discharge and sediment waves for the River Severn, UK. *Hydrological Sciences Journal,* **42**(5), pp.649-660.

BUTLER, D., E. FRIEDLER and K. GATT. 1995. Characterising the quantity and quality of domestic wastewater inflows. *Water Science and Technology,* **31**(7), pp.13-24.

BUTTLE, J. M. 1994. Hydrological response to reforestation in the Ganaraski river basin, Southern Ontario. *Canadian Geographer / Le Géographe canadien,* **38**(3), pp.240-253.

CABEZAS, H., J. C. BARE and S. K. MALLICK. 1999. Pollution prevention with chemical process simulators: the generalized waste reduction (WAR) algorithm—full version. *Computers & Chemical Engineering,* **23**(4–5), pp.623-634.

CAI, Q.-H. and Z.-Y. HU. 2006. Studies on eutrophication problem and control strategy in the Three Gorges Reservoir. *Acta Hydrobiologica Sinica,* **30**(1), p11.

CALAMARI, D., E. ZUCCATO, S. CASTIGLIONI, R. BAGNATI and R. FANELLI. 2003. Strategic survey of therapeutic drugs in the rivers Po and Lambro in Northern Italy. *Environmental Science & Technology,* **37**(7), pp.1241-1248.

CÁNOVAS, C. R., C. G. HUBBARD, M. OLÍAS, J. M. NIETO, S. BLACK and M. L. COLEMAN. 2008. Hydrochemical variations and contaminant load in the Río Tinto (Spain) during flood events. *Journal of Hydrology,* **350**(1-2), pp.25-40.

CAPELLA, J. V., A. BONASTRE, R. ORS and M. PERIS. 2014. A step forward in the in-line river monitoring of nitrate by means of a wireless sensor network. *Sensors and Actuators B: Chemical,* **195**, pp.396-403.

CARPENTER, S. R., N. F. CARACO, D. L. CORRELL, R. W. HOWARTH, A. N. SHARPLEY and V. H. SMITH. 1998. Nonpoint pollution of surface waters with phosphorus and nitrogen. *Ecological Applications,* **8**(3), pp.559-568.

CARRIVICK, J. L., R. JONES and G. KEEVIL. 2011. Experimental insights on geomorphological processes within dam break outburst floods. *Journal of Hydrology,* **408**(1–2), pp.153-163.

CARSON, R. 1962. *Silent Spring.* New York: Mariner Books.

CASAS, M. A., S. N. LANE, R. J. HARDY, G. BENITO and P. J. WHITING. 2010. Reconstruction of subgrid-scale topographic variability and its effect upon the spatial structure of three-dimensional river flow. *Water Resources Research,* **46**(3).

CASSIE, S. and L. SEALE. 2003. Chemical storage tank systems–good practice. *CIRIA Publication C* [online]. **598**, pp.1-243. Available from: http://www.balmoral-group.com/tnk/bls-microsite/pdf/ciria-construction-industry-regs.pdf.

CHA, S. M., M.-J. KANG, Y. PARK, S. W. LEE and J. H. KIM. 2014. Water quality changes according to the midstream weir construction in the Yeongsan River, Korea. *Desalination and Water Treatment,* **53**(11), pp.1-6.

CHADWICK, A., J. MORFETT and M. BORTHWICK. 2004. *Hydraulics in Civil and Environmental Engineering.* Fourth ed. London: Spon Press.

CHAPRA, S. and P. WHITEHEAD. 2009. Modelling impacts of pollution in river systems: a new dispersion model and a case study of mine discharges in the Abrud, Aries and Mures River System in Transylvania, Romania. *Hydrology Research,* **40**(2-3), pp.306-322.

CHEN, Y.-Y., M.-S. LI, H.-C. HSU and C.-O. NG. 2012. Theoretical and experimental study of particle trajectories for nonlinear water waves propagating on a sloping bottom. *Philosophical Transactions of the Royal Society A: Mathematical,* **370**, pp.1543-1571.

CHOW, V., T. 1959. *Open-Channel Hydraulics.* London: McGraw-Hill.

CHUNG, S. W., I. H. KO and Y. K. KIM. 2008. Effect of reservoir flushing on downstream river water quality. *Journal of Environmental Management,* **86**(1), pp.139-147.

CHUNGUO, C. and L. ZIHUI. 1988. Chemical speciation and distribution of arsenic in water, suspended solids and sediment of Xiangjiang River, China. *Science of the Total Environment,* **77**(1), pp.69-82.

CONSTANTIN, A. 2000. Existence of permanent and breaking waves for a shallow water equation : a geometric approach. *Annales de l'institut Fourier,* **50**(2), pp.321-362.

CONSTANTIN, A. and J. ESCHER. 1998. Global existence and blow-up for a shallow water equation. *Annali della Scuola Normale Superiore di Pisa - Classe di Scienze,* **26**(2), pp.303-328.

CONSTANTIN, A. and J. ESCHER. 2007. Particle trajectories in solitary water waves. *Bulletin of the American Mathematical Society,* **44**(3), pp.423-431.

CONTENTO, G. 2000. Numerical wave tank computations of nonlinear motions of two-dimensional arbitrarily shaped free floating bodies. *Ocean Engineering,* **27**(5), pp.531-556.

CORRELL, D. L. 1998. The role of phosphorus in the eutrophication of receiving waters: a review. *Journal of Environmental Quality,* **27**(2), pp.261-266.

of on Council Directive 75/440/EEC Surface Water Directive.

of on Council Directive concerning the quality of bathing water (Bathing Water Directive).

of on Council Directive on the quality of fresh waters needing protection or improvement in order to support fish life (Freshwater Fish Directive).

COX, B. A. 2003. A review of currently available in-stream water-quality models and their applicability for simulating dissolved oxygen in lowland rivers. *Science of The Total Environment,* **314–316**, pp.335-377.

CRISTEA, V. M., E. D. BAGIU and P. S. AGACHI. 2010. Simulation and Control of Pollutant Propagation in Somes River Using COMSOL Multiphysics. *Computer Aided Chemical Engineering,* **28**, pp.985-990.

CROWTHER, J., D. KAY and M. D. WYER. 2002. Faecal-indicator concentrations in waters draining lowland pastoral catchments in the UK: relationships with land use and farming practices. *Water Research,* **36**(7), pp.1725-1734.

CRUMPTON, W. G. and C. M. HERSH. 1987. Diurnal Patterns of Ammonium Loading and Loss in a Stream Receiving Secondary Treatment Effluent. *Journal (Water Pollution Control Federation),* **59**(1), pp.60-62.

CZERNUSZENKO, W., P.-M. ROWIŃSKI and A. SUKHODOLOV. 1998. Experimental and numerical validation of the dead-zone model for longitudinal dispersion in rivers. *Journal of Hydraulic Research,* **36**(2), pp.269-280.

D'AGOSTINO, V., M. CESCA and L. MARCHI. 2010. Field and laboratory investigations of runout distances of debris flows in the Dolomites (Eastern Italian Alps). *Geomorphology,* **115**(3), pp.294-304.

D'ARCY, B. and A. FROST. 2001. The role of best management practices in alleviating water quality problems associated with diffuse pollution. *Science of The Total Environment,* **265**(1–3), pp.359-367.

DAMBERGER, C. 1973. *Oil skimming apparatus*. Google Patents.

DAVIS, J. R. and K. KOOP. 2006. Eutrophication in Australian rivers, reservoirs and estuaries–a southern hemisphere perspective on the science and its implications. *Hydrobiologia,* **559**(1), pp.23-76.

DAVIS, R. E. and A. ACRIVOS. 1967. Solitary internal waves in deep water. *Journal of Fluid Mechanics,* **29**(03), pp.593-607.

DAWSON, E. J. and M. G. MACKLIN. 1998. Speciation of heavy metals in floodplain and flood sediments: a reconnaissance survey of the Aire Valley, West Yorkshire, Great Britain. *Environmental Geochemistry and Health,* **20**(2), pp.67-76.

DAY, J., M. HOCKINGS and G. JONES. 2002. Measuring effectiveness in marine protected areas—principles and practice. *In: World Congress on Aquatic Protected Areas, Australia,* Cairns Australia. Great Barrier Reef Marine Authority, pp.2-3.

DAY, T. J. 1976. On the precision of salt dilution gauging. *Journal of Hydrology,* **31**(3–4), pp.293-306.

DE SAINT-VENANT, A. J. C. B. 1871. Theorie du mouvement non permanent des eaux, avec application aux crues des rivire res et a l'introduction des marees dans leurs lits. *Comptes Rendus des séances de l'Académie des Sciences,* **73**, pp.237-240.

DE SMEDT, F., W. BREVIS and P. DEBELS. 2005. Analytical solution for solute transport resulting from instantaneous injection in streams with transient storage. *Journal of Hydrology,* **315**(1–4), pp.25-39.

DEFRA. 2012. *National Policy Statement fo Waste Water: A Frame work document for planning decisions on nationally significant waste water infrastructure.* London: The Stationary Office.

DEHBI, A. 2008. A CFD model for particle dispersion in turbulent boundary layer flows. *Nuclear Engineering and Design,* **238**(3), pp.707-715.

DELLI PRISCOLI, J. 2000. Water and civilization: using history to reframe water policy debates and to build a new ecological realism. *Water Policy,* **1**(6), pp.623-636.

DELVIGNE, G. A. L. and C. E. SWEENEY. 1988. Natural dispersion of oil. *Oil and Chemical Pollution,* **4**(4), pp.281-310.

DENG, Z., V. SINGH and L. BENGTSSON. 2001. Longitudinal Dispersion Coefficient in Straight Rivers. *Journal of Hydraulic Engineering,* **127**(11), pp.919-927.

DI BALDASSARRE, G. and S. UHLENBROOK. 2012. Is the current flood of data enough? A treatise on research needs for the improvement of flood modelling. *Hydrological Processes,* **26**(1), pp.153-158.

DONG, C. and C. HUANG. 2004. Generation and Propagation of Water Waves in a Two-Dimensional Numerical Viscous Wave Flume. *Journal of Waterway, Port, Coastal, and Ocean Engineering,* **130**(3), pp.143-153.

DOW, K., P. STEFFLER and D. ZHU. 2009. Case Study: Intermediate field mixing for a bank discharge in a natural river. *Journal of Hydraulic Engineering,* **135**(1), pp.1-12.

DROLC, A. and J. Z. KONČAN. 1996. Water quality modelling of the river Sava, Slovenia. *Water Research,* **30**(11), pp.2587-2592.

DUDA, A. M. 1993. Addressing nonpoint sources of water pollution must become and international priority. *Water Science and Technology,* **28**(3-5), pp.1-11.

DUONG, T. T., S. MORIN, M. COSTE, O. HERLORY, A. FEURTET-MAZEL and A. BOUDOU. 2010. Experimental toxicity and bioaccumulation of cadmium in freshwater periphytic diatoms in relation with biofilm maturity. *Science of The Total Environment,* **408**(3), pp.552-562.

EA. 2007. *The Comon Incident Classification Scheme*. Bristol: Environment Agency.

EA. 2009a. *Humber district river basin management plan*. Bristol: Environment Agency.

EA. 2009b. *Water for life and livelihoods; River Basin Mangement Plan Humber River Basin District*. Bristol: Environment Agency.

EA. 2011. *Category 1 and 2 pollution incdients by source to air, land and water. In:* ENVIRONMENT-AGENCY (Ed.): Environment Statistics Service, DEFRA.

GREAT BRITAIN. 2000/60/EC. *Directive of the European Parliament and of the council of 23 October 2000 establishing a framework for the community action in the field of water policy* Official Journal.

ECKERT, J. M. and E. R. SHOLKOVITZ. 1976. The flocculation of iron, aluminium and humates from river water by electrolytes. *Geochimica et Cosmochimica Acta,* **40**(7), pp.847-848.

EDWARDS, S. J. and B. V. KJELLERUP. 2013. Applications of biofilms in bioremediation and biotransformation of persistent organic pollutants, pharmaceuticals/personal care products, and heavy metals. *Applied Microbiology and Biotechnology,* **97**(23), pp.9909-9921.

EGGLETON, J. and K. V. THOMAS. 2004. A review of factors affecting the release and bioavailability of contaminants during sediment disturbance events. *Environment International,* **30**(7), pp.973-980.

EHRENDORFER, M. and A. H. MURPHY. 1988. Comparative evaluation of weather forecasting systems: Sufficiency, quality, and accuracy. *Monthly Weather Review,* **116**(9), pp.1757-1770.

ELHADI, N., A. HARRINGTON, I. HILL, Y. L. LAU and B. G. KRISHNAPPAN. 1984. River mixing—A state-of-the-art report. *Canadian Journal of Civil Engineering,* **11**(3), pp.585-609.

ELÍASSON, J., S. P. KJARAN, S. L. HOLM, M. T. GUDMUNDSSON and G. LARSEN. 2007. Large hazardous floods as translatory waves. *Environmental Modelling & Software,* **22**(10), pp.1392-1399.

ELLIS, B. 1991. Urban runoff quality in the UK: problems, prospects and procedures. *Applied Geography,* **11**(3), pp.187-200.

ELLIS, J. B. and T. HVITVED-JACOBSEN. 1996. Urban drainage impacts on receiving waters. *Journal of Hydraulic Research,* **34**(6), pp.771-783.

ELLIS, J. B. and G. MITCHELL. 2006. Urban diffuse pollution: key data information approaches for the Water Framework Directive. *Water and Environment Journal,* **20**(1), pp.19-26.

ENGELUND, F. 1974. Flow and bed topography in channel bends. *Journal of the Hydraulics Division,* **100**(11), pp.1631-1648.

of on Environmental Quality Standards Directive.

ESTÈBE, A., J.-M. MOUCHEL and D. R. THÉVENOT. 1998. Urban runoff impacts on particulate metal concentrations in River Seine. *Water, Air, and Soil Pollution,* **108**(1-2), pp.83-105.

ETIM, E. U. and P. C. ONIANWA. 2013. Impact of effluent of an industrial estate on Oruku River in Southwestern Nigeria. *World Applied Sciences Journal,* **21**(7), pp.1075-1083.

GREAT BRITAIN. C. O. EUROPE. 2006. *Directive 2006/7/EC of the European Parliament and of the Council of 15 February 2006 concerning the management of bathing water quality and repealing Directive*. Official Journal of the EUropean Union: EU.

FAGHERAZZI, S., P. RASETARINERA, M. HUSSAINI and D. FURBISH. 2004. Numerical solution of the dam-break problem with a discontinuous Galerkin method. *Journal of Hydraulic Engineering,* **130**(6), pp.532-539.

FALCONER, I. R. 1989. Effects on human health of some toxic cyanobacteria (blue-green algae) in reservoirs, lakes, and rivers. *Toxicity Assessment,* **4**(2), pp.175-184.

FALCONER, I. R. 1999. An Overview of problems caused by toxic blue–green algae (cyanobacteria) in drinking and recreational water. *Environmental Toxicology,* **14**(1), pp.5-12.

FAN, C., C.-H. KO and W.-S. WANG. 2009. An innovative modeling approach using Qual2K and HEC-RAS integration to assess the impact of tidal effect on River Water quality simulation. *Journal of Environmental Management,* **90**(5), pp.1824-1832.

FARHADIAN, M., O. HADDAD, S. SEIFOLLAHI-AGHMIUNI and H. LOÁICIGA. 2014. Assimilative capacity and flow dilution for water quality protection in rivers. *Journal of Hazardous, Toxic, and Radioactive Waste,* **19**(2), p04014027.

FARMER, J. G., L. J. EADES, A. B. MACKENZIE, A. KIRIKA and T. E. BAILEY-WATTS. 1996. Stable lead isotope record of lead pollution in Loch Lomond sediments since 1630 A.D. *Environmental Science & Technology,* **30**(10), pp.3080-3083.

FENTON, J. D. 2011. *Accuracy of Muskingum-Cunge flood routing*. Alternative Hydraulics Paper, Vienna: Institute of Hydraulic and Water Resources Engineering.

FICK, A. 1855. Ueber Diffusion. *Annalen der Physik,* **170**(1), pp.59-86.

FIELD, J. A. and R. L. REED. 1996. Nonylphenol polyethoxy carboxylate metabolites of nonionic surfactants in U.S. paper mill effluents, municipal sewage treatment plant effluents, and river waters. *Environmental Science & Technology,* **30**(12), pp.3544-3550.

FISCHER, H. B. 1969. The effect of bends on dispersion in streams. *Water Resources Research,* **5**(2), pp.496-506.

FISCHER, H. B. 1973. Longitudinal dispersion and turbulent mixing in open-channel flow. *Annual Review of Fluid Mechanics,* **5**(1), pp.59-78.

FOULGER, T. R. and G. E. PETTS. 1984. Water quality implications of artificial flow fluctuations in regulated rivers. *Science of The Total Environment,* **37**(2-3), pp.177-185.

FRIEDLER, E., D. BUTLER and D. M. BROWN. 1996. Domestic WC usage patterns. *Building and Environment,* **31**(4), pp.385-392.

FU, G., S. KHU and D. BUTLER. 2010. Optimal distribution and control of storage tank to mitigate the impact of new developments on receiving water quality. *Journal of Environmental Engineering,* **136**(3), pp.335-342.

GALVIN, C. 1972. Wave breaking in shallow water. *In:* R. MEYER, E., ed. *Waves on Beaches.* New York: Academic Press.

GARCIA, A., K. JORDE, E. HABIT, D. CAMMANO and O. PARRA. 2011. Downstream environmental effects of dam operations; changes in habitat quality for native fish species. *River Research and Applications,* **27**, pp.312-327.

GEORGE-ARES, A., R. R. LESSARD, G. P. CANEVARI, K. W. BECKER and R. J. FIOCCO. 2001. Modification of the dispersant Corexit®9500 for use in freshwater. *International Oil Spill Conference Proceedings,* **2001**(2), pp.1209-1211.

GERBEAU, J.-F. and B. PERTHAME. 2001. Derivation of viscous Saint-Venant system for laminar shallow water; numerical validation. *Discrete and Contiuous Dynamical Systems - Series B,* **1**(1), pp.89-102.

GIBBONS, D. 1986. *The Economic Value of Water.* New York: Resources for the Future.

GIDO, K., B., R. LARSON, D. and L. AHLM, A. 2000. Stream-channel position of adult rainbow trout downstream of Navajo Reservoir, New Mexico, following changes in reservoir release. *North American Journal of Fisheries Management,* **20**, pp.250-258.

GILVEAR, D. J. 1989. Experimental analysis of reservoir release wave routing in upland boulder bed rivers. *Hydrological Processes,* **3**(3), pp.261-276.

GILVEAR, D. J. and G. E. PETTS. 1985. Turbidity and suspended solids variations downstream of a regulating reservoir. *Earth Surface Processes and Landforms,* **10**, pp.363-373.

GLOVER, B. J. and P. JOHNSON. 1974. Variations in the natural chemical concentration of river water during flood flows, and the lag effect. *Journal of Hydrology,* **22**(3-4), pp.303-316.

GOBAS, F. and H. A. MORRISON. 2000. *Bioconcentration and biomagnification in the aquatic environment*. Lewis Publishers: Boca Raton, FL.

GÖBEL, P., C. DIERKES and W. G. COLDEWEY. 2007. Storm water runoff concentration matrix for urban areas. *Journal of Contaminant Hydrology,* **91**(1–2), pp.26-42.

GOOSEFF, M. N., K. E. BENCALA and S. M. WONDZELL. 2008. Solute transport along stream and river networks. *River confluences, tributaries and the fluvial network*, pp.395-418.

GOSMAN, A. D. and E. LOANNIDES. 1983. Aspects of Computer Simulation of Liquid-Fueled Combustors. *Journal of Energy,* **7**(6), pp.482-490.

GOVERS, G. 1985. Selectivity and transport capacity of thin flows in relation to rill erosion. *CATENA,* **12**(1), pp.35-49.

GOVERS, G. 1992. Relationship between discharge, velocity and flow area for rills eroding loose non layered materials. *Earth Surface Processes and Landforms,* **17**(5), pp.515-528.

GRAF, J. B. 1995. Measured and predicted velocity and longitudinal dispersion at steady and unsteady flow, Colorado River, Glen Cayon Dam to Lake Mead. *JAWRA Journal of the American Water Resources Association,* **31**(2), pp.265-281.

GRAY, J. S. 2002. Biomagnification in marine systems: the perspective of an ecologist. *Marine Pollution Bulletin,* **45**(1), pp.46-52.

GRUTSCH, J., F. and R. MALLATT, C. 1971. *Process for removing contaminants from waste-water*. Patent number:

GUASCH, H., J. ARMENGOL, E. MARTí and S. SABATER. 1998. Diurnal variation in dissolved oxygen and carbon dioxide in two low-order streams. *Water Research,* **32**(4), pp.1067-1074.

GUEVARA-RIBA, A., A. SAHUQUILLO, R. RUBIO and G. RAURET. 2004. Assessment of metal mobility in dredged harbour sediments from Barcelona, Spain. *Science of The Total Environment,* **321**(1–3), pp.241-255.

GUEZENNEC, A.-G., C. MICHEL, C. JOULIAN, M. DICTOR, C. and F. BATTAGLIA-BRUNET. 2012. Treatment of Arsenic Contaminated Mining Water using Biofilms. *In: Interfaces Against Pollution*. Publications Scientifiques Du BRGM.

GUO, Q., C.-Y. FAN, R. RAGHAVEN and R. FIELD. 2004. Gate and vacuum flushing of sewer sediment: Laboratory testing. *Journal of hydraulic engineering,* **130**(5), pp.463-466.

GUO, W., Z. LI, S. CHENG, W. LIANG, F. HE and Z. WU. 2014. Performance of a pilot-scale constructed wetland for stormwater runoff and domestic sewage

treatment on the bank of a polluted urban river. *Water Science & Technology,* **69**(7), pp.1410-1418.
GUYMER, I. 1998. Longitudinal Dispersion in Sinuous Channel with Changes in Shape. *Journal of Hydraulic Engineering,* **124**(1), pp.33-40.
HAINES, B., C. JORDAN, H. CLARK and K. E. CLARK. 1983. Acid rain in an Amazon rainforest. *Tellus B,* **35B**(1), pp.77-80.
HAMRICK, J. M. and T. S. WU. 1997. Computational design and optimization of the EFDC/HEM3D surface water hydrodynamic and eutrophication models. *In:* G. DELICH and M. F. WHEELER, eds. *Next generation environmental models and computational methods.* Philadelphia: Society of Industrial and Applied Mathematics, pp.143-156.
HARDY, R. J., J. L. BEST, S. N. LANE and P. E. CARBONNEAU. 2009. Coherent flow structures in a depth-limited flow over a gravel surface: The role of near-bed turbulence and influence of Reynolds number. *Journal of Geophysical Research: Earth Surface,* **114**(1).
HARDY, R. J., J. L. BEST, S. N. LANE and P. E. CARBONNEAU. 2010. Coherent flow structures in a depth-limited flow over a gravel surface: The influence of surface roughness. *Journal of Geophysical Research: Earth Surface,* **115**(3).
HARDY, R. J., J. L. BEST, D. R. PARSONS and G. M. KEEVIL. 2011a. On determining the geometric and kinematic characteristics of coherent flow structures over a gravel bed: A new approach using combined PLIF-PIV. *Earth Surface Processes and Landforms,* **36**(2), pp.279-284.
HARDY, R. J., S. N. LANE and D. YU. 2011b. Flow structures at an idealized bifurcation: A numerical experiment. *Earth Surface Processes and Landforms,* **36**(15), pp.2083-2096.
HARREMOËS, P. 1982. Immediate and delayed oxygen depletion in rivers. *Water Research,* **16**(7), pp.1093-1098.
HARWOOD, J. J. 2014. Molecular markers for identifying municipal, domestic and agricultural sources of organic matter in natural waters. *Chemosphere,* **95**(0), pp.3-8.
HAWTHORNE, W. R. 1951. Secondary circulation in fluid flow. *Proceedings of the Royal Society of London. Series A. Mathematical and Physical Sciences,* **206**(1086), pp.374-387.
HEALY, R. and T. RUSSELL. 1993. A finite-volume Eulerian-Lagrangian Localized Adjoint Method for solution of the advection-dispersion equation. *Water Resources Research,* **29**(7), pp.2399-2413.
HEIDEL, S. G. 1966a. The progressive lag of sediment concentration in release waves. *Transactions of American Geophysical Union,* **37**, pp.56-66.
HEIDEL, S. G. 1966b. The progressive lag of sediment concentration with flood waves. *Transactions of American Geophysical Union,* **37**, pp.56-66.
HENSON, S. S., D. S. AHEARN, R. A. DAHLGREN, E. V. NIEUWENHUYSE, K. W. TATE and W. E. FLEENOR. 2007. Water quality response to a pulsed-flow event on the Mokelumne river, California. *River Research and Applications,* **23**(2), pp.185-200.
HERODOTUS. 1987. *The History The History.* University of Chicago Press.
HERSCHY, R. W. 1998. *Hydrometry : principles and practices.* Second ed. New York: Wiley.
HESTHAGEN, T. 1987. Fish Kills of Atlantic Salmon (Salmo Salar) and Brown Trout (Salmo Trutta) in an Acidified River of SW Norway. *In:* H. MARTIN, ed. *Acidic Precipitation.* Springer Netherlands, pp.619-628.
HILLEL, D. 1991. *Out of the Earth.* New York: MacMillan.
HILLMAN, M. and G. BRIERLEY. 2005. A critical review of catchment-scale stream rehabilitation programmes. *Progress in Physical Geography,* **29**(1), pp.50-76.
HIROTA, R. 1973. Exact N-soliton solutions of the wave equation of long waves in shallow-water and in nonlinear lattices. *Journal of Mathematical Physics,* **14**(7), pp.810-814.

HIRT, C. W. and B. D. NICHOLS. 1981. Volume of fluid (VOF) method for the dynamics of free boundaries. *Journal of Computational Physics,* **39**(1), pp.201-225.
HOLDEN, J., P. CHAPMAN, S. PALMER, P. KAY and R. GRAYSON. 2012. The impacts of prescribed moorland burning on water colour and dissolved organic carbon: A critical synthesis. *Journal of environmental management,* **101**, pp.92-103.
HOLMAN, J. P. 2002. *Heat Transfer.* New York: McGraw-Hill.
HORN, H. 2003. The relative importance of climate and nutrients in controlling phytoplankton growth in Saidenbach Reservoir. *Hydrobiologia,* **504**(1-3), pp.159-166.
HOUSE, W. A. and M. S. WARWICK. 1998. Hysteresis of the solute concentration/discharge relationship in rivers during storms. *Water Research,* **32**(8), pp.2279-2290.
HOWDEN, N. J. K., M. J. BOWES, A. D. J. CLARK, N. HUMPHRIES and C. NEAL. 2009. Water quality, nutrients and the European union's Water Framework Directive in a lowland agricultural region: Suffolk, south-east England. *Science of The Total Environment,* **407**(8), pp.2966-2979.
HSU, H.-C., Y.-Y. CHEN and H.-H. HWUNG. 2012. Experimental study of the particle paths in solitary water waves. *Philosophical Transactions of the Royal Society A: Mathematical, Physical and Engineering Sciences,* **370**(1964), pp.1629-1637.
HUAMAIN, C., Z. CHUNRONG, T. CONG and Z. YONGGUAN. 1999. Heavy Metal Pollution in Soils in China: Status and Countermeasures. *Ambio,* **28**(2), pp.130-134.
HUANG, F., X. WANG, L. LOU, Z. ZHOU and J. WU. 2010. Spatial variation and source apportionment of water pollution in Qiantang River (China) using statistical techniques. *Water Research,* **44**(5), pp.1562-1572.
HUANG, T., Y. MA, H. CONG and P. TAN. 2013. Application of the technology of water lifting and aeration on improving water quality in a Deep Canyon Reservoir – a case study from northern China. *Desalination and Water Treatment,* **52**(7-9), pp.1636-1646.
HUNT, B. 2006. Asymptotic Solutions for One-Dimensional Dispersion in Rivers. *Journal of Hydraulic Engineering,* **132**(1), pp.87-93.
IVEY, G. N. and R. I. NOKES. 1989. Vertical mixing due to the breaking of critical internal waves on sloping boundaries. *Journal of Fluid Mechanics,* **204**, pp.479-500.
IWAGAKI, Y. and T. SAKAI. 1970. Horizontal water particle velocity of finite amplitude waves. *Coastal Engineering 1970 Proceedings.* Washington D.C.: ASCE.
J., B. E. and W. YU. 1995. Bacteriology of urban runoff: The combined sewer as a bacterial reactor and generator. *Water Science and Technology,* **31**(7), pp.303-310.
JAKOB, C., C. T. ROBINSON and U. UEHLINGER. 2003. Longitudinal effects of experimental floods on stream benthos downstream from a large dam. *Aquatic Sciences,* **65**, pp.223-231.
JARRETT, R. 1984. Hydraulics of High-Gradient Streams. *Journal of Hydraulic Engineering,* **110**(11), pp.1519-1539.
JI, Z., J. HAMRICK and J. PAGENKOPF. 2002. Sediment and Metals Modeling in Shallow River. *Journal of Environmental Engineering,* **128**(2), pp.105-119.
JING, S.-R., Y.-F. LIN, D.-Y. LEE and T.-W. WANG. 2001. Nutrient removal from polluted river water by using constructed wetlands. *Bioresource Technology,* **76**(2), pp.131-135.
JORDAN, P., A. ARNSCHEIDT, H. MCGROGAN and S. MCCORMICK. 2007. Characterising phosphorus transfers in rural catchments using a continuous bank-side analyser. *Hydrology and Earth System Sciences Discussions,* **11**(1), pp.372-381.

KADOMTSEV, B. B. and V. I. PETVIASHVILI. 1970. On the stability of solitary waves in weakly dispersing media. *Fluid Mechanics,* **15**(6), pp.539-541.

KANNEL, P., S. KANEL, S. LEE, Y.-S. LEE and T. GAN. 2011. A Review of Public Domain Water Quality Models for Simulating Dissolved Oxygen in Rivers and Streams. *Environmental Modeling & Assessment,* **16**(2), pp.183-204.

KARR, J. R. 1981. Assessment of Biotic Integrity Using Fish Communities. *Fisheries,* **6**(6), pp.21-27.

KAY, D. and J. STONER. 1988. The effects of catchment land use on stream water quality in an acid-susceptible region of West Wales: the implications for compliance with EC drinking water and fishery directives. *Applied Geography,* **8**(3), pp.191-205.

KAY, P., P. A. BLACKWELL and A. B. A. BOXALL. 2005a. Column studies to investigate the fate of veterinary antibiotics in clay soils following slurry application to agricultural land. *Chemosphere,* **60**(4), pp.497-507.

KAY, P., P. A. BLACKWELL and A. B. A. BOXALL. 2005b. Transport of veterinary antibiotics in overland flow following the application of slurry to arable land. *Chemosphere,* **59**(7), pp.951-959.

KAY, P., R. GRAYSON, M. PHILLIPS, K. STANLEY, A. DODSWORTH, A. HANSON, A. WALKER, M. FOULGER, I. MCDONNELL and S. TAYLOR. 2012. The effectiveness of agricultural stewardship for improving water quality at the catchment scale: Experiences from an NVZ and ECSFDI watershed. *Journal of Hydrology,* **422–423**(0), pp.10-16.

KELLY, M., S. JUGGINS, R. GUTHRIE, S. PRITCHARD, J. JAMIESON, B. RIPPEY, H. HIRST and M. YALLOP. 2008. Assessment of ecological status in U.K. rivers using diatoms. *Freshwater Biology,* **53**(2), pp.403-422.

KEMERER, J. A., N. HENDRICKSON and R. MULLINAUX. 1985. A case history: oil spill onto the Prado flood control basin, a freshwater wetlands cleanup. *International Oil Spill Conference Proceedings,* **1985**(1), pp.325-329.

KENDALL, C. and J. MCDONNELL, J.,. 1998. *Isotope Tracers in Catchment Hydrology.* Amsterdam: Elsevier Science.

KILPATRICK, F. A. and J. F. WILSON. 1982. Measurement of time of travel in streams by dye tracing. *Techniques of Water-Resources Investigations of the United States Geological Survey.* Denver: USGS.

KIM, C. H., A. H. CLÉMENT and K. TANIZAWA. 1999. Recent Research And Development of Numerical Wave Tanks - A Review.

KIM, G., J. YUR and J. KIM. 2007. Diffuse pollution loading from urban stormwater runoff in Daejeon city, Korea. *Journal of Environmental Management,* **85**(1), pp.9-16.

KINGTON, J. 2010. *Climate and weather.* HarperCollins UK.

KLEBERCZ, O., W. M. MAYES, Á. D. ANTON, V. FEIGL, A. P. JARVIS and K. GRUIZ. 2012. Ecotoxicity of fluvial sediments downstream of the Ajka red mud spill, Hungary. *Journal of Environmental Monitoring,* **14**(8), pp.2063-2071.

KLEKOWSKI, E. and D. E. LEVIN. 1979. Mutagens in a river heavily polluted with paper recycling wastes: Results of field and laboratory mutagen assays. *Environmental Mutagenesis,* **1**(3), pp.209-219.

KLUGMAN, J., F. RODRIGUEZ, S. BEEJADHUR, S. BHATTACHARJESS, M. CHATTERJEE, H.-J. CHOI, A. FUCHS, M. GEBRETSADIK, Z. GIDWITZ, M. HEGER, V. KEHAYOVA, J. PINEDA, E. SAMMAN and S. TWIGG. 2011. *Human Development Report 2011, Sustainability and Equity: A Better Future for All*. New York: United Nations.

KNELLER, B., S. BENNETT and W. MCCAFFREY. 1997. Velocity and turbulence structure of density currents and internal solitary waves: potential sediment transport and the formation of wave ripples in deep water. *Sedimentary Geology,* **112**(3), pp.235-250.

KOO, W. and M.-H. KIM. 2004. Freely floating-body simulation by a 2D fully nonlinear numerical wave tank. *Ocean Engineering,* **31**(16), pp.2011-2046.

KRATZ, W. A. and J. MYERS. 1955. Nutrition and Growth of Several Blue-Green Algae. *American Journal of Botany,* **42**(3), pp.282-287.
KREIN, A. and R. DE SUTTER. 2001. Use of artificial flood events to demonstrate the invalidity of simple mixing models / Utilisation de crues artificielles pour prouver l'invalidité des modèles de mélange simple. *Hydrological Sciences Journal,* **46**(4), pp.611-622.
KRIIPSALU, M., M. MARQUES and A. MAASTIK. 2008. Characterization of oily sludge from a wastewater treatment plant flocculation-flotation unit in a petroleum refinery and its treatment implications. *Journal of Material Cycles and Waste Management,* **10**(1), pp.79-86.
KUJAWINSKI, E. B., M. C. KIDO SOULE, D. L. VALENTINE, A. K. BOYSEN, K. LONGNECKER and M. C. REDMOND. 2011. Fate of Dispersants Associated with the Deepwater Horizon Oil Spill. *Environmental Science & Technology,* **45**(4), pp.1298-1306.
KURTENBACH, A., S. MÖLLER, A. KREIN and W. SYMADER. 2006. On the relationship between hydrographs and chemographs. *Hydrological Processes,* **20**(14), pp.2921-2934.
LA, V. T. and S. J. COOKE. 2011. Advancing the science and practice of fish kill investigations. *Reviews in Fisheries Science,* **19**(1), pp.21-33.
LANE, S., K. BRADBROOK, K. RICHARDS, P. BIRON and A. ROY. 2000. Secondary circulation cells in river channel confluences: measurement artefacts or coherent flow structures? *Hydrological Processes,* **14**(11-12), pp.2047-2071.
LANE, S. N., K. F. BRADBROOK, K. S. RICHARDS, P. A. BIRON and A. G. ROY. 1999. The application of computational fluid dynamics to natural river channels: three-dimensional versus two-dimensional approaches. *Geomorphology,* **29**(1–2), pp.1-20.
LANE, S. N. and R. I. FERGUSON. 2005. Modelling reach-scale fluvial flows. *In:* P. D. BATES, S. N. LANE and R. I. FERGUSON, eds. *Computational Fluid Dynamics; Applications in Environmental Hydraulics.* New York: Wliey.
LAU, J., D. BUTLER and M. SCHÜTZE. 2002. Is combined sewer overflow spill frequency/volume a good indicator of receiving water quality impact? *Urban Water,* **4**(2), pp.181-189.
LAURSEN, A. E. and S. P. SEITZINGER. 2004. Diurnal patterns of denitrification, oxygen consumption and nitrous oxide production in rivers measured at the whole-reach scale. *Freshwater Biology,* **49**(11), pp.1448-1458.
LAXEN, D. P. H. and R. M. HARRISON. 1977. The highway as a source of water pollution: An appraisal with the heavy metal lead. *Water Research,* **11**(1), pp.1-11.
LE MAHAUTE, B., D. DIVOKY and A. LIN. 1968. Shallow water waves a comparison of theories and experiments. *Coastal Engineering Proceedings,* **11**.
LEEKS, G. J. L. and M. D. NEWSON. 1989. Responses of the sediment system of a regulated river to a scour valve release:Llyn Clywedog, Mid-Wales, UK. *Regulated Rivers: Research & Management,* **3**, pp.93-106.
LESSARD, R. R. and G. DEMARCO. 2000. The Significance of Oil Spill Dispersants. *Spill Science & Technology Bulletin,* **6**(1), pp.59-68.
LEVEILLE, T. P., D. SHANE and J. MORRIS. 1995. Northridge earthquake pipeline rupture into the Santa Clara River. *International Oil Spill Conference Proceedings,* **1995**(1), pp.489-494.
LEVITAN, L., I. MERWIN and J. KOVACH. 1995. Assessing the relative environmental impacts of agricultural pesticides: the quest for a holistic method. *Agriculture, Ecosystems & Environment,* **55**(3), pp.153-168.
LI, L.-Q., C.-Q. YIN, Q.-C. HE and L.-L. KONG. 2007. First flush of storm runoff pollution from an urban catchment in China. *Journal of Environmental Sciences,* **19**(3), pp.295-299.
LI, R., C. KELLY, R. KEEGAN, L. XIAO, L. MORRISON and X. ZHAN. 2013. Phosphorus removal from wastewater using natural pyrrhotite. *Colloids and Surfaces A: Physicochemical and Engineering Aspects,* **427**(0), pp.13-18.

LIANG, X., S. ZHU, R. YE, R. GUO, C. ZHU, C. FU, G. TIAN and Y. CHEN. 2014. Biological thresholds of nitrogen and phosphorus in a typical urban river system of the Yangtz delta, China. *Environmental Pollution,* **192**, pp.251-258.

LIGHTHILL, M. J. and G. B. WHITHAM. 1955. On Kinematic Waves. I. Flood Movement in Long Rivers. *Proceedings of the Royal Society of London. Series A. Mathematical and Physical Sciences,* **229**(1178), pp.281-316.

LITCHFORD, R. J. and S.-M. JENG. 1991. Efficient Statistical Transport Model for Turbulent Particle Dispersion in Sprays. *AIAA Journal,* **29**(9), pp.1443-1451.

LOGANATHAN, P., S. VIGNESWARAN, J. KANDASAMY and N. S. BOLAN. 2013. Removal and Recovery of Phosphate From Water Using Sorption. *Critical Reviews in Environmental Science and Technology,* **44**(8), pp.847-907.

LONGUET-HIGGINS, M. S. 1953. Mass Transport in Water Waves. *Philosophical Transactions of the Royal Society of London. wseris A, Mathematical and Physical Sciences,* **245**, pp.535-581.

LONGUET-HIGGINS, M. S. 1960. Mass transport in the boundary layer at a free oscillating surface. *Journal of Fluid Mechanics,* **8**(02), pp.293-306.

LOPES, L., F,G., J. DO CARMO, S,A., R. CORTES, M,V. and D. OLIVEIRA. 2004. Hydrodynamics adn water quality modelling in a regulated river segment: application on the instream flow definition. *Ecological Modelling,* **173**, pp.197-218.

LOSKA, K., J. CEBULA, J. PELCZAR, D. WIECHUŁA and J. KWAPULIŃSKI. 1997. Use of enrichment, and contamination factors together with geoaccumulation indexes to evaluate the content of Cd, Cu, and Ni in the Rybnik water reservoir in Poland. *Water, Air, and Soil Pollution,* **93**(1-4), pp.347-365.

MACINNES, J. and F. BRACCO. 1992. Stochastic particle dispersion modeling and the tracer-particle limit. *Physics of Fluids A: Fluid Dynamics (1989-1993),* **4**(12), pp.2809-2824.

MADDOCK, I. P., A. HARBY, P. H. KEMP and P. WOOD. 2013. Rapid flow fluctuations and impacts on fish in the aquatic ecosystem. *Ecohydraulics: An integrated Approach.* London: John Wiley & Sons.

MADORE, M. S., J. B. ROSE, C. P. GERBA, M. J. ARROWOOD and C. R. STERLING. 1987. Occurrence of Cryptosporidium Oocysts in Sewage Effluents and Selected Surface Waters. *The Journal of Parasitology,* **73**(4), pp.702-705.

MADSEN, O. S. 1971. On the generation of long waves. *Journal of Geophysical Research,* **76**(36), pp.8672-8683.

MAGNIEN, R. E. 2001. The Dynamics of Science, Perception, and Policy during the Outbreak of Pfiesteria in the Chesapeake Bay: An outbreak of Pfiesteria in the Chesapeake Bay in 1997 affected both fish and humans, leading to a media frenzy, intensive scientific investigations, and ultimately policy changes that would influence the national debate on pollution controls and the link between environmental and human health. *BioScience,* **51**(10), pp.843-852.

MALAJ, E., P. C. VON DER OHE, M. GROTE, R. KÜHNE, C. P. MONDY, P. USSEGLIO-POLATERA, W. BRACK and R. B. SCHÄFER. 2014. Organic chemicals jeopardize the health of freshwater ecosystems on the continental scale. *Proceedings of the National Academy of Sciences,* **111**(26), pp.9549-9554.

MALATRE, K. and P. GOSSE. 1995. Is it possible to influence water temperature and quality in the river Seine upstream of Paris in summer by managing the upstream reservoirs. *Water Science and Technology,* **31**(8), pp.67-77.

MALLE, K.-G. 1994. Accidental Spills–Frequency, Importance, Control, Countermeasures. *Water Science & Technology,* **29**(3), pp.149-163.

MALLIN, M. A., J. M. BURKHOLDER, M. R. MCIVER, G. C. SHANK, H. B. GLASGOW, B. W. TOUCHETTE and J. SPRINGER. 1997. Comparative Effects of Poultry and Swine Waste Lagoon Spills on the Quality of Receiving Streamwaters. *J. Environ. Qual.,* **26**(6), pp.1622-1631.

MANNINA, G. and G. VIVIANI. 2010. A hydrodynamic water quality model for propagation of pollutants in rivers. *Water Science and Technology,* **62**(2), pp.288-299.

MANYUMBA, F., E. WOOD and N. HORAN. 2009. Meeting the phosphorus consent with biological nutrient removal under UK winter conditions. *Water and Environment Journal,* **23**(2), pp.83-90.

MARJORIBANKS, T. I., R. J. HARDY, S. N. LANE and D. R. PARSONS. 2014. High-resolution numerical modelling of flow - Vegetation interactions. *Journal of Hydraulic Research,* **52**(6), pp.775-793.

MARSALEK, J., Q. ROCHFORT, B. BROWNLEE, T. MAYER and M. SERVOS. 1999. An exploratory study of urban runoff toxicity. *Water Science and Technology,* **39**(12), pp.33-39.

MARTINELLI, L. A., A. V. KRUSCH, R. L. VICTORIA, P. B. CAMARGO, M. BERNARDES, E. S. FERRAZ, J. M. MORAES and M. C. BALLESTER. 1999. Effects of sewage on the chemical composition of Piracicaba River, Brazil. *Water, Air, and Soil Pollution,* **110**, pp.67-79.

MASSELINK, G. and M. G. HUGHES. 2003. *Introduction to Coastal Processes and Geomorphology.* London: Hodder Arnold.

MATTSON, C. P., N. C. VALLARLO, D. J. SMITH, S. ANISFIELD and G. POTERA. 1977. Hackensak estuary oil spill: Cutting oil-soaked marsh grass as an innovative damage control technique. *International Oil Spill Conference Proceedings,* **1977**(1), pp.243-246.

MAYER, H. 1999. Air pollution in cities. *Atmospheric Environment,* **33**(24–25), pp.4029-4037.

MEAD, H. 2010. *What was learned from 1995's drought crisis* [online]. [Accessed 17/06/14]. Available from: http://www.thetelegraphandargus.co.uk/news/news_behind/8260818.Why_the_water_will_keep_on_flowing/.

MEYBECK, M. and R. HELMER. 1989. The quality of rivers: From pristine stage to global pollution. *Global and Planetary Change,* **1**(4), pp.283-309.

MINGHAM, C. and D. CAUSON. 1998. High-Resolution Finite-Volume Method for Shallow Water Flows. *Journal of Hydraulic Engineering,* **124**(6), pp.605-614.

MISKEWITZ, R. and C. UCHRIN. 2013. In-stream dissolved oxygen impacts and sediment oxygen demand resulting from combined sewer overflow discharges. *Journal of Environmental Engineering (United States),* **139**(10), pp.1307-1313.

MITCHELL, G. 2005. Mapping hazard from urban non-point pollution: a screening model to support sustainable urban drainage planning. *Journal of Environmental Management,* **74**(1), pp.1-9.

MITCHELL, G. N. 1991. Water quality issues in the British uplands. *Applied Geography,* **11**(3), pp.201-214.

MODENESI, K., L. T. FURLAN, E. TOMAZ, R. GUIRARDELLO and J. R. NÚNEZ. 2004. A CFD model for pollutant dispersion in rivers. *Brazilian Journal of Chemical Engineering,* **21**, pp.557-568.

MONAGHAN, J. J. and A. KOS. 2000. Scott Russell's wave generator. *Physics of Fluids (1994-present),* **12**(3), pp.622-630.

MONGE-CORELLA, S., J. GARCIA-PEREZ, N. ARAGONES, M. POLLAN, B. PEREZ-GOMEZ and G. LOPEZ-ABENTE. 2008. Lung cancer mortality in towns near paper, pulp and board industries in Spain: a point source pollution study. *BMC Public Health,* **8**(1), p288.

MOORE, D. 2005. Slug injection using salt in solution. *Streamline Watershed Management Bulletin,* **8**(2), pp.1-6.

MORRISON, G., O. S. FATOKI, L. PERSSON and A. EKBERG. 2001. Assessment of the impact of point source pollution from the Keiskammahoek Sewage Treatment Plant on the Keiskamma River - pH, electrical conductivity, oxygen-demanding substance (COD) and nutrients. *Water SA,* **27**(4), pp.475-480.

MUIRHEAD, R., W., R. DAVIES-COLLEY, J., A. DONNISON, M. and J. NAGELS, W. 2004. Faecal bacteria yields in artificial flood events: quantifying in-stream stores. *Water Research,* **38**, pp.1215-1224.

MULLISS, R. M., D. M. REVITT and R. B. SHUTES. 1996. The impacts of urban discharges on the hydrology and water quality of an urban watercourse. *Science of The Total Environment,* **189-190**, pp.385-390.

MURATA, A., H. TAKADA, K. MUTOH, H. HOSODA, A. HARADA and N. NAKADA. 2011. Nationwide monitoring of selected antibiotics: Distribution and sources of sulfonamides, trimethoprim, and macrolides in Japanese rivers. *Science of The Total Environment,* **409**(24), pp.5305-5312.

NAGY, A. S., J. SZABÓ and I. VASS. 2013. Trace metal and metalloid levels in surface water of Marcal River before and after the Ajka red mud spill, Hungary. *Environmental Science and Pollution Research,* **20**(11), pp.7603-7614.

NASH, J. 1957. The form of the instantaneous unit hydrograph. *IAHS Publ,* **45**(3), pp.114-121.

NATHAN, R. and T. MCMAHON. 1990. Evaluation of automated techniques for base flow and recession analyses. *Water Resources Research,* **26**(7), pp.1465-1473.

NEAL, C., H. DAVIES and M. NEAL. 2008. Water quality, nutrients and the water framework directive in an agricultural region: The lower Humber Rivers, northern England. *Journal of Hydrology,* **350**(3–4), pp.232-245.

NEAL, C., H. P. JARVIE, M. NEAL, A. LOVE, L. HILL and H. WICKHAM. 2005. Water quality of treated sewage effluent in a rural area of the upper Thames basin, southern England, and the impacts of such effluents on rivering phosphorus concentrations. *Journal of Hydrology,* **304**, pp.103-117.

NEAL, C., M. NEAL, L. HILL and H. WICKHAM. 2006. River water quality of the River Cherwell: An agricultural clay-dominated catchment in the upper Thames Basin, southeastern England. *Science of The Total Environment,* **360**(1-3), pp.272-289.

NEDEAU, E. J., R. W. MERRITT and M. G. KAUFMAN. 2003. The effect of an industrial effluent on an urban stream benthic community: water quality vs. habitat quality. *Environmental Pollution,* **123**(1), pp.1-13.

NEPF, H. M., C. G. MUGNIER and R. A. ZAVISTOSKI. 1997. The Effects of Vegetation on Longitudinal Dispersion. *Estuarine, Coastal and Shelf Science,* **44**(6), pp.675-684.

NICHOLAS, A. P., S. D. SANDBACH, P. J. ASHWORTH, M. L. AMSLER, J. L. BEST, R. J. HARDY, S. N. LANE, O. ORFEO, D. R. PARSONS, A. J. H. REESINK, G. H. SAMBROOK SMITH and R. N. SZUPIANY. 2012. Modelling hydrodynamics in the Rio Paraná, Argentina: An evaluation and inter-comparison of reduced-complexity and physics based models applied to a large sand-bed river. *Geomorphology,* **169-170**, pp.192-211.

NIXON, S. 2009. Eutrophication and the macroscope. *Hydrobiologia,* **629**(1), pp.5-19.

NORDIN, C. F. and B. M. TROUTMAN. 1980. Longitudinal dispersion in rivers: The persistence of skewness in observed data. *Water Resources Research,* **16**(1), pp.123-128.

OERTEL, M. and D. B. BUNG. 2012. Initial stage of two-dimensional dam-break waves: Laboratory versus VOF. *Journal of Hydraulic Research,* **50**(1), pp.89-97.

OKAY, O., H. GÜÇLÜ, E. SONER and T. BALKAŞ. 1985. Boron pollution in the Simav River, Turkey and various methods of boron removal. *Water Research,* **19**(7), pp.857-862.

OKUNISH, K., T. SAITO and T. YOSHIDA. 1992. Accuracy of stream gauging by dilution methods. *Journal of Hydrology,* **137**, pp.231-243.

OLD, G. H., G. J. L. LEEKS, J. C. PACKMAN, B. P. G. SMITH, S. LEWIS, E. J. HEWITT, M. HOLMES and A. YOUNG. 2003. The impact of a convectional summer rainfall event on river flow and fine sediment transport in a highly urbanised catchment: Bradford, West Yorkshire. *The Science of The Total Environment,* **314-316**, pp.495-512.

OLNESS, A. 1995. Water Quality: Prevention, Identification and Management of Diffuse Pollution. *J. Environ. Qual.,* **24**(2), pp.383-383.

ONGERTH, J. E. and H. H. STIBBS. 1987. Identification of Cryptosporidium oocysts in river water. *Applied and Environmental Microbiology,* **53**(4), pp.672-676.

ORT, C. and H. SIEGRIST. 2009. Assessing wastewater dilution in small rivers with high resolution conductivity probes. *Water Sci. Technol,* **59**(8), pp.1593-1601.

OS. 2014. *Map of South Penninies*, 1:25000. Ordnance Survey.

OUATTARA, N. K., T. GARCIA-ARMISEN, A. ANZIL, N. BRION and P. SERVAIS. 2014. Impact of Wastewater Release on the Faecal Contamination of a Small Urban River: The Zenne River in Brussels (Belgium). *Water, Air, & Soil Pollution,* **225**(8), pp.1-12.

PÅHLSSON, A.-M. 1989. Toxicity of heavy metals (Zn, Cu, Cd, Pb) to vascular plants. *Water, Air, and Soil Pollution,* **47**(3-4), pp.287-319.

PANETH, N., P. VINTEN-JOHANSEN, H. BRODY and M. RIP. 1998. A rivalry of foulness: official and unofficial investigations of the London cholera epidemic of 1854. *American Journal of Public Health,* **88**(10), pp.1545-1553.

PARK, J. C., M. H. KIM, H. MIYATA and H. H. CHUN. 2003. Fully nonlinear numerical wave tank (NWT) simulations and wave run-up prediction around 3-D structures. *Ocean Engineering,* **30**(15), pp.1969-1996.

PARK, J. G. and L. R. CURTIS. 1997. Mercury distribution in sediments and bioaccumulation by fish in two oregon reservoirs: Point-source and nonpoint-source impacted systems. *Archives of Environmental Contamination and Toxicology,* **33**(4), pp.423-429.

PEREZ-TELLO, M., H. SOHN and P. SMITH. 2001. Experimental investigation and three-dimensional computational fluid-dynamics modeling of the flash-converting furnace shaft: Part II. Formulation of three-dimensional computational fluid-dynamics model incorporating the particle-cloud description. *Metallurgical and Materials Transactions B,* **32**(5), pp.869-886.

PETTICREW, E. L., A. KREIN and D. E. WALLING. 2007. Evaluating fine sediment mobilization and storage in a gravel-bed river using controlled reservoir releases. *Hydrological Processes,* **21**(2), pp.198-210.

PETTS, G. E. 1984. *Impounded Rivers: Perspectives for Ecological Management.* Chichester: Wiley.

PETTS, G. E., T. R. FOULGER, D. J. GILVEAR, J. D. PRATTS and M. C. THOMS. 1985. Wave-movement and water-quality variations during a controlled release from kielder reservoir, north Tyne River, U.K. *Journal of Hydrology,* **80**, pp.371-389.

PICKUP, G. and R. J. HIGGINS. 1979. Estimating sediment transport in a braided gravel channel — The Kawerong River, Bougainville, Papua New Guinea. *Journal of Hydrology,* **40**(3–4), pp.283-297.

PIERSON, W. J. 1962. Perturbation analysis of the Navier-Stokes equations in Lagrangian form with selected linear solutions. *Journal of Geophysical Research,* **67**(8), pp.3151-3160.

POKROVSKY, O. S., A. FEURTET-MAZEL, R. E. MARTINEZ, S. MORIN, M. BAUDRIMONT, T. DUONG and M. COSTE. 2010. Experimental study of cadmium interaction with periphytic biofilms. *Applied Geochemistry,* **25**(3), pp.418-427.

PONCE, V. 1991. Kinematic Wave Controversy. *Journal of Hydraulic Engineering,* **117**(4), pp.511-525.

PRETTY, J. N., C. BRETT, D. GEE, R. E. HINE, C. F. MASON, J. I. L. MORISON, H. RAVEN, M. D. RAYMENT and G. VAN DER BIJL. 2000. An assessment of the total external costs of UK agriculture. *Agricultural Systems,* **65**(2), pp.113-136.

PRITCHARD, D. and A. J. HOGG. 2002. On sediment transport under dam-break flow. *Journal of Fluid Mechanics,* **473**, pp.265-274.

PURDOM, C. E., P. A. HARDIMAN, V. V. J. BYE, N. C. ENO, C. R. TYLER and J. P. SUMPTER. 1994. Estrogenic Effects of Effluents from Sewage Treatment Works. *Chemistry and Ecology,* **8**(4), pp.275-285.

QURESHI, M. E., J. CONNOR, M. KIRBY and M. MAINUDDIN. 2007. Economic assessment of acquiring water for environmental flows in the Murray Basin*. *Australian Journal of Agricultural and Resource Economics,* **51**(3), pp.283-303.

RASKIN, I., P. B. A. N. KUMAR, S. DUSHENKOV and D. E. SALT. 1994. Bioconcentration of heavy metals by plants. *Current Opinion in Biotechnology,* **5**(3), pp.285-290.

REED, M., Ø. JOHANSEN, P. J. BRANDVIK, P. DALING, A. LEWIS, R. FIOCCO, D. MACKAY and R. PRENTKI. 1999. Oil Spill Modeling towards the Close of the 20th Century: Overview of the State of the Art. *Spill Science & Technology Bulletin,* **5**(1), pp.3-16.

RENZETTI, S. 2002. Water Demand Forecasting. *The Economics of Water Demands.* Springer, pp.145-156.

of on Reservoirs Act.

REVITT, D. M., J. B. ELLIS and N. R. LLEWELLYN. 2002. Seasonal removal of herbicides in urban runoff. *Urban Water,* **4**(1), pp.13-19.

RHOADS, D. C., P. L. MCCALL and J. Y. YINGST. 1978. Disturbance and Production on the Estuarine Seafloor: Dredge-spoil disposal in estuaries such as Long Island Sound can be managed in ways that enhance productivity rather than diminish it. *American Scientist,* **66**(5), pp.577-586.

RICHARDSON, K. and P. A. CARLING. 2006. The hydraulics of a straight bedrock channel: Insights from solute dispersion studies. *Geomorphology,* **82**(1–2), pp.98-125.

ROBSON, M., K. SPENCE and L. BEECH. 2006. Stream quality in a small urbanised catchment. *Science of The Total Environment,* **357**(1–3), pp.194-207.

ROGERS, K. 1998. Managing science/management partnerships: a challenge of adaptive management. *Conservation ecology,* **2**(2), pR1.

ROLLS, R. J., A. J. BOULTON, I. O. GROWNS and S. E. MAXWELL. 2011. Response by fish assemblages to an environmental flow release in a temperate coastal Australian river: A paired catchment analysis. *River Research and Applications,* **27**(7), pp.867-880.

ROTHWELL, J. J., S. G. ROBINSON, M. G. EVANS, J. YANG and T. E. H. ALLOTT. 2005. Heavy metal release by peat erosion in the Peak District, southern Pennines, UK. *Hydrological Processes,* **19**(15), pp.2973-2989.

ROULEAU, S., P. LESSARD and D. BELLEFLEUR. 1997. Simulation of a transient failure in a wastewater treament plant: A case study. *Water Science and Technology,* **36**(5), pp.349-355.

ROUT, P. R., P. BHUNIA and R. R. DASH. 2014. Modeling isotherms, kinetics and understanding the mechanism of phosphate adsorption onto a solid waste: Ground burnt patties. *Journal of Environmental Chemical Engineering,* **2**(3), pp.1331-1342.

RUIZ, M. and J. VELASCO. 2010. Nutrient Bioaccumulation in Phragmites australis: Management Tool for Reduction of Pollution in the Mar Menor. *Water, Air, and Soil Pollution,* **205**(1-4), pp.173-185.

RÜTHER, N., J. JACOBSEN, N. R. B. OLSEN and G. VATNE. 2010. Prediction of the three-dimensional flow field and bed shear stresses in a regulated river in mid-Norway. *Hydrological Research,* **41**(2), pp.145-152.

RÜTHER, N. and Ø. PEDERSEN. 2014. 3D numerical modeling of flow over a gravel river bed due to hydropower peaking. *In: 37. Dresdner Wasserbaukolloquium 2014 Dresden*. Institut für Wasserbau und Technische Hydromechanik.

RUTHERFORD, J. C. 1994. *River Mixing.* Chichester: John Wiley and Sons.

RYU, S., M. H. KIM and P. J. LYNETT. 2003. Fully nonlinear wave-current interactions and kinematics by a BEM-based numerical wave tank. *Computational Mechanics,* **32**(4-6), pp.336-346.

SAGET, A., G. CHEBBO and J.-L. BERTRAND-KRAJEWSKI. 1996. The first flush in sewer systems. *Water Science and Technology,* **33**(9), pp.101-108.

SANDERS, G., K. C. JONES, J. HAMILTON-TAYLOR and H. DORR. 1995. PCB and PAH fluxes to a dated UK peat core. *Environmental Pollution,* **89**(1), pp.17-25.

SANDERS, M., S. SIVERTSEN and G. SCOTT. 2002. Origin and Distribution of Polycyclic Aromatic Hydrocarbons in Surficial Sediments from the Savannah River. *Archives of Environmental Contamination and Toxicology,* **43**(4), pp.0438-0448.

SCHINDLER, D. W., R. E. HECKY, D. L. FINDLAY, M. P. STAINTON, B. R. PARKER, M. J. PATERSON, K. G. BEATY, M. LYNG and S. E. M. KASIAN. 2008. Eutrophication of lakes cannot be controlled by reducing nitrogen input: Results of a 37-year whole-ecosystem experiment. *Proceedings of the National Academy of Sciences,* **105**(32), pp.11254-11258.

SCHMID, B. 2002. Persistence of Skewness in Longitudinal Dispersion Data: Can the Dead Zone Model Explain It After All? *Journal of Hydraulic Engineering,* **128**(9), pp.848-854.

SCHOLEFIELD, D., T. LE GOFF, J. BRAVEN, L. EBDON, T. LONG and M. BUTLER. 2005. Concerted diurnal patterns in riverine nutrient concentrations and physical conditions. *Science of The Total Environment,* **344**(1–3), pp.201-210.

SCHOLES, L. N. L., R. B. E. SHUTES, D. M. REVITT, D. PURCHASE and M. FORSHAW. 1999. The removal of urban pollutants by constructed wetlands during wet weather. *Water Science and Technology,* **40**(3), pp.333-340.

SEAGER, J. and L. MALTBY. 1989. Assessing the impact of episodic pollution. *Hydrobiologia,* **188-189**(1), pp.633-640.

SEAMAN, J. C., J. S. AREY and P. M. BERTSCH. 2001. Immobilization of Nickel and Other Metals in Contaminated Sediments by Hydroxyapatite Addition. *J. Environ. Qual.,* **30**(2), pp.460-469.

SEO, I. and K. BAEK. 2004. Estimation of the Longitudinal Dispersion Coefficient Using the Velocity Profile in Natural Streams. *Journal of Hydraulic Engineering,* **130**(3), pp.227-236.

SEO, I., K. O. BAEK and T. M. JEON. 2006. Analysis of transverse mixing in natural streams under slug tests. *Journal of Hydraulic Research,* **44**(3), pp.350-362.

SHAW, E., M. 1988. *Hydrology in Practice.* New York: Chapman & Hall.

SHENG, Y., Y. QU, C. DING, Q. SUN and R. J. G. MORTIMER. 2013. A combined application of different engineering and biological techniques to remediate a heavily polluted river. *Ecological Engineering,* **57**(0), pp.1-7.

SHIH, T.-H., W. W. LIOU, A. SHABBIR, Z. YANG and J. ZHU. 1995. A new k-ϵ eddy viscosity model for high reynolds number turbulent flows. *Computers & Fluids,* **24**(3), pp.227-238.

SHILLER, A., M. and E. BOYLE. 1985. Dissolved zinc in rivers. *Nature,* **317**(6032), pp.49-52.

SHIRAKURA, Y., K. TANIZAWA and S. NAITO. Development of 3-D Fully Nonlinear Numerical Wave Tank to Simulate Floating Bodies Interacting With Water Waves. *In*, *2000/1/1/*. International Society of Offshore and Polar Engineers.

SHOTBOLT, L., S. M. HUTCHINSON and A. D. THOMAS. 2006. Sediment Stratigraphy and Heavy Metal Fluxes to Reservoirs in the Southern Pennine Uplands, UK. *Journal of Paleolimnology,* **35**(2), pp.305-322.

SINGH, V. P. 1995. Accuracy of kinematic wave and diffusion wave approximations for space-independent flows on infiltrating surfaces with lateral inflow neglected in the momentum equation. *Hydrological Processes,* **9**(7), pp.783-796.

SKINNER, J. A., K. A. LEWIS, K. S. BARDON, P. TUCKER, J. A. CATT and B. J. CHAMBERS. 1997. An Overview of the Environmental Impact of Agriculture in the U.K. *Journal of Environmental Management,* **50**(2), pp.111-128.

SLAWSON, P. and G. CSANADY. 1967. On the mean path of buoyant, bent-over chimney plumes. *Journal of Fluid Mechanics,* **28**(02), pp.311-322.

SOONTHORNNONDA, P. and E. R. CHRISTENSEN. 2008. Source apportionment of pollutants and flows of combined sewer wastewater. *Water research,* **42**(8), pp.1989-1998.

SOZANSKI, M. M., E. S. KEMPA, K. GROCHOLSKI and J. BIEN. 1997. The rheological experiment in sludge properties research. *Water Science and Technology,* **36**(11), pp.69-78.

SPAULDING, M. L. 1988. A state-of-the-art review of oil spill trajectory and fate modeling. *Oil and Chemical Pollution,* **4**(1), pp.39-55.

STEEGEN, A., G. GOVERS, J. NACHTERGAELE, I. TAKKEN, L. BEUSELINCK and J. POESEN. 2000. Sediment export by water from an agricultural catchment in the Loam Belt of central Belgium. *Geomorphology,* **33**(1–2), pp.25-36.

STEWART, G., M. AIDAR, C. JOLY and S. SCHMIDT. 2002. Impact of point source pollution on nitrogen isotope signatures (δ15N) of vegetation in SE Brazil. *Oecologia,* **131**(3), pp.468-472.

STEWART, T. R., K. F. HEIDEMAN, W. R. MONINGER and P. REAGAN-CIRINCIONE. 1992. Effects of improved information on the components of skill in weather forecasting. *Organizational Behavior and Human Decision Processes,* **53**(2), pp.107-134.

STIGEBRANDT, A., B. LILJEBLADH, L. DE BRABANDERE, M. FORTH, Å. GRANMO, P. HALL, J. HAMMAR, D. HANSSON, M. KONONETS, M. MAGNUSSON, F. NORÉN, L. RAHM, A. TREUSCH and L. VIKTORSSON. 2014. An Experiment with Forced Oxygenation of the Deepwater of the Anoxic By Fjord, Western Sweden. *AMBIO*, pp.1-13.

STOKER. 1958. Long Waves in Shallow Water. *Water Waves, The Mathematical Theory with Applications.* New York: John Wiley & Sons, pp.291-305.

STOKES, G. G. 1847. On the theory of oscillatory waves. *Transactions of the Cambridge Philosophy society,* **8**, pp.441-445.

STOREY, E., S. KAYES, I. DE VRIES and C. FRANKLIN. 2008. Effect of water depth, velocity and temperature on the surfacing frequency of the bimodally respiring turtle Elseya albagula. *Functional ecology,* **22**(5), pp.840-846.

STRUTHERS, A. and J. T. ILLIDGE. Risk assessment: an object oriented tool for the rapid evaluation of liquid spill potential and loss prevention. *Computers & Chemical Engineering,* **19, Supplement 1**(0), pp.453-458.

SUBRA, W. and J. WATERS. 1996. Non point source pollution. *In: Geoscience and Remote Sensing Symposium, 1996. IGARSS '96. 'Remote Sensing for a Sustainable Future.', International,* 27-31 May 1996, pp.2231-2233 vol.4.

SVENDSEN, I. A. and C. STAUB. 1981. Horizontal particle velocities in long waves. *Journal of Geophysical Research: Oceans,* **86**(C5), pp.4138-4148.

SWAN, C. 1990. Convection within an experimental wave flume. *Journal of Hydraulic Research,* **28**(3), pp.273-282.

SYCHEV, V., V.,, V. SYCHEV, V.,, A. RUBAN, I., and G. KOROLEV, L.,. 1998. *Asymptotic Theory of Separated Flows.* Cambridge: Cambridge Univeristy Press.

TAEBI, A. and R. L. DROSTE. 2004a. First flush pollution load of urban stormwater runoff. *Journal of Environmental Engineering and Science,* **3**(4), pp.301-309.

TAEBI, A. and R. L. DROSTE. 2004b. Pollution loads in urban runoff and sanitary wastewater. *Science of The Total Environment,* **327**(1-3), pp.175-184.

TANIK, A., B. BELER BAYKAL and I. E. GONENC. 1999. The impact of agricultural pollutants in six drinking water reservoirs. *Water Science and Technology,* **40**(2), pp.11-17.

TAYFUR, G. and V. SINGH. 2005. Predicting Longitudinal Dispersion Coefficient in Natural Streams by Artificial Neural Network. *Journal of Hydraulic Engineering,* **131**(11), pp.991-1000.

TAYLOR, G. 1953. Dispersion of soluble matter in solvent flowing slowly through a tube. *Proceedings of the Royal Society of London. Series A. Mathematical and Physical Sciences,* **219**(1137), pp.186-203.

TEALDI, S., C. CAMPOREALE, E. PERUCCA and L. RIDOLFI. 2010. Longitudinal dispersion in vegetated rivers with stochastic flows. *Advances in Water Resources,* **33**(5), pp.562-571.

TERNES, T. A. 1998. Occurrence of drugs in German sewage treatment plants and rivers. *Water Research,* **32**(11), pp.3245-3260.

THOMAS, R. E., D. R. PARSONS, S. D. SANDBACH, G. M. KEEVIL, W. A. MARRA, R. J. HARDY, J. L. BEST, S. N. LANE and J. A. ROSS. 2011. An experimental study of discharge partitioning and flow structure at symmetrical bifurcations. *Earth Surface Processes and Landforms,* **36**(15), pp.2069-2082.

TIPTON, D. K., D. W. GOZY and D. A. COLEMAN. 2000. *Hazardous materials and waste reduction management system*. Google Patents.

TKALICH, P. and E. S. CHAN. 2002. Vertical mixing of oil droplets by breaking waves. *Marine Pollution Bulletin,* **44**(11), pp.1219-1229.

TONIOLO, H., P. HARFF, J. MARR, C. PAOLA and G. PARKER. 2004. Experiments on reworking by successive unconfined subaqueous and subaerial muddy debris flows. *Journal of Hydraulic Engineering,* **130**(1), pp.38-48.

TOOLBOX, E. 2010. *Material Properties* [online]. [Accessed 07/04/2012]. Available from: https://www.engineeringtoolbox.com.

TURNER, R. E. and N. N. RABALAIS. 2003. Linking landscape and water quality in the Mississippi River basin for 200 years. *BioScience,* **53**(6), pp.563-572.

TUROWSKI, J. M., A. BADOUX, J. LEUZINGER and R. HEGGLIN. 2013. Large floods, alluvial overprint, and bedrock erosion. *Earth Surface Processes and Landforms,* **38**(9), pp.947-958.

GREAT BRITAIN. U. NATIONS. 1982. *United Nations Convention on the Law of the Sea of 1 December 1982*. UN.

UN. 2000. *Millenium Goals: Ensure Sustainability* [online]. [Accessed 03/06/2014]. Available from: http://www.un.org/millenniumgoals/environ.shtml.

VAN DER WEIJDEN, C. H. and J. J. MIDDELBURG. 1989. Hydrogeochemistry of the River Rhine: Long term and seasonal variability, elemental budgets, base levels and pollution. *Water Research,* **23**(10), pp.1247-1266.

VAN RIJN, L. C., L. C. VAN RIJN and L. C. VAN RIJN. 1993. *Principles of sediment transport in rivers, estuaries and coastal seas.* Aqua publications Amsterdam.

VAROL, M., B. GÖKOT, A. BEKLEYEN and B. ŞEN. 2012. Water quality assessment and apportionment of pollution sources of the Tigris River (Turkey) using multivariate statistical techniques - a case study. *River Research and Applications,* **28**(9), pp.1428-1438.

VOGT, T., E. HOEHN, P. SCHNEIDER, A. FREUND, M. SCHIRMER and O. A. CIRPKA. 2010. Fluctuations of electrical conductivity as a natural tracer for bank filtration in a losing stream. *Advances in Water Resources,* **33**(11), pp.1296-1308.

VÖRÖSMARTY, C. J., P. B. MCINTYRE, M. O. GESSNER, D. DUDGEON, A. PRUSEVICH, P. GREEN, S. GLIDDEN, S. E. BUNN, C. A. SULLIVAN, C. R. LIERMANN and P. M. DAVIES. 2010. Global threats to human water security and river biodiversity. *Nature,* **467**(7315), pp.555-561.

WAI, O. and Q. LU. 1999. Gradient-Adaptive-Sigma (GAS) Grid for 3D Mass-Transport Modeling. *Journal of Hydraulic Engineering,* **125**(2), pp.141-151.

WALLIS, S., P. YOUNG and K. BEVEN. 1989. EXPERIMENTAL INVESTIGATION OF THE AGGREGATED DEAD ZONE MODEL. *In: ICE Proceedings*: Thomas Telford, pp.1-22.

WARNER, N. R., C. A. CHRISTIE, R. B. JACKSON and A. VENGOSH. 2013. Impacts of Shale Gas Wastewater Disposal on Water Quality in Western Pennsylvania. *Environmental Science & Technology,* **47**(20), pp.11849-11857.

WATKINS, K., L. CARVAJAL, D. COPPARD, R. FUENTES, A. GHOSH, C. GIAMBERARDINI, C. JOHANSSON, P. SECK, C. UGAZ and S. YAQUB. 2006. *Human Development Report 2006, Beyond Scarcity: Power, poverty and the global water crisis*. New York: United Nations.

WEBER, J. E. H. 2011. Do we observe Gerstner waves in wave tank experiments? *Wave Motion,* **48**(4), pp.301-309.
WEI, W. and W.-P. CAO. 2013. Application of Biofilm in Biological-ecological Restoration to Control Polluted Water. *Water Saving Irrigation,* **11**, p012.
WENDEL, M. W., P. T. WILLIAMS and J. H. PLATFOOT. 1997. CFD modeling of the dispersion of contaminants in the clinch river. *In: International Water Conference, Pittsburg, USA*. USGS.
WHITEHOUSE, P., M. CRANE, C. JOHN REDSHAW and C. TURNER. 1996. Aquatic toxicity tests for the control of effluent discharges in the UK — the influence of test precision. *Ecotoxicology,* **5**(3), pp.155-168.
WHITHAM, G. B. 2011. *Linear and nonlinear waves.* John Wiley & Sons.
WIEGEL, R. L. 1960. A presentation of cnoidal wave theory for practical application. *Journal of Fluid Mechanics,* **7**(02), pp.273-286.
WILKINSON, H. 2011. *CHEMICAL STANDARDS DATABAS* [online]. [Accessed 05/04/2014]. Available from: http://evidence.environment-agency.gov.uk/ChemicalStandards.
WILSENACH, J., M. MAURER, T. LARSEN and M. VAN LOOSDRECHT. 2003. From waste treatment to integrated resource management. *Water Science & Technology,* **48**(1), pp.1-9.
WOITKE, P., J. WELLMITZ, D. HELM, P. KUBE, P. LEPOM and P. LITHERATY. 2003. Analysis and assessment of heavy metal pollution in suspended solids and sediments of the river Danube. *Chemosphere,* **51**(8), pp.633-642.
WORRALL, F. and T. BURT. 2005. Predicting the future DOC flux from upland peat catchments. *Journal of Hydrology,* **300**(1–4), pp.126-139.
WU, Y., L. HUI, H. WANG, Y. LI and R. ZENG. 2007. Effectiveness of riverbank filtration for removal of nitrogen from heavily polluted rivers: a case study of Kuihe River, Xuzhou, Jiangsu, China. *Environmental Geology,* **52**(1), pp.19-25.
XIA, J., B. LIN, B. LIN and Y. WANG. 2012. Modelling of man-made flood routing in the lower Yellow River, China. *Proceedings of the ICE-Water Management,* **165**(7), pp.377-391.
XIA, Y., Y. LI, X. LI, M. GUO, D. SHE and X. YAN. 2013. Diurnal pattern in nitrous oxide emissions from a sewage-enriched river. *Chemosphere,* **92**(4), pp.421-428.
XIAO, J., H. WANG, S. CHU and M.-H. WONG. 2012. Dynamic remediation test of polluted river water by Eco-tank system. *Environmental Technology,* **34**(4), pp.553-558.
XU, F., H. ZHOU, J. ZHOU and X. YANG. 2012. A mathematical model for forecasting the dam-break flood routing process of a landslide dam. *Mathematical Problems in Engineering,* **2012**.
XU, J. 2014. The influence of dilution on downstream channel sedimentation in large rivers: the Yellow River, China. *Earth Surface Processes and Landforms,* **39**(4), pp.450-462.
YANTASEE, W., C. L. WARNER, T. SANGVANICH, R. S. ADDLEMAN, T. G. CARTER, R. J. WIACEK, G. E. FRYXELL, C. TIMCHALK and M. G. WARNER. 2007. Removal of Heavy Metals from Aqueous Systems with Thiol Functionalized Superparamagnetic Nanoparticles. *Environmental Science & Technology,* **41**(14), pp.5114-5119.
YAPA, P. D. and H. TAO SHEN. 1994. Modelling river oil spills: a review. *Journal of Hydraulic Research,* **32**(5), pp.765-782.
YEH, W. W. G. 1985. Reservoir management and operations models: A state-of-the-art review. *Water resources research,* **21**(12), pp.1797-1818.
ZHANG, W. and D. ZHU. 2011. Transverse Mixing in an Unregulated Northern River. *Journal of Hydraulic Engineering,* **137**(11), pp.1426-1440.
ZHANG, X. T., B. C. KHOO and J. LOU. 2006. Wave propagation in a fully nonlinear numerical wave tank: A desingularized method. *Ocean Engineering,* **33**(17–18), pp.2310-2331.

ZHAO, T., X. CAI and D. YANG. 2011. Effect of streamflow forecast uncertainty on real-time reservoir operation. *Advances in Water Resources,* **34**(4), pp.495-504.

ZHAO, T., J. ZHAO, J. R. LUND and D. YANG. 2014. Optimal Hedging Rules for Reservoir Flood Operation from Forecast Uncertainties. *Journal of Water Resources Planning and Management*.