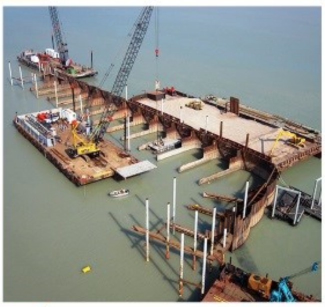
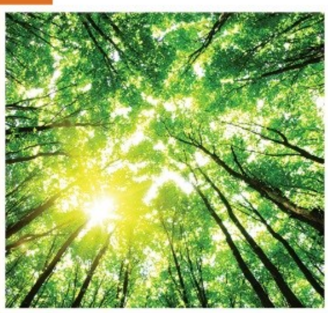
Site Assessment Report – Phase 2

Report No. PH 00857 00





Appendix A – Marine & River Outfall Modelling

A

<u>*Document Ref. No. 1207/2/15P*</u>

Preliminary Report

Arklow WWTP
Investigation of the Impact of
Treated Wastewater Discharges
To
Avoca River and Irish Sea

Consulting Engineers:

Byrne Looby PHMcCarthy Limited
Consulting Engineers
Dublin

Prepared by:

Irish Hydrodata Limited
Ballygarvan
Cork

Ph. 021-4311255
Fax 021-4311740
e-mail: admin@hydrodata.ie

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Revision History	Note	Date
1207/1/15D	Draft issued to BL-PHMcCfor comment	5/4/15
1207/2/15P	Preliminary report issued	15/4/15

Glossary

ADF	Average Daily Flow
ATT	Admiralty Tide Tables
BOD	Biochemical Oxygen Demand
DIN	Dissolved Inorganic Nitrogen (as N)
DWF	Dry Weather Flow
EC	E.Coli
ELV	Emission Limit Value
EPA	Environmental Protection Agency
IHD	Irish Hydrodata Ltd
MHWN	Mean High Water Neap
MHWS	Mean High Water Spring
MLWN	Mean Low Water Neap
MLWS	Mean Low Water Spring
NHA	National Heritage Area
OPW	Office of Public Works
PE	Population Equivalent
SAC	Special Area of Conservation
SS	Suspended Solids
T90	Decay time
T_{90}	E.Coli Decay Time
TA	Total Ammonia (as N)
TON	Total Oxidised Nitrogen (as N)
UWTR	Urban Wastewater Treatment Regulations
WFD	Water Framework Directive
WQ	Water Quality
WWTP	Wastewater Treatment Plant

1. Introduction

1.1 Background Information

Arklow is a significant urban centre on the east coast. It is served by an outdated sewage system from which untreated municipal wastewaters discharge directly into the harbour. A treatment plant has been in planning for a number of years and various detailed designs including marine outfall studies have been completed. Improved treatment technologies and plant operation now facilitate discharges to waterbodies which would not have been possible in the past. Recent investigative studies by consulting engineers Byrne Looby PHMcCarthy have identified additional potentially suitable treatment plant sites on the seafront and to the west of the town (Figure 1.1). This study seeks to examine the possible impacts of discharges to the nearby waterbodies from a plant located in either of these environs.

There are three waterbodies in the locality identified under the Water Framework Directive (WFD). These are listed in Table 1.1 and illustrated in Figure 1.2. The results of the WFD monitoring programme indicate that there are some water quality issues with the Lower Avoca river and the Avoca estuary. These relate to historic leakages from upstream mines and untreated municipal wastewater discharges to the estuary. Arklow has numerous sandy beaches, all of which are used extensively during the summer months. The beaches at Brittas Bay and Clogga (Figure 1.3) are designated bathing waters.

There are two marine SAC's in the vicinity; these are the Wicklow Head reef and the Blackwater Bank (Figure 1.4). The Arklow town marsh, located on the northern bank of the Avoca river, is a proposed NHA (Figure 1.5).

1.2 Study Brief

The purpose of the study was to:

- make an assessment of effects of treated wastewater discharges to the Avoca river and the Arklow coastal area;
- establish suitable effluent discharge standards;
- ensure compliance with all EC and national regulations;
- assess and compare potential outfall locations.

The brief called for various scenarios to be focused on. In the marine these include spring and neap tides and calm and windy conditions. The river discharge focused on 95%ile flows in the Avoca. Under the Urban Wastewater Treatment Regulations 2001 secondary treatment of effluent is mandatory. This will significantly reduce overall biological impacts. The main concerns regarding the proposed discharges are the impacts on nutrient levels and on bacterial concentrations in nearby bathing waters.

1.3 Regulatory Framework

The main regulatory constraints that apply to the discharges are:

* Urban Wastewater Treatment Regulations 2001 (SI 254/2001);
* European Communities (Water Policy) Regulations (SI 722/2003);
* European Communities Environmental Objectives (Surface Waters) Regs 2009 (SI 272/2009);
* Bathing Water Quality Regulations 2008 (SI 79/2008);
* European Communities (Quality of Salmonid Waters) Regulations 1988 (SI 293/1988).

1.4 Summary of Study Works

The study consisted of a review of available data and previous reports. (Irish Hydrodata Ltd conducted outfall investigations for the Arklow WWTP in 1985, 1991 and 2005). Subsequently hydrodynamic & water quality models were constructed to simulate the impacts of the proposed discharges, allow comparisons to be made and suitable discharge standards to be set.

Waterbody	Risk Scores	WFD Status 2012	Quality
Avoca Lower River	At risk of not achieving Good	Unassigned	Moderate
Avoca Estuary Transitional	At risk of not achieving Good	Moderate	Intermediate
Coastal, Brittas Bay HA10	Expected to achieve Good	Good	Unpolluted

Table 1.1 - Local WFD waterbodies.

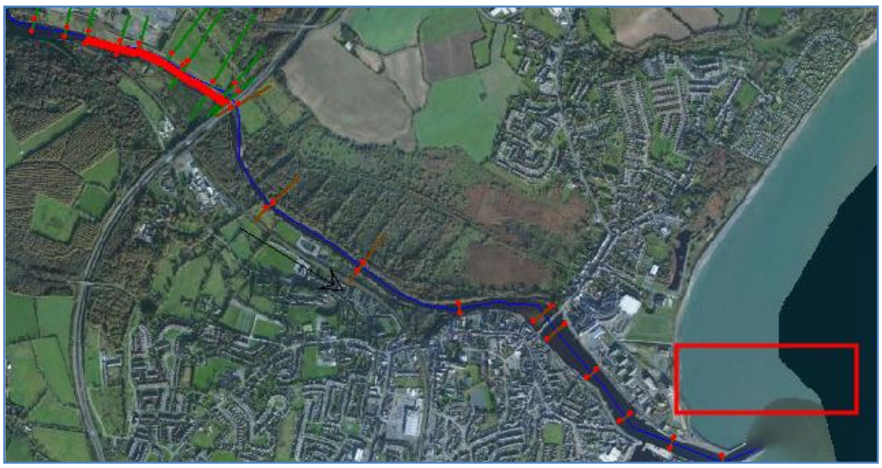


Figure 1.1 – Potential outfall points on Avoca river or to coastal waterbody

Figure 1.1 - Local WFD waterbodies:

Avoca River,
Avoca Estuary,
Brittas Bay (HA10)

Figure 1.2 - Designated bathing waters

Figure 1.3 - Wicklow Head (SAC 2274), & Blackwater Bank (SAC 2953)

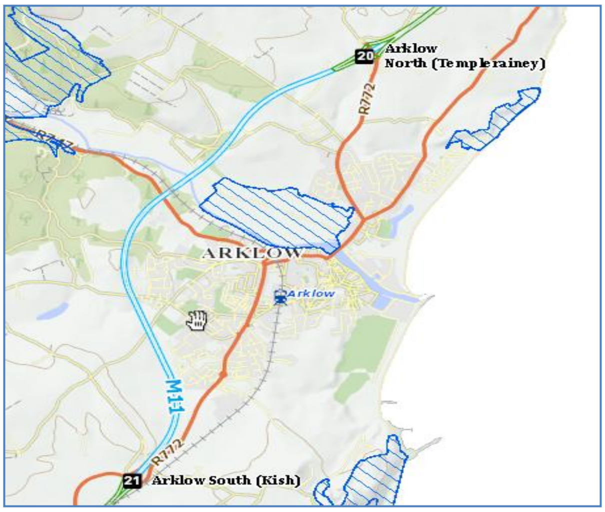


Figure 1.4 - Proposed NHA sites

2. Area Characteristics

2.1 Coastal Bathymetry

The general bathymetry for the Arklow area is available on the Admiralty chart of the area (ref:1) and is presented in Figure 2.1.

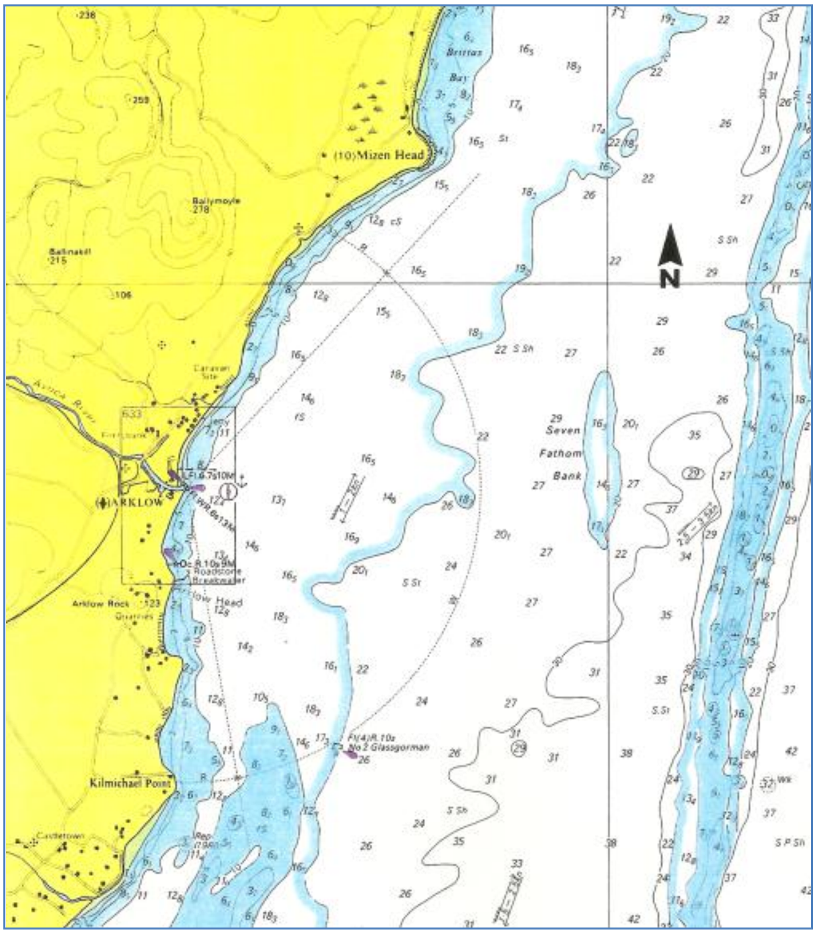


Figure 2.1 - Coastal bathymetry

2.2 Tidal Levels

Tidal patterns in the locality are semi-diurnal. Ranges are small and the tidal elevation curves are somewhat complex due to the proximity of a degenerate amphidrome near Courtown (ref:2). The Admiralty Tide Tables (ATT) publication NP-201-15 (ref: 3) provides summary tidal level information for Arklow based on historic information. This data is presented in Table 2.1. In 1985 Irish Hydrodata Ltd (IHD) conducted detailed studies in the area as part of outfall investigations (ref:4). Digital tidal data was collected for 30 days and fully analysed. Derived statistics are also included in Table 2.1. The OPW operate a water level recorder in Arklow Docks (Figure 2.2). Comparison of the OPW data with IHD data indicates that the ATT are underestimating the statistical water levels by between 0.05 and 0.15m. Therefore the IHD data is used for this study.

Figure 2.3 shows a prediction of water levels for 2015 relative to Malin Head datum. The associated percentage exceedance plot is shown in Figure 2.4.

Tide	Tide	ATT Level OD Malin	IHD Level OD Malin
MLWS	Mean high water springs	-0.53	-0.44
MLWN	Mean high water neaps	-0.23	-0.14
MHWN	Mean low water neaps	0.07	0.12
MHWS	Mean low water springs	0.27	0.42

Table 2.1 - Summary tidal statistics

Figure 2.2 - OPW water level gauge locations. (www.waterlevel.ie)

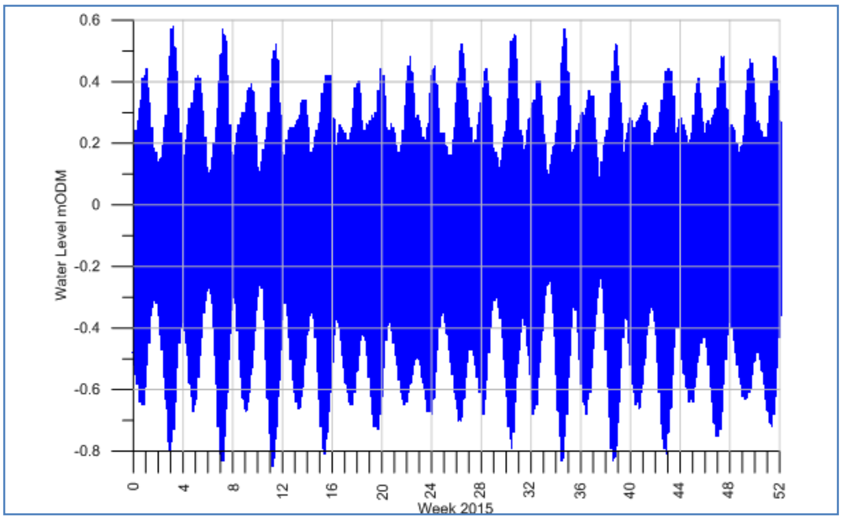


Figure 2.3 - Hourly tidal prediction for 2015 relative to Malin Head datum

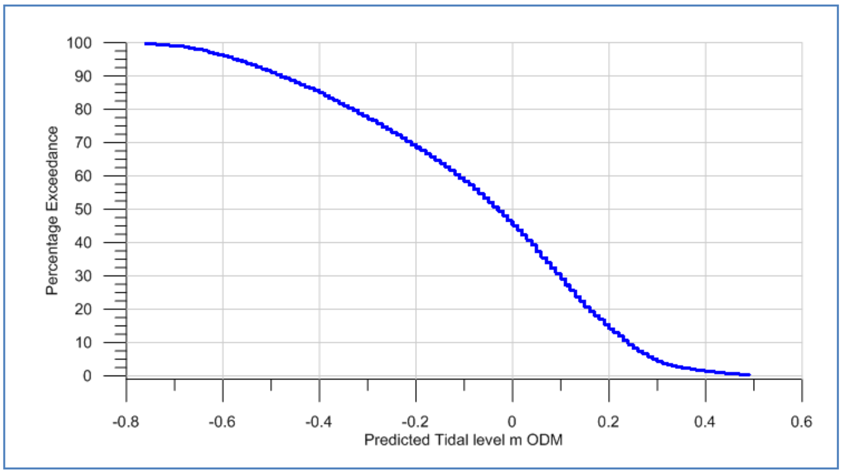


Figure 2.4 - Percentage exceedance of tidal level for 2015

2.3 Coastal Oceanography

Previously Irish Hydrodata Ltd conducted detailed studies at Arklow for various marine long sea outfalls and a possible river discharge. These studies were conducted between 1985 and 2005 (refs:4-6). The information on physical characteristics of the coastal waterbody obtained for those investigations has been used in this study. Example data are presented in Figures 2.5 to 2.11. The oceanography can be described as energetic with strong tidal currents, brief slack waters, large tidal excursions and good dispersive characteristics. Table 2.2 summarises information from the 1985 study.

A recording current meter was deployed for 30 days during the 1985 survey. This was located approximately 1000m east northeast from the harbour mouth on the then proposed outfall line (Figure 2.10). It was positioned at a height of 1.5m above the seabed. The 95%ile speed recorded at the current meter location was 0.05m/s (Figure 2.11).

	Current Speeds m/s		**Drogue Excursions**	
Tide	Flood	Ebb	Flood	Ebb
Spring	0.66	0.59	15km	15km
Neap	0.42	0.35	11km	6km

Table 2.2 - Summary depth averaged oceanographic information

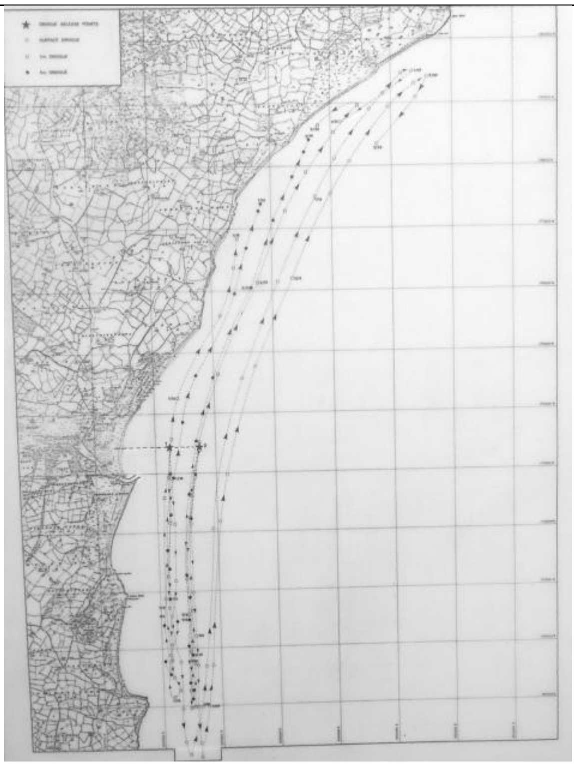


Figure 2.5 - Spring Tide Drogue Release

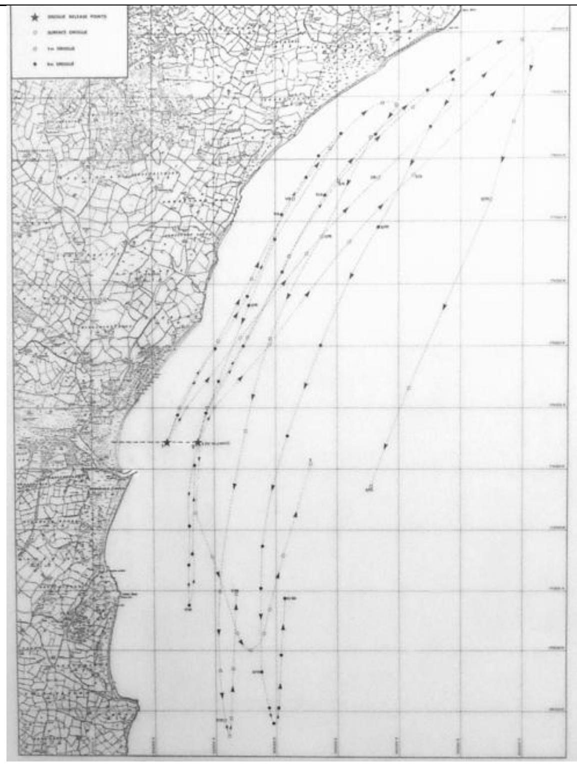


Figure 2.6 - Spring Tide Drogue Release

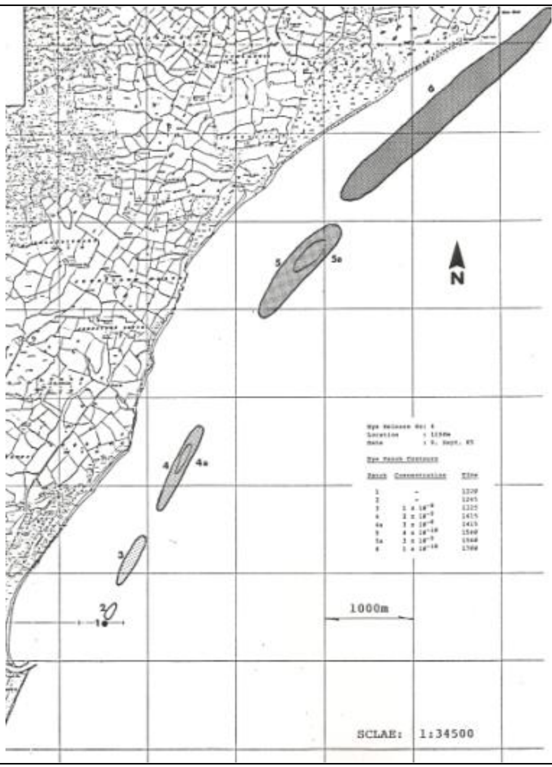


Figure 2.7 Spring Flood Tide Dye Release

Figure 2.8 - Harbour Drogue Release

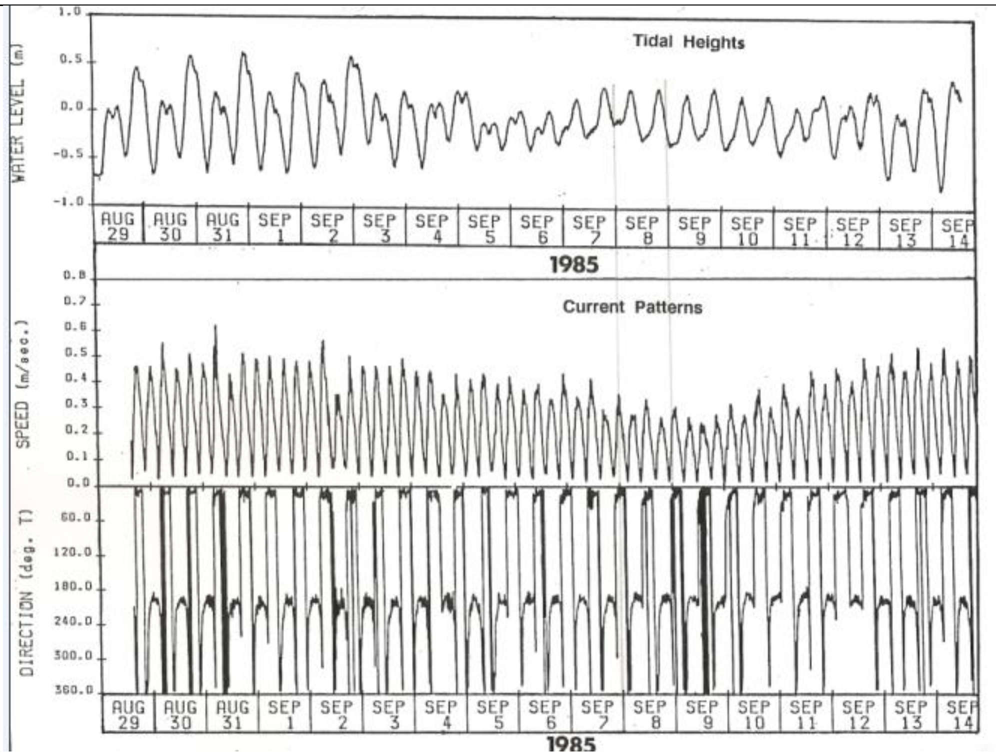


Figure 2.9 - Current meter data from previous study (ref:4)

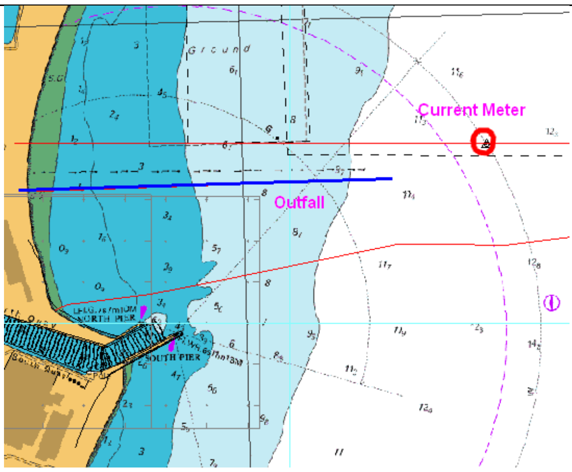


Figure 2.10 – Recording current meter location and proposed outfall line

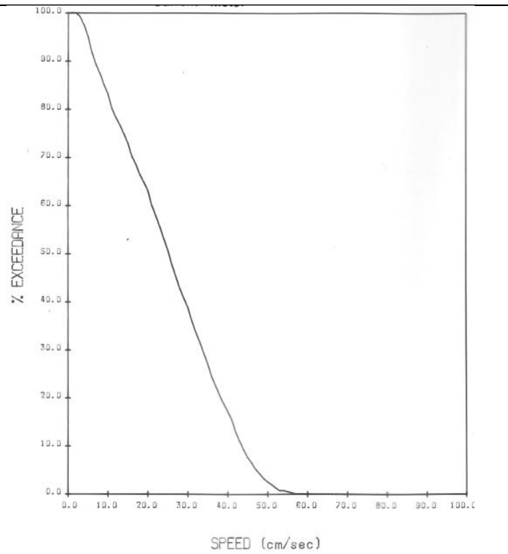


Figure 2.11 – Current speed exceedence plot

2.4 Avoca River

The Avoca river is a substantial waterbody with a primarily upland catchment of some 650m^2. From Woodenbridge to the sea the river bed profile is relatively flat with a gradient of about 1:700. Topographic data was collected as part of the overall investigations (ref:7). Figure 2.12 shows the locations of the channel profiles. The main river channel is typically rectangular and 50 to 70m wide (Figure 2.13). In the lower reaches two weir type structures control the river levels, one at the town bridge (crest level approx 0.3 m below Malin) and the other (crest level approx 0.44 below Malin) approximately 250m upstream of the N11 bridge (Figure 2.14). The weir at the town is part of the bridge structure while the one upstream is a rubble construction. The longitudinal profile in Figure 2.14 shows that the river water surface profile is influenced by tidal levels for a distance of almost 5km upstream from the harbour mouth. The tidal statistics from Table 2.1 are shown overlain on the river profile. The modelled profile is for a river flow of 3.09m^3/s. (details of the model are described in Section 4.3). Saline intrusion has not been detected in EPA sampling at Station RS10A031100 which is located approximately 2.9km from the harbour mouth. The Avoca flow characteristics based on EPA Hydrometric data system are: DWF = 0.8 m^3/s, 95%ile = 3.09 m^3/s and 50%ile = 15 m^3/s.

Figure 2.12 - River topographic section locations

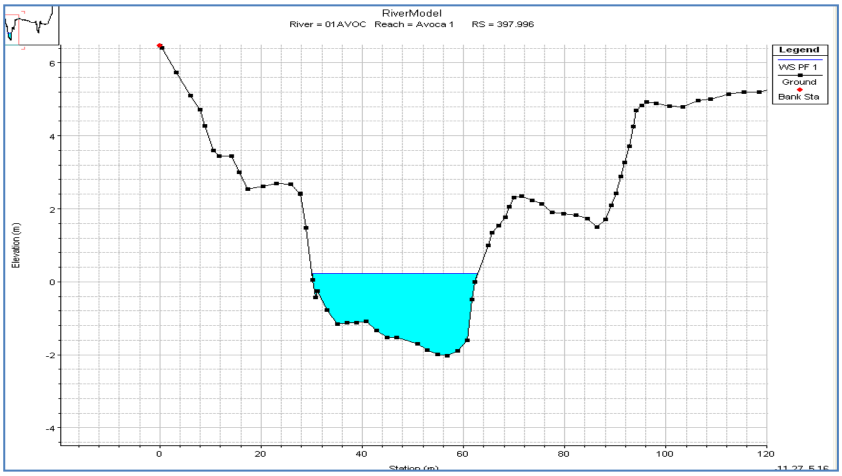


Figure 2.13 - Typical river cross-section.

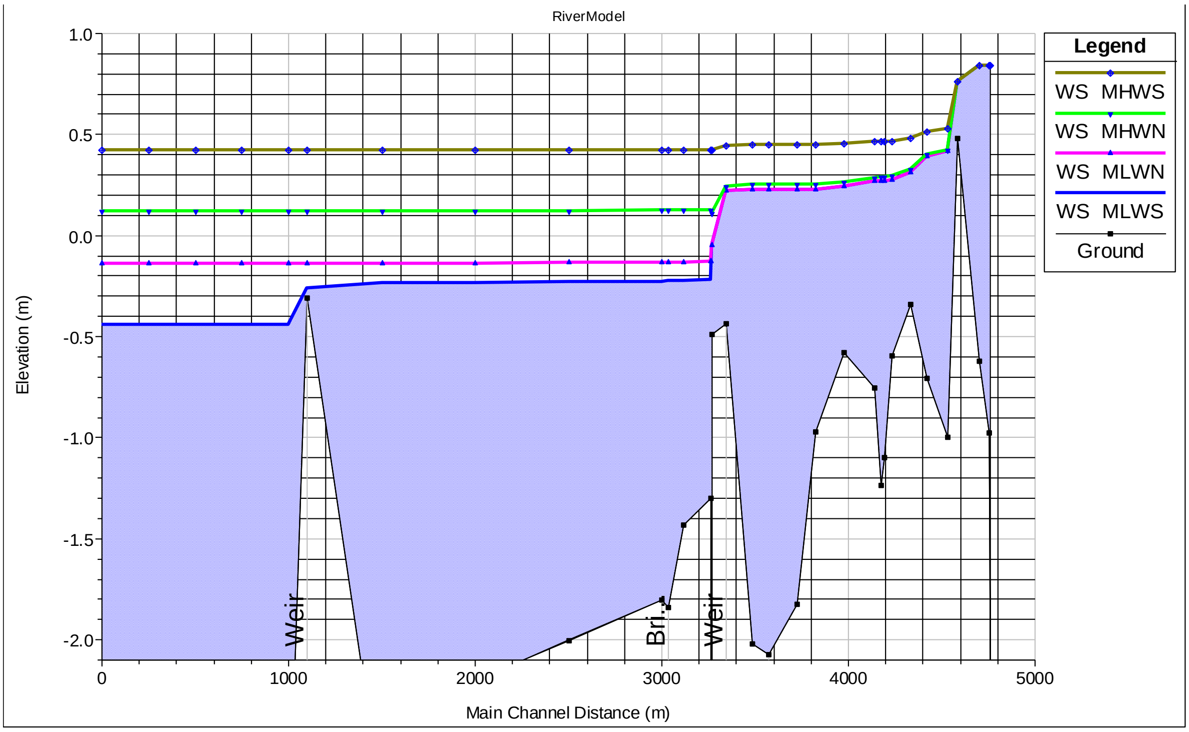


Figure 2.14 - River longitudinal section, modelled water profiles for Q=3.09m^3/s

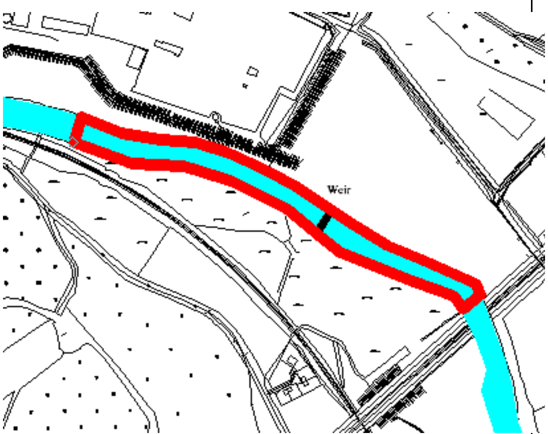
3. Design Parameters

3.1 WWTP Design Requirements

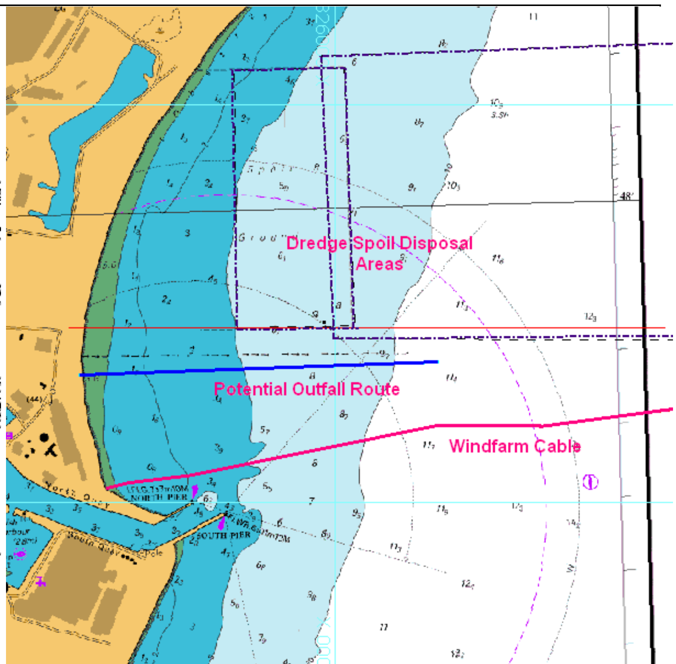
The proposed WWTP will be constructed in two phases. Phase 1 has a design population equivalent (p.e.) of 18,000p.e. while for Phase 2 it doubles to 36,000p.e. The longer term p.e. is used in this study. The associated discharge dry weather flow (DWF) is 0.101 m^3/s and the average flow is 0.127 m^3/s.

The potential WWTP outfall locations being considered in this study are indicated in Figure 3.1. The precise locations of any plant, structure or associated outfalls have yet to be decided. In the case of the upstream river outfall a potential discharge point will lie somewhere within a 500m reach. Apart from the local mixing zone the overall assimilative capacity is dependent on the river flows and not the precise location. Nutrient levels are the defining factor in determining suitability.

For the marine outfall the discharge point may be moved further offshore to provide more dilution and dispersion and therefore less treatment in the plant. There are additional constraints at this location in the form of the Arklow Bank windfarm cable and the proximity of licenced dredge spoil disposal sites. The proposed outfall route lies within the cable exclusion corridor and any works would require detailed investigation and consultation.



River Outfall Site



Marine Outfall Route and Constraints

Figure 3.1 - Potential outfall locations

3.2 WWTP Discharge Characteristics

The proposed p.e. for the plant is greater than 10,000 therefore secondary treatment is required in accordance with the UWWT regulations. None of the local waterbodies have been designated 'Sensitive' and therefore minimum design parameters for the plant are as listed in Table 3.1.

Parameters	Concentration	Minimum Percentage Reduction
BOD_5	25mg/l O_2	70-90
COD	25mg/l O_2	75
TSS	35 mg/l	90

Table 3.1 - Urban Wastewater Treatment Regulations requirements

A river discharge has the potential to impact three waterbodies while the coastal discharge will impact only one. The target water quality standards for such waterbodies are listed in Table 3.2. The primary objectives are to satisfy the 'Good Status' for river waters and the 'High Status' for coastal waters.

There are no designated bathing waters nearby (Clogga Beach is 3km south of the harbour). However for the purposes of this study the Bathing Water Quality Regulations (2008) are considered to apply to all coastal beaches immediately to the north and south of the harbour mouth. The Avoca is not a designated salmonid water.

Parameter	River Waters	Transitional	Coastal Waters
	Target	Target	Target
BOD (mg O_2/l)	High Status[1] (mean/95%ile) 1.13/2.2 Good Status[1] (mean/95%ile) 1.5/2.6	[1]4.0mg/l (95%ile)	
SS mg/l	[3]25 mg/l		
Total Ammonia (mg N/l)	High Status[1] (mean/95%ile) 0.04/ 0.09 Good Status[1] (mean/95%ile) 0.065 / 0.140		[2]0.03mg/l 95%'ile
MRP (mg P/l)	High Status[1] (mean/95%ile) 0.025 / 0.045mg/l Good Status[1] (mean/95%ile) 0.035 / 0.075	0.06mg/l (0-17psu) 0.04mg/l (34psu) median	
DIN (mg N/l)			Good Status[1] <2.6mg/l(0psu) <0.25mg/l(34.5psu) High Status[1] <0.17mg/l(34.5psu)
Bathing Waters E coli *	(Excellent Quality)[4] <500 ec/100ml (95%ile) (Good Quality)[4] <1000 ec/100ml (95%ile)	(Excellent Quality) <250 ec/100ml (95%ile) (Good Quality)[4] <500 ec/100ml (95%ile)	(Excellent Quality)[4] <250 ec/100ml (95%ile) (Good Quality)[4] <500 ec/100ml (95%ile)

Table 3.2 - Target water quality standards

[1] SI 272/2009 (Surface Waters)
[2] EPA Discussion Document (1997)
[3] *SI 273 98 (Salmonid Waters)*
[4] SI 79/2008,2006/7/EC

4. River Outfall Evaluation

4.1 Analysis Methods

The potential impacts of the proposed discharges from a river outfall were assessed using various calculations and hydraulic modelling methods. These included:

1. Mass balance calculation;
2. Travel time estimates (HEC_RAS model);
3. Coastal contaminant dispersion modelling.

Key lineal dimensional features of the river reach are outlined in Table 4.1.

Feature	Distance m
Assumed River Outfall Location	0
EPA sampling station 10A031100	650
Sigma Aldrich P0089-05	750
Transitional Waters -Avoca Estuary	1300
Coastal Waters - Irish Sea	3600
Arklow Bathing Beachs	3700
Clogga Beach (South)	6700
Brittas Beach (North)	13700

Table 4.1 - Dimensional features

Table 4.2 shows the potential dilutions available assuming complete mixing based on flow values.

River State	River Flow	Dilution
DWF	0.8 m^3/s	7.8
95%ile	3.09 m^3/s	24
50%ile	15 m^3/s	118

Table 4.2 - Dilution of WWTP discharge by river waters

4.2 Mass Balance Calculations

The objective of this calculation is to estimate discharge ELV's that will ensure that the downstream river water concentrations meet the WQ targets outlined in Table 3.2. A mass balance calculation was performed for the average effluent flow (Q_{eff} = 0.127m^3/s). The background water quality was taken from EPA site 10A031100.

Table 4.3 shows the computed downstream concentrations. The proposed ELV's have been chosen to ensure the concentrations remain well below the target levels for 'Good

Status' under Environmental Objectives (Surface Waters) Regulations 2009. The discharges will also comply with European Communities (Quality of Salmonid Waters) Regulations 1988 as both suspended solids and un-ionised ammonia levels (based on TA and ref:8) will be below the required limits.

Parameter	Background Conc. 10A031100	Proposed ELV	Downstream Conc.	Contribution from discharge	Good SWR 2009
	mg/l	*mg/l*	*mg/l*	*mg/l*	*mg/l*
BOD	1.2	10	1.55	0.35	<2.6
SS	9.4	35	10.41	1.0	-
PO4-P	0.007	1	0.046	0.039	<0.075
Total Ammonia -N	0.071	1	0.108	0.037	<0.14

Table 4.3 - Computed concentrations after full mixing

An industrial facility, Sigma Aldrich Ireland Limited, is located approximately 750m downstream of the assumed outfall location. This facility discharges treated waste waters to the Avoca under IPC licence P0089-05. In view of the relatively short distance the mass balance calculations have been repeated taking both discharges to assess the impact on the river further downstream. The predicted concentrations and the WWTP ELV's required to meet target WQ limits for this scenario are presented in Table 4.4.

Parameter	Background Conc.	Proposed ELV	Downstream Conc.	Contribution from discharge	Good SWR 2009
	mg/l	*mg/l*	*mg/l*	*mg/l*	*mg/l*
BOD	1.67	10	2.00	0.33	<2.6
SS	10.0	35	10.98	0.98	-
PO4-P	0.011	0.7	0.038	0.027	<0.075
Total Ammonia -N	0.114	0.7	0.137	0.023	<0.14

Table 4.4 - Computed concentrations after full mixing, increased background

Note: River Background from sampling pt 10A031100 and contribution from P0089-05

Note: Calculations based on river 95%ile flow (3.09m^3/s) and WWTP AvF (0.127m^3/s)

4.3 Downstream E.coli Concentration Estimates

Discharges to the river travel downstream to the sea at a rate that is dependent on the river flow. The treated wastewater initially has a high coliform count (1 x 10^6 ec/100ml). This is diluted by the river waters and as it moves downstream bacterial die-off takes place. The die-off rate is defined in terms of a T_{90}, the time for a 90% reduction in levels. The T_{90} value varies depending on the physical conditions such as water depth, sunlight, temperature and water quality. Literature indicates that the typical values range from 4-10 hours. For the purposes for this study a more conservative value of 12 hours has been adopted.

The cumulative travel times in the river for a range of flows were computed with a HEC-RAS model (ref:9). The model used river cross-section data mentioned in Section 2.4. The downstream boundary elevation was taken to be the MLWS level to produce a worst case scenario. The computed e.coli concentrations as a function of river flow are presented in Figure 4.1. The worst case occurs when river flows are about 15m^3/s and travel times are reasonably quick. At lower flows the travel time is longer and decay reduces concentrations. At the higher flows the greater volume of river water available for mixing also helps to reduce concentrations.

The predicted bacterial concentrations at the harbour mouth for two flows are presented in Table 4.5. Once the river exits the harbour mouth further dilutions are available. The coastal model (described in the next section) indicates that peak levels on the nearby beaches (Table 4.6) will be within the 'excellent' category limit (<250 ec/100ml) as defined by the Bathing Water Quality Regulations 2008 during low flow conditions and well within to the 'Good' category limit (<500 ec/100ml) during higher winter flows.

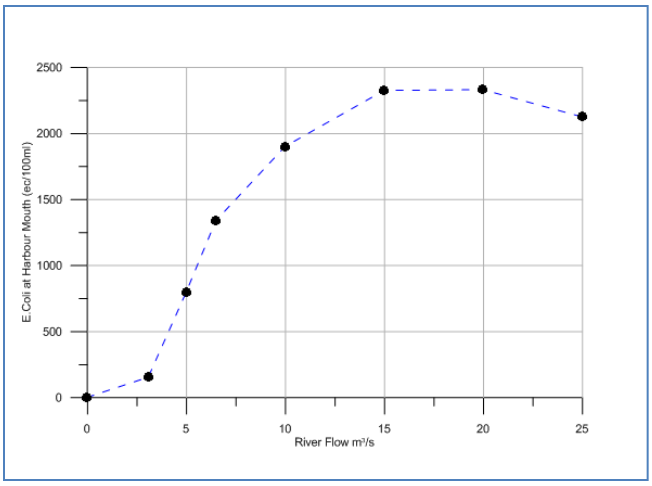


Figure 4.1 - Predicted e.coli concentration at harbour mouth vs river flow

River Flow	Travel Time from Discharge Location	E.coli Concentration in River Waters at Harbour Mouth ec/100ml
Q = 3.09 m^3/s	55 hrs	154
Q = 15.0 m^3/s	5.75hrs	2342

Table 4.5 - Predicted e.coli concentrations at harbour mouth.

	Neap Tide				Spring Tide			
	Calm		Wind		Calm		Wind	
Flow m^3/s	Beach		Beach		Beach		Beach	
	North	South	North	South	North	South	North	South
3.09	3	6	5	4	2	3	3	4
15.0	220	438	280	260	91	160	197	280

Table 4.6 - Predicted e.coli concentrations on bathing beaches ec/100ml

5. Marine Outfall Evaluation

5.1 Analysis Methods

The potential impacts of the proposed discharges on the marine waters were assessed using various calculations and hydraulic modelling methods. These included:

1. Initial Dilution Simulations;
2. Water Circulation Modelling;
3. Contaminant Dispersion Modelling.

For method 1 a jet type model was used to estimate near-field dilutions at the discharge locations. Method 2 uses bathymetry and tides to simulate hydrodynamic patterns in the wider far-field area. Method 3 uses contaminant simulations, driven by hydrodynamics of method 2, to evaluate the location-specific impacts of discharges within the mid and far-field areas.

5.2 Discharge Characteristics

The WWTP will provide secondary treatment as a minimum under Urban Wastewater Treatment Regulations 2001. Table 5.1 lists water quality standards that are achievable with a modern plant.

Parameter	Abbreviation	Design Value
Population Equivalent	PE	36000 pe
Dry Weather Flow	DWF	0.101 m^3/s
Average Daily Flow	ADF	0.127 m^3/s
Discharge Standards		
Biochemical Oxygen Demand	BOD	25mg/l
Suspended Solids	SS	35mg/l
Total Ammonia (as N)	TA	10mg/l
Total Oxidised Nitrogen (as N)	TON	35mg/l
Dissolved Inorganic Nitrogen (as N)	DIN	45mg/l
E.Coli	EC	1 x 10^6 ec/100ml
E.Coli Decay Time	T_{90}	12 hours

Table 5.1 - Discharge standards used in the outfall assessment

Target water quality values for coastal waters on the basis of various regulations were outlined in Table 3.2. Only three of these are of particular significance for the marine outfall configurations being examined. These are e.coli, total ammonia and DIN. The relatively high levels of bacterial contamination in the treated effluent mean that this is usually the most critical parameter in outfall evaluation when bathing areas are located nearby.

5.3 Potential Outfall Locations

Three offshore discharge locations were examined, each moving further to the east from the shoreline. The locations are shown in Figure 5.1. A discharge from the harbour mouth was also considered to facilitate evaluation of the impact of a river outfall. Summary information for each discharge location is presented in Table 5.2. The outfall length is measured from the low water mark.

Outfall Location	Pipe Length	ING Easting (m)	ING Northing (m)	Bed Level (m) CD	Depth (m) OD Malin
1	400	325770	173330	5	6.1
2	650	326020	173340	9.5	10.6
3	900	326270	173350	11.0	12.1
4	Harbour Mouth	325698	173000	4.5	5.6

Table 5.2 - Potential outfall locations

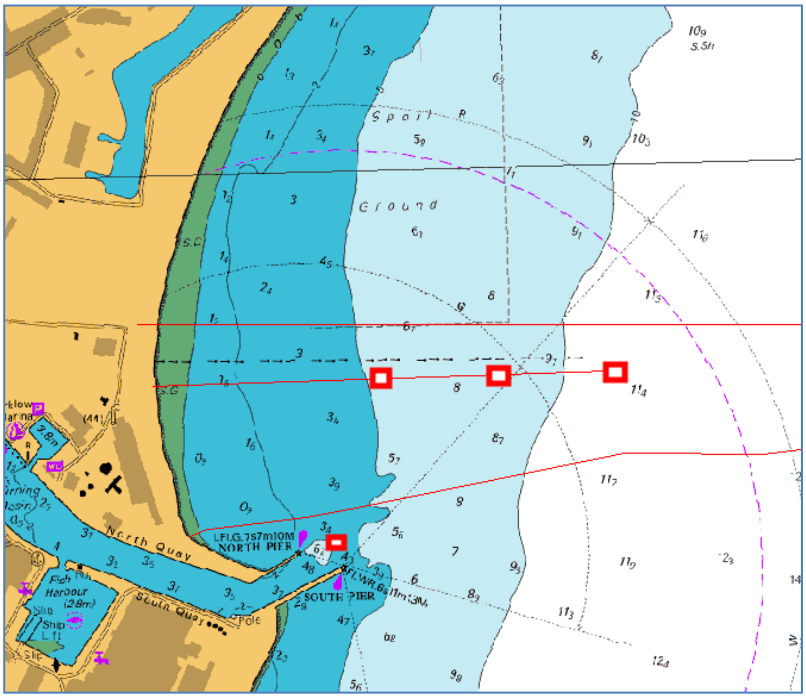


Figure 5.1 - Potential outfall locations

5.4 Initial Dilutions at Outfall Discharge Locations

Initial dilution calculations were carried out for the MLWS tide level conditions. The discharges, located near the seabed, come to the surface in a plume at a rate dependant on the buoyancy forces arising from temperature and salinity differences between the effluent and ambient waters. Estimates of initial dilutions were made with the IJP model (ref:10). Calculations were made for a 6-port diffuser configuration and a range of current speeds. The speed data is based on the current meter exceedance profile shown in Figure 2.10. Table 5.3 presents dilutions and associate displacement of the surface plume centroid from the discharge location in the direction of the current. Diffuser ports are 10m apart and plumes from each of the six remain separate in the early stages of dilution.

For the simulated configuration both the 650m and the 900m long outfalls would meet the 95%ile initial dilution target of 50 considered necessary to eliminate any slicks or odours (ref:11). Even at slack water dilution for the 900m outfall is above the target level. The initial dilution available for the 400m outfall is very much reduced due to the shallow waters at the discharge point.

	400m Outfall		650m Outfall		900m Outfall	
Tide Level = MLWS	Dilution	Displ	Dilution	Displ	Dilution	Displ
Current Speed = 0m/s	21	0	40	0	55	0
Current Speed = 0.05m/s (95%ile)	33	5	67	7	98	8
Current Speed = 0.26m/s (50%ile)	87	16	208	27	322	40
Current Speed = 0.43m/s (10%ile)	115	25	282	50	450	70

Table 5.3 - Predicted initial dilutions and displacements (ADF = 127 l/s, 6 ports, port diameter = 0.16m, , port spacing = 10m).

For comparative purposes the dilution estimates for the 95%ile tidal current have been used to calculate the near-field concentration of the parameters BOD, SS, TA, DIN and EC. Background concentrations have been taken from EPA data for Southern Irish Sea HA10 (2007-2009). The results are presented in Tables 5.4-5.7 and show that in almost all cases a relatively small amount of additional mid-field dilution (<10 fold) will bring these parameters below target WQ levels. The exception is e.coli for which results show that additional dilutions of up to 118 will be required.

Parameter	Treated Eff. Conc	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.68	4	-
SS (mg/l)	35	2	2.48	2	1.5
DIN (mg/l N)	45	0.157	1.476	01.37	8.1
T Amm (mg/l N)	10	0.02	0.314	0.03	10.45
EC fc/100ml	1×10^6	20	29431	250	118

Table 5.4 - 400m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 33 (95%ile current)

Parameter	Treated Eff. Conc	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.34	4	-
SS (mg/l)	35	2	2.48	2	-
DIN (mg/l N)	45	0.157	0.81	0.17	4.2
T Amm (mg/l N)	10	0.02	0.167	0.03	5.2
EC fc/100ml	1×10^6	20	14726	250	59

Table 5.5 - 650m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 67 (95%ile current)

Parameter	Treated Eff. Conc.	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.23	4	-
SS (mg/l)	35	2	2.33	2	1.2
DIN (mg/l N)	45	0.157	0.61	0.17	3.6
T Amm (mg/l N)	10	0.02	0.121	0.03	4.0
EC fc/100ml	1×10^6	20	10121	250	40

Table 5.6 -900m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 98 (95%ile current)

Parameter	Treated Eff. Conc	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.26	4	-
SS (mg/l)	35	2	2.38	2	1.2
DIN (mg/l N)	45	0.157	0.667	0.17	4.0
T Amm (mg/l N)	10	0.02	0.133	0.03	4.5
EC fc/100ml	1×10^6	20	11383	250	46

Table 5.7 - 400m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 87 (50%ile current)

Parameter	Treated Eff. Conc	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.11	4	-
SS (mg/l)	35	2	2.16	2	1.1
DIN (mg/l N)	45	0.157	0.372	0.17	2.2
T Amm (mg/l N)	10	0.02	0.068	0.03	2.3
EC fc/100ml	1×10^6	20	4805	250	19

Table 5.8 - 650m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 208 (50%ile current)

Parameter	Treated Eff. Conc.	Background Conc.	Conc After I.D.	Target Level	Additional Far Field Dilution Reqd
BOD (mg/l O2)	25	2	2.07	4	-
SS (mg/l)	35	2	2.1	2	-
DIN (mg/l N)	45	0.157	0.296	0.17	2
T Amm(mg/l N)	10	0.02	0.051	0.03	2
EC fc/100ml	1×10^6	20	3116	250	12

Table 5.9 - 900m Outfall, Eff Q = 0.127m^3/s, Initial Dilution = 322 (95%ile current)

5.5 Water Circulation Modelling

Tidal circulation in the coastal waters off Arklow was investigated with a 2-dimensional numerical model M2D (ref: 12). The model is a general-purpose modelling package for simulating flow and transport in surface water systems. The configuration used for this study is suited to mid and far-field simulations, i.e. away from the immediate discharge point. The model has been used in various formats for earlier studies on the Arklow outfall (ref:5,6).

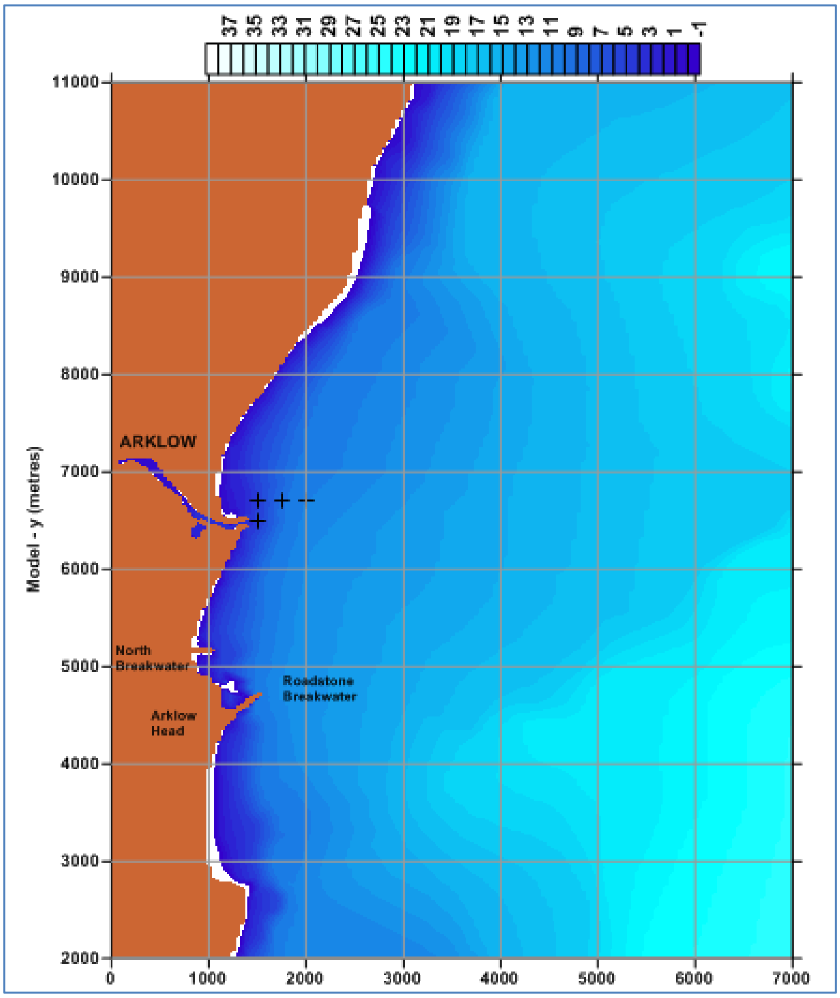


Figure 5.2 - Arklow model extents and bathymetry (chart datum)

The circulation model employed a 50 x 25m rectangular grid centred on Arklow. Bathymetry was taken from Admiralty Chart No. 1787 (Figure 2.1) mapped onto the spatial grid. The model was used to simulate typical conditions using spring and neap tidal ranges as outlined in Table 2.1. The model was calibrated with tidal elevation, current meter and drogue and dye track data (ref:4-6). Initial runs with typical coefficient settings were found to reproduce the observed tidal elevations to an acceptable level. Simulated drogue tracks closely resembled measured data (Figures 5.3, 5.4).

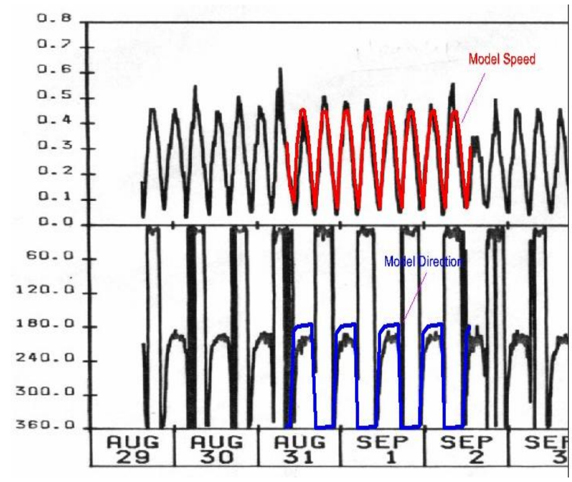


Figure 5.3 - Comparison of modelled and measured currents.

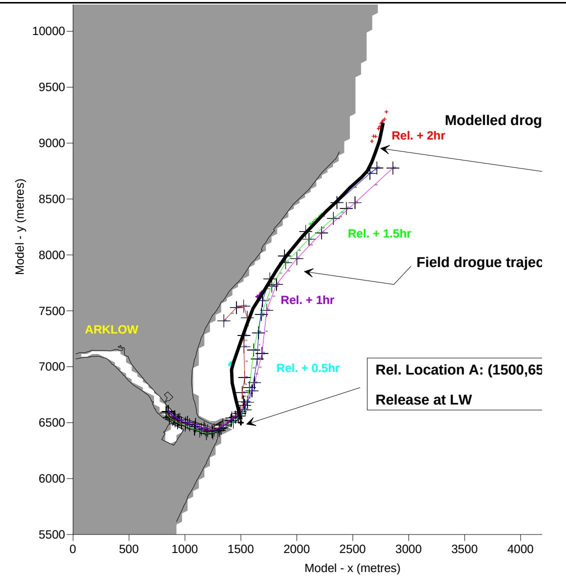


Figure 5.4 - Comparison of modelled and measured drogue trajectories

5.6 Contaminant Dispersion Simulations

The contaminant dispersion module LAG (ref:13) was used to simulate far-field dispersion. In this module the effluent stream is simulated as a continuous stream of particles. These particles are advected and dispersed through the model domain and then used to calculate contaminant concentrations at different horizontal locations and at different stages of the tide. Particle positions are tracked at 1m resolution in the model domain. Outputs are in the form of contour plots of parameter concentration as shown in Figure 5.5 or as time series at a point as shown in Figure 5.6.

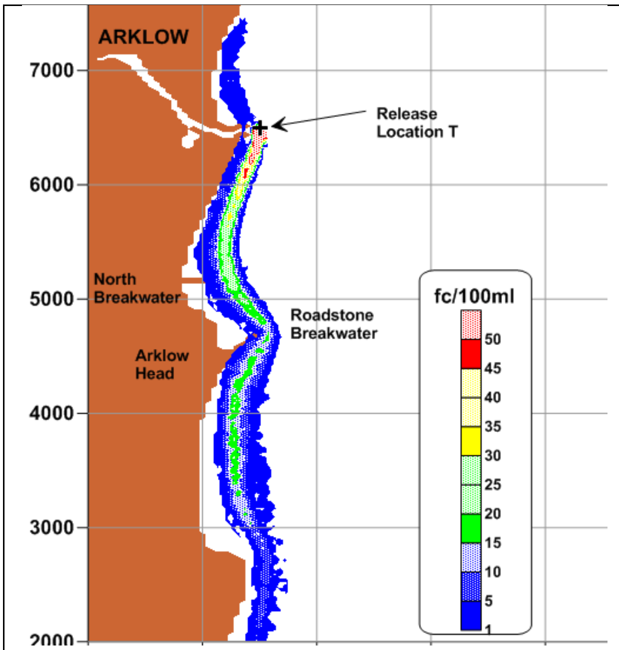


Figure 5.5 - Model Concentration Plot

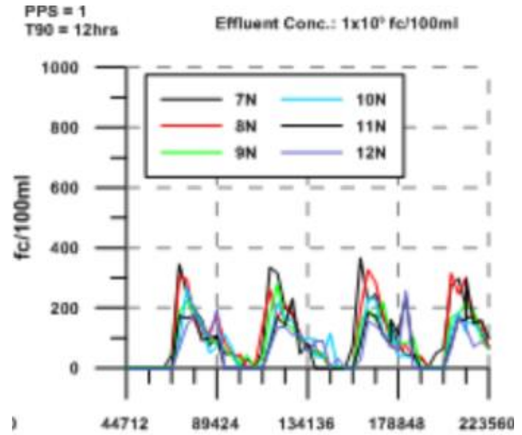


Figure 5.6 - Model Time Series Plot

While the concentration and time series plots provide a visual appreciation of plume dynamics a more quantitative method is required to allow comparison and evaluation of the outfall options and to assess the likely impact of the discharges on bathing waters. For this reason the shoreline area was divided up into a series of sampling strips as shown in Figure 5.7. Each of the strips is 100 wide and extends 200 m from the shoreline. There are 17 strips to the north of the harbour and 18 to the south. During the modelling process the highest average concentration in any model cell (50m x 25m) in each sampling strip is extracted at each time step and tabulated.

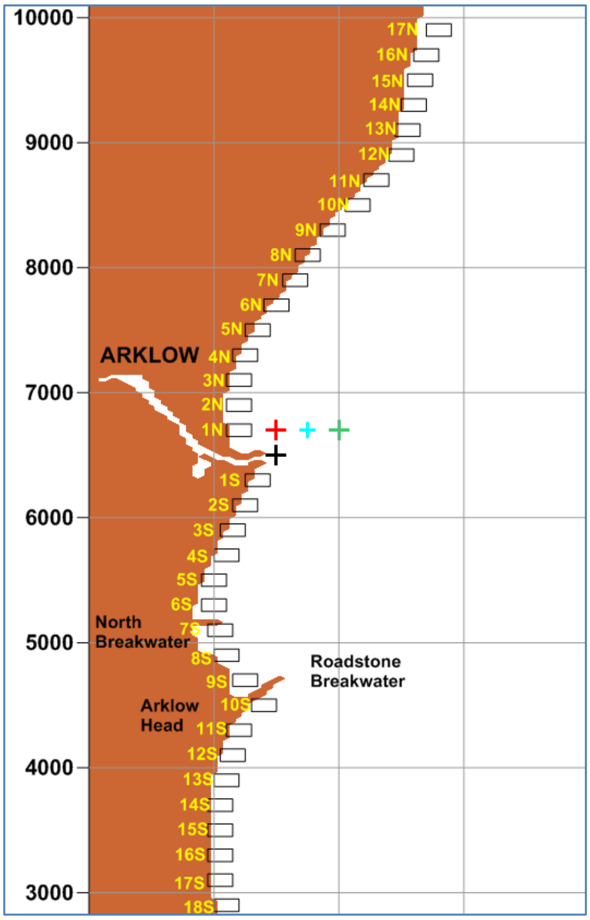


Figure 5.7 - Model sampling strips for effluent concentration estimates

__Simulation of E.Coli Concentrations__

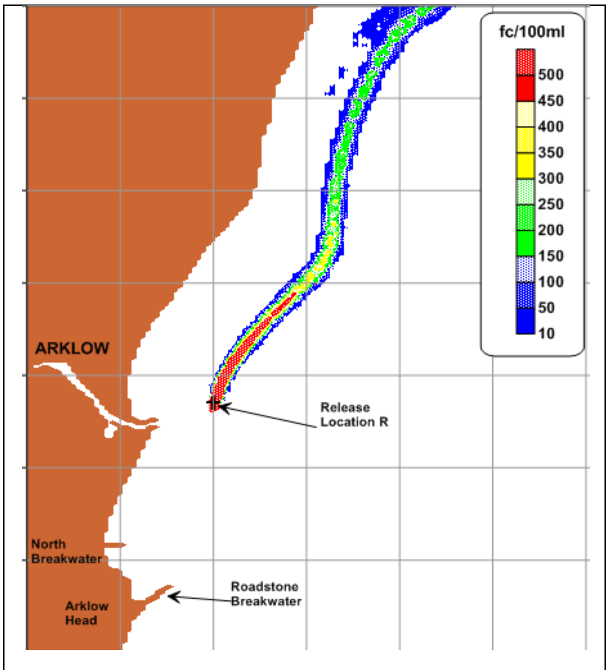
Simulations of e.coli dispersion were conducted from each of the three outfalls. Model runs were conducted for both neap and spring tides for calm and windy conditions. The effect of wind was examined as a global parameter increase in contaminant diffusion rates applied to calm spring and neap flow fields. The effective wind speed was taken to be 7.5m/s.

Model results in the form of coliform concentration contours for the 900m outfall option are shown in Figures 5.8 to 5.11. The maximum concentration value at each of the sampling location, to the north and south of the harbour mouth, was extracted from the timeseries plots and are summarised in Table 5.10. Elevated shoreline concentrations are predicted from the 400m outfall during both calm and windy conditions. The 650m outfall also produced high levels during windy conditions while the models indicate that the shoreline levels arising from the 900m outfall remain below the target 'Excellent' category limit of 250 ec/100ml.

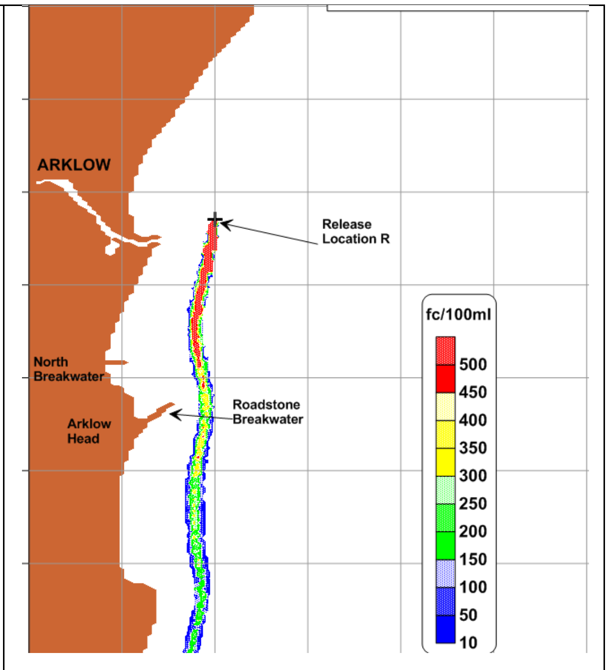
Analysis of the local wind climate conducted for a previous outfall study (ref:5) showed that during the summer months winds with an onshore component occur for approximately 30% of the time. Thus both the 400m and 650m outfalls would require additional disinfection if the discharges are to comply with the bathing water regulations.

	Neap Tide				Spring Tide			
	Calm		Windy		Calm		Windy	
Outfall Length	North Beach	South Beach	North Beach	South Beach	North Beach	South Beach	North Beach	South Beach
400	206	324	560	402	171	277	493	429
650	15	16	233	287	3	37	274	257
900	0	12	194	179	2	38	140	239

Table 5.10 - Coliform dispersion simulations – Maximum averages in sampling cells.

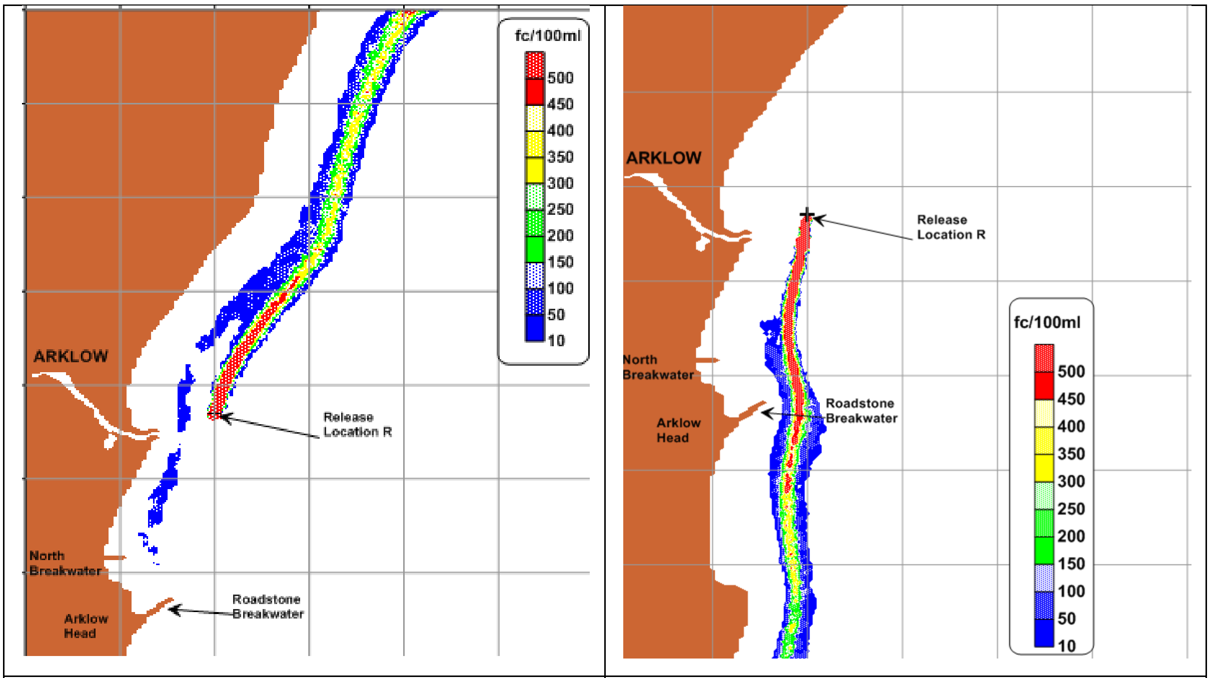


Spring Flood



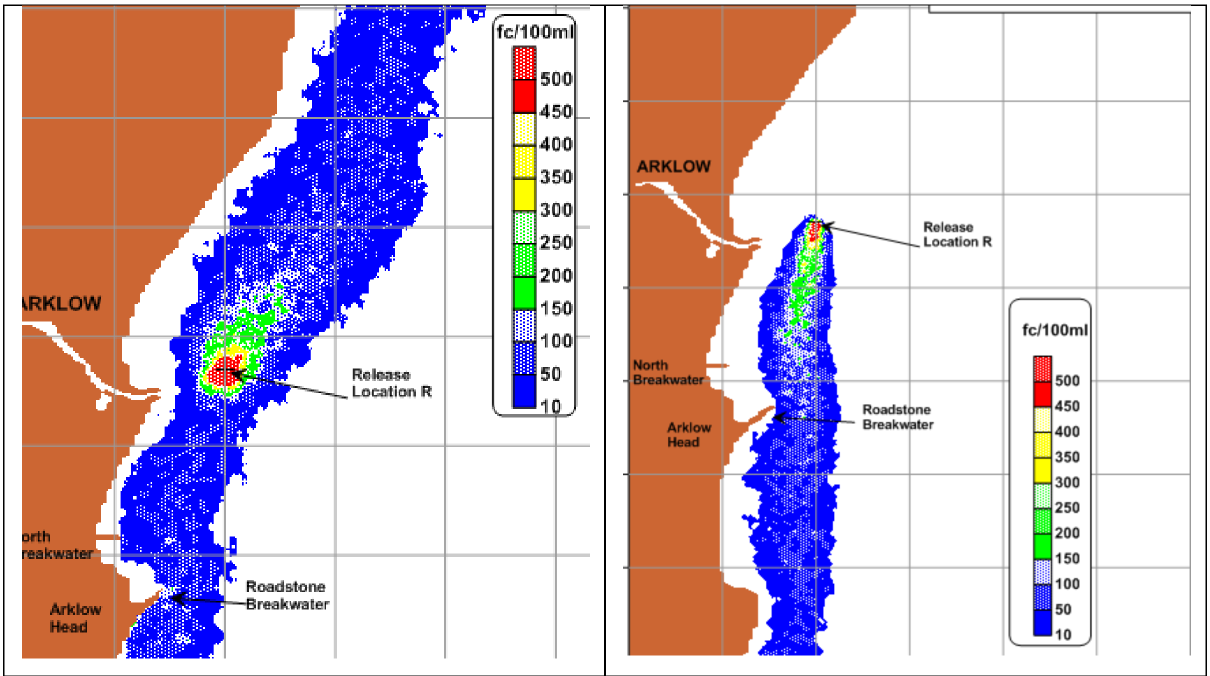
Spring Ebb

Figure 5.8a - Simulated e.coli concentrations for 900m outfall during Spring & Neap tides and calm conditions



Neap Flood | Neap Ebb

Figure 5.8b - Simulated e coli concentrations for 900m outfall during Spring & Neap tides and calm conditions



Spring Flood | Spring Ebb

Figure 5.9a - Simulated e coli concentrations for 900m outfall during Spring & Neap tides and windy conditions.

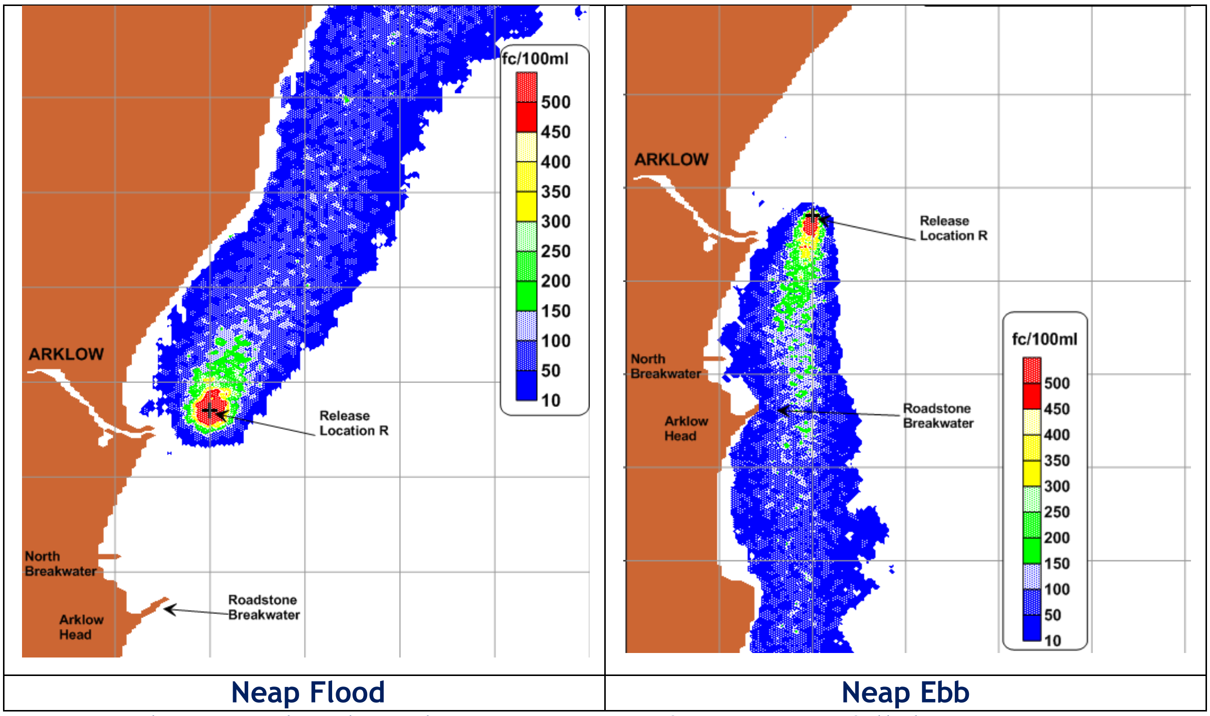


Figure 5.9b - Simulated e coli concentrations for 900m outfall during Spring & Neap tides and windy conditions.

Biochemical Oxygen Demand

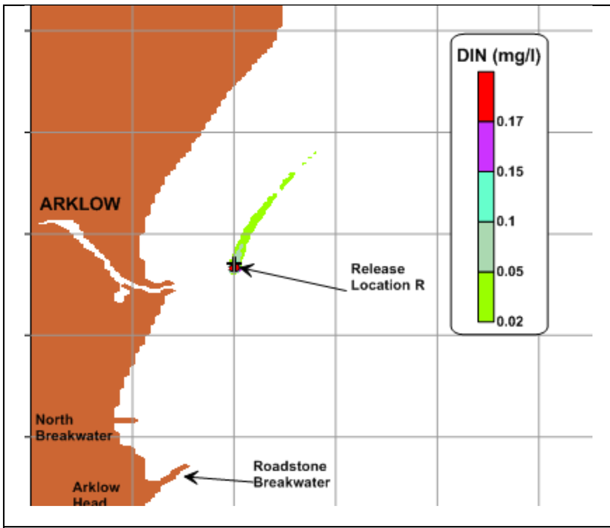
Initial dilution calculations, Tables 5.4 to 5.6, show that concentrations of this parameter will be below the target water quality level of 4mg/l as soon as the plume surfaces above the diffuser point.

Dissolved Inorganic Nitrogen

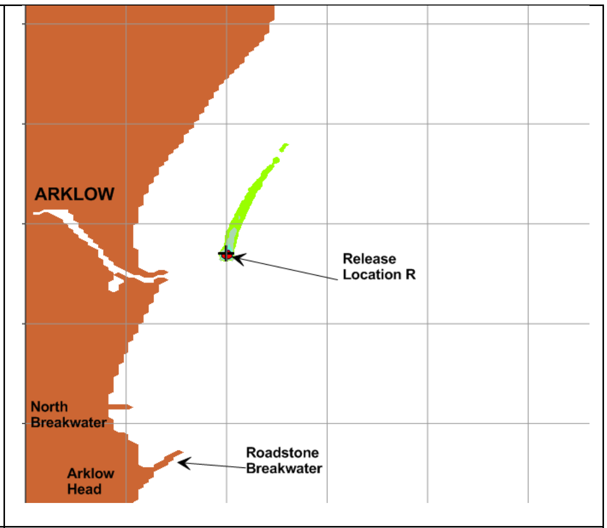
Calculations show that concentrations of this parameter will be close to but above the target water quality level of 0.17mg/l N (High Status) following initial dilution. A further mid-field/far-field dilution of between 4 and 8 will be required. Simulations with the coastal dispersion model, presented in Figure 5.10, show that the additional dilution will be achieved quickly and within about 100 m of the diffuser.

Total Ammonia

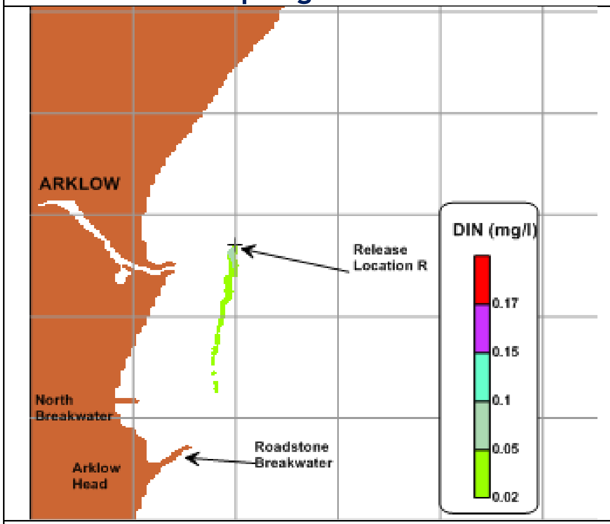
Calculations show that concentrations of TA will be close to but above the target quality level of 0.03mg/l N (EPA) following initial dilution. A further mid-field/far-field dilution of between 4 and 11 will be required. Simulations with the coastal dispersion model, presented in Figure 5.11, show that the additional dilution will be achieved quickly and within about 100 m of the diffuser.



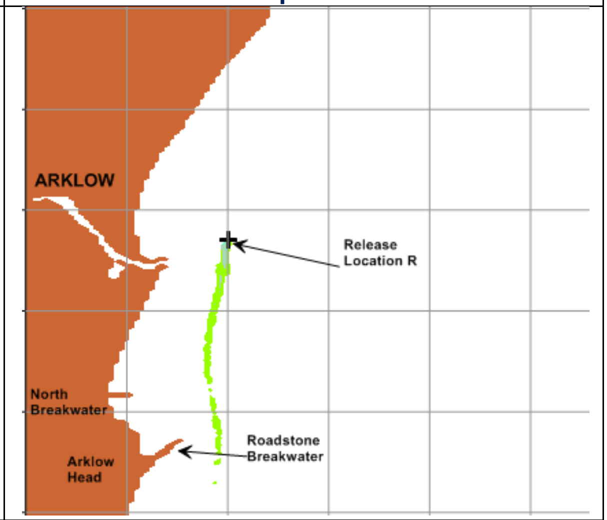
Spring Flood



Neap Flood

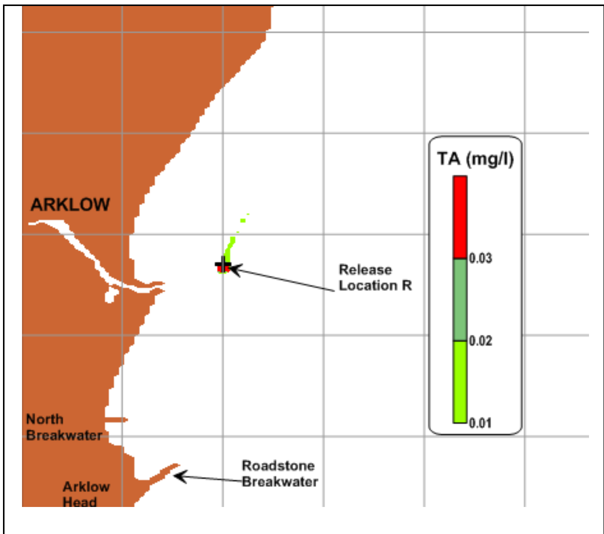


Spring Ebb

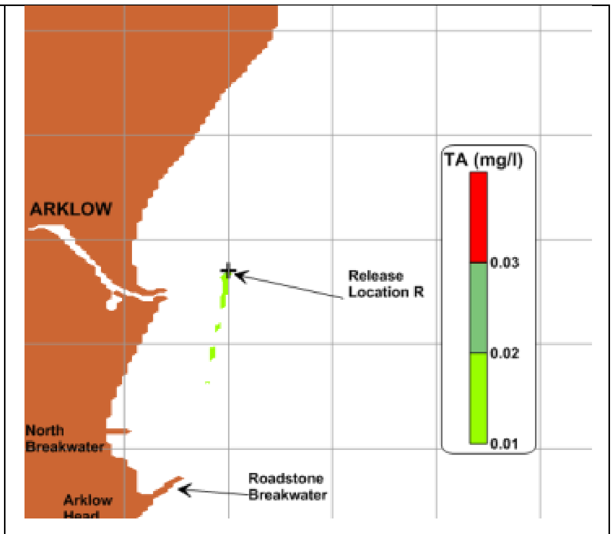


Neap Ebb

Table 5.10 - Predicted peak DIN concentrations (above background)



Spring Flood



Spring Ebb

Table 5.11 - Predicted peak TA concentrations (above background)

5.7 Summary of Marine Results

Initial dilution calculations have shown that bacterial concentrations are the critical parameter for the marine outfall evaluation as all other water quality parameters will be close to or below target levels very soon after discharge.

All three of the locations examined will provide sufficient initial dilution (from plume exiting diffuser to time it surfaces) to reduce nutrient concentrations to close to target levels. Mid-field dilution then ensures that these targets are met within 100m of the discharge point.

E.coli bacteria are present in the treated water at much higher concentrations. The models show that only the 900m outfall will ensure compliance with the bathing water 'Excellent' category during calm and windy conditions. Both of the other outfall options (400m & 650m) would require the provision of disinfection to meet this target.

6. Conclusions

An assessment of the impact of waste water discharges to the Avoca river and the Arklow coastal waters was conducted with the aid of numerical models. The assessment was conducted for a PE of 36000 with an average daily flow of $0.127m^3/s$. The analysis has allowed conclusions to be made regarding the proposed discharges and the level of treatment required in the WWTP to ensure compliance with relevant regulations.

Assessment of the river outfall was made both on the basis of EPA background water quality data and also taking discharges from the Sigma Aldrich plant, 750m downstream of the assumed outfall position, into consideration. The proposed range of ELV's are summarised in Table 6.1.

Analysis of the marine outfall options has shown that the coastal water depths and current speeds are sufficient to ensure rapid dilution of all contaminants other than e.coli bacteria. Models indicate that only the 900m outfall will ensure compliance with the 'Excellent' category of Bathing Water Quality Regulations 2008. The proposed ELV's are summarised in Table 6.1.

These findings are provisional and the analyses and proposed ELV's should to be formally discussed with the EPA prior to making a final decision on a preferred WWTP location.

Parameter	River Outfall	900m Marine Outfall
Biochemical Oxygen Demand	10mg/l	25mg/l
Suspended Solids	35mg/l	35mg/l
Total Ammonia-N	0.7 to 1mg/l	10mg/l
TON-N	35mg/l	35mg/l
PO4-P	0.7 to 1mg/l	
E.coli	1×10^6 ec/100ml	1×10^6 ec/100ml

Table 6.1 - Proposed WWTP discharge ELV's

References

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   Chart No: 1787, Carnsore Point to Wicklow Head.
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10. Hunter 1997, User Manual for Outfall Modelling Package. Curtin University Aus.

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