Chapter 3 The 'Old Head_2' Model

The previous chapter was concerned with the datasets which were used to construct the models used in this Environmental Impact Assessment. This chapter describes the Old Head_2 (OH_2) model which was used to simulate the bacteria, *Norovirus* and the Nitrogen Cascade for the different cases considered in this report.

All of our work makes use of the well-known MIKE 21 modelling system supplied under licence by the Danish Hydraulics Institute (DHI)[11]. DHI provides very extensive documentation on this system and is not included in this report.

3.1 OH_2 model layout

The development of every numerical model involves a compromise between a high resolution grid[12] which resolves the flow in great detail and the time it takes for a computer to calculate the results[13]. The model run time is a function of the number of grid points in a model[14] and the timestep. Generally if the grid spacing is halved the model runtime increases by a factor of 8. Given that the run time for models such as the OH_2 could be in the order of days, and not hours, the issue of resolution and run time is always of concern.

Nested grids are the means by which this problem can be overcome. A nested grid implies that different areas of the model are resolved with different grid spacing. Areas that are of great importance to the study may be resolved with a high resolution while the area surrounding it may be resolved with a lower resolution. The higher resolution grid must sit inside (hence the 'nested' term)

[11] http://www.dhigroup.com/

[12] We use the ULTIMATE high-accuracy finite difference scheme in MIKE 21.

[13] The size of the generated result files is also a concern. High resolution grids generate larger result files than those with a lower resolution. Files larger than 4GB are quite problematic for any personal computer today.

[14] Determined by the extent of the model and the grid spacing

the coarser grid. At the boundary the water level and fluxes are passed from one grid to the next so that a single unified model is developed. For MIKE 21 the nested grid must be exactly 3 times smaller than the coarser grid. A 30m grid can be nested within a 90m grid but not a 100m grid. The 90m grid may then be nested within a 270m grid. MIKE 21 allows up to 9 grids to be successively nested within each other. All the models developed as part of this study use nested grids.

The layout of the OH_2 model is presented in Fig. 3.1. The model consists of three separate nested grids each with a different spatial resolution. The outer grid has a 90m resolution and covers from the Old Head of Kinsale to Robert's Cove. The second grid has a 30m resolution and covers from Robert's Cove to the Waterworks weir. A third grid of 10m resolution resolves the flow through the narrow Belvelly Channel in the inner harbour.

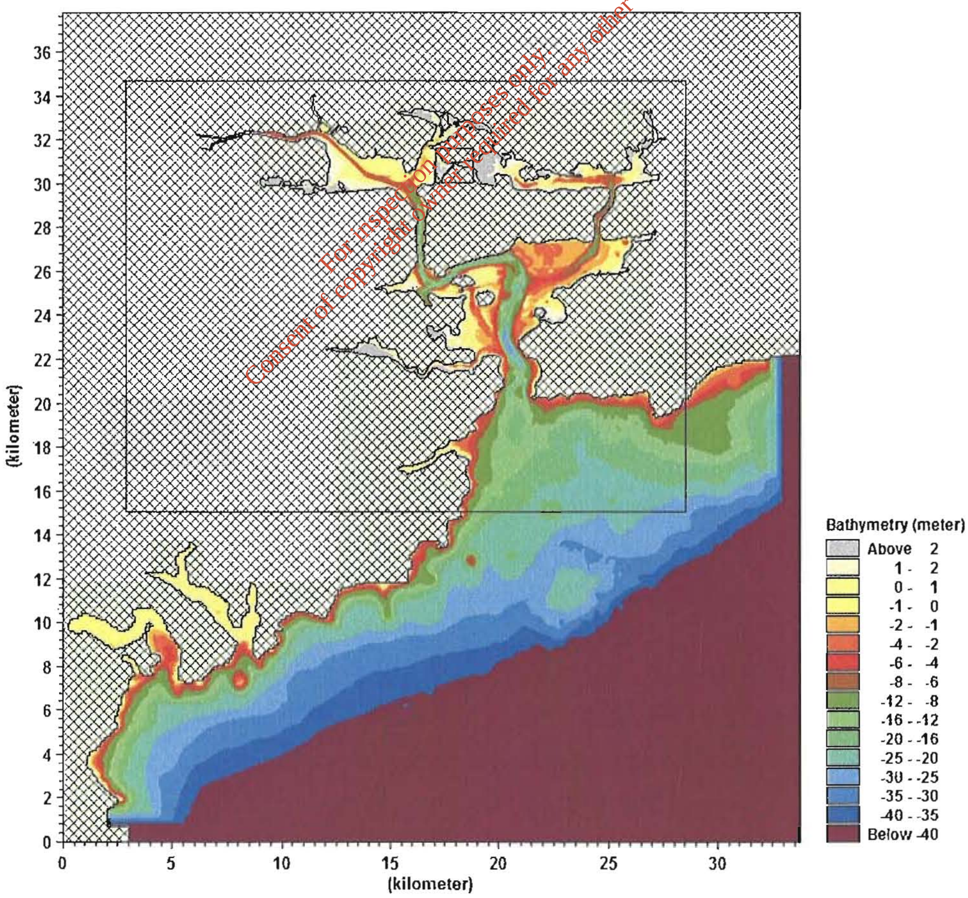

Fig. 3.1 Layout of the OH_2 model

The extent of the 90m grid is determined by the location of the 3 grid points from the CS3 model from which the boundary conditions are obtained.

Modelling a large area also ensures that discharges from the outfalls are not lost through the boundary. If the boundary had been located at Roches Point, as it is for the RP_2 model, particles released from the Carrigaline/Crosshaven outfall will be carried past the boundary at Roches Point on the ebb tide and taken out of the model. On the ensuing flood tide the model will underestimate the concentrations in the harbour as the particles which should be transported from outside Roches Point back into the harbour have been lost. This may lead to an unacceptable error in the results. Consequently, the RP_2 model is of questionable accuracy in simulating the release of bacteria or viruses from the Carrigaline/Crosshaven outfall. This problem is overcome by using the OH_2 model.

We have resolved the harbour and area immediately outside Roches Point with a 30m grid. This resolution is more than sufficient to resolve the flow through the East and West passage, Lough Mahon and the North Channel behind Great Island.

3.2 Boundary Conditions – CS3 model

The boundary conditions of the OH_2 model were provided by the output from the CS3 numerical model, maintained by the Proudman Laboratory in the UK which covers part of the North West Atlantic Shelf. In other words, the OH_2 model is itself embedded in an even larger model.

Boundary conditions for numerical models such as the OH_2 model are typically provided by recorded measurements of water levels[15]. Such an approach was too expensive for this project[16]. In addition there is a substantial risk of the

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[15] Coupled in some cases with recorded velocities.

[16] Deploying gauges in the open sea, such as near the Old Head of Kinsale, is far more expensive than doing so within estuaries. At the start of the project a quote was obtained to

gauges being lost when deployed in the open sea. We have used the Proudman data as our boundary conditions for the OH_2 model in a direct and simple manner.

There are limitations in our approach:

1. Any errors in the CS3 model are propagated into the OH model.
2. The resolution of the CS3 model is 12km. Therefore the data derived from it cannot contain detail at scales less than 24km (Nyquist sampling theorem).
3. No downscaling, or intermediate grid, has been used to transfer data from the 12km grid of the CS3 to the 90m outer grid of the OH_2 model. To overcome this particular problem would have required additional data for points further out in the Celtic sea and the development of a much larger OH_2 model.

These limitations, however, do not lead to unrealistic boundary conditions. As we will see in the next section, the output from the model driven with the Proudman data, when adjusted slightly, is capable of reproducing the observed tides in Cork Harbour to within an error of 20cm.

The annual hindcasts for 1992 and 2004 of tide-plus-surge, and tide-only levels from the three grid points closest to the mouth of Cork Harbour were purchased from the Proudman Laboratory for the previous *Norovirus* study. The tide-plus-surge data (1 hour frequency) were interpolated between the data points, and extrapolated between the data points and the land, to form a profile series[17]. The two profile series describe the variation in water level and fluxes along the two open boundaries of the model and drive the hydrodynamics. See Fig. 3.2.

deploy 3 gauges measuring water levels and velocities some distance outside the harbour mouth. The cost was far in excess of the proposed budget for the project.

[17] A profile series contains data, which describes the variation in time of a variable along a line in space.

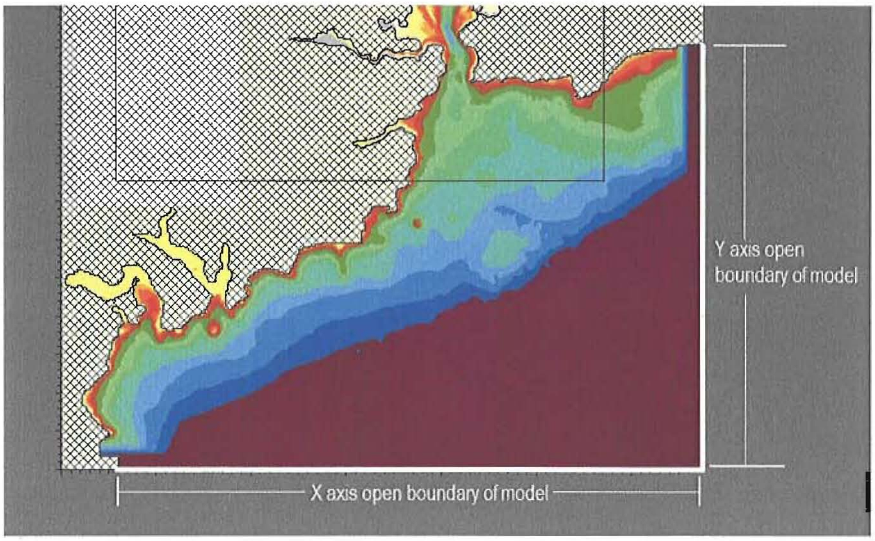

Fig. 3.2 Extent and location of the open boundaries of the OH_2 model

3.3 Calibration of the OH_2 model

The OH_2 model has been validated using parameters taken from the RP_2 model. The RP_2 model has been calibrated and validated using recorded water levels, current speeds and direction from the 1992 data survey. This is described in Appendix A of this report. The validation of the OH_2 model is presented in the following section.

3.4 Validation of the OH model

The validation of the OH_2 model is presented in the following plots. The spring tide water level validation for the gauge at Cobh is presented in Fig. 3.3. We can see from the plot that the difference between the modelled and the measured is less than 20cm with the exception of the first two high tides in the plot where it is less than 25cm. We can also see that the gauge at Cobh has a number of erroneous readings at three of the recorded high tides. The gauge has topped-out for approximately 3 hours on each of these 3 occasions.

The spring tide water level validation for the gauge at Tivoli is presented in Fig. 3.4. We can see from the plot that the difference between the modelled and the measured within 25cm except for two of the tides[18].

From this we can conclude that the OH_2 model is capable of reproducing the tides in Cork Harbour to within a satisfactory level.

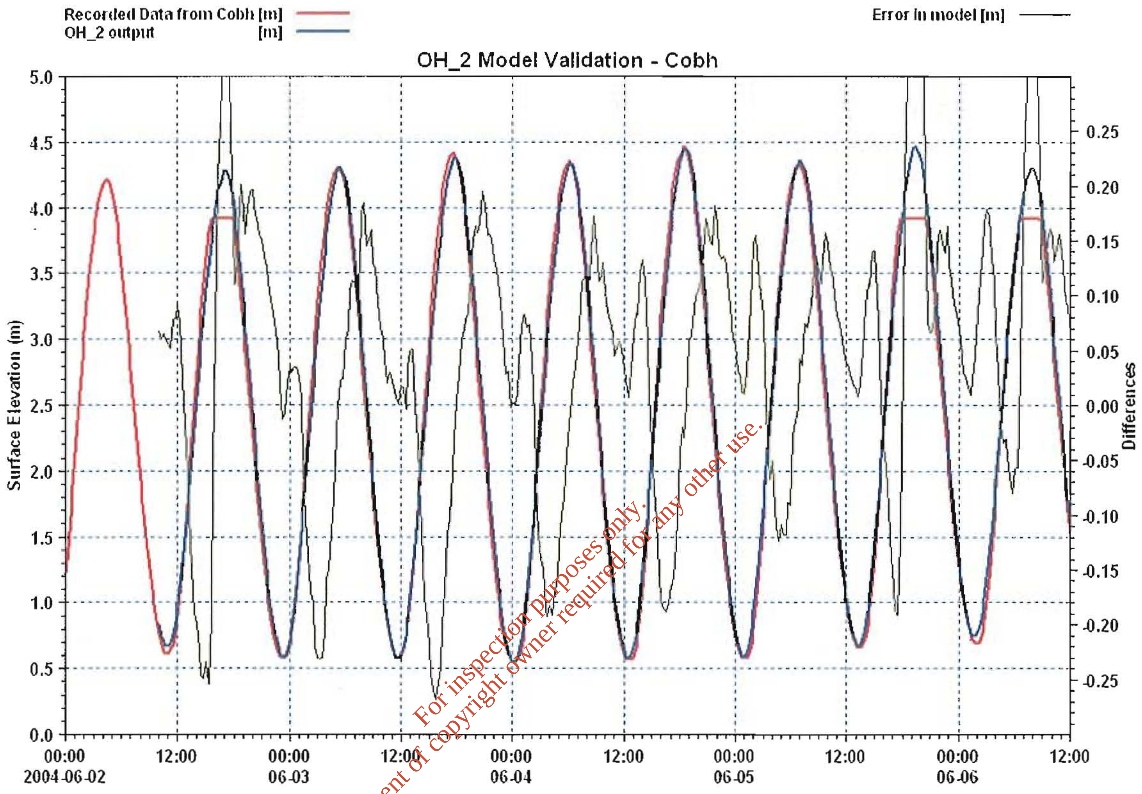

Fig. 3.3 Cobh Spring Tide Water Level Validation

[18] No current speed or direction measurements were available for this validation period.

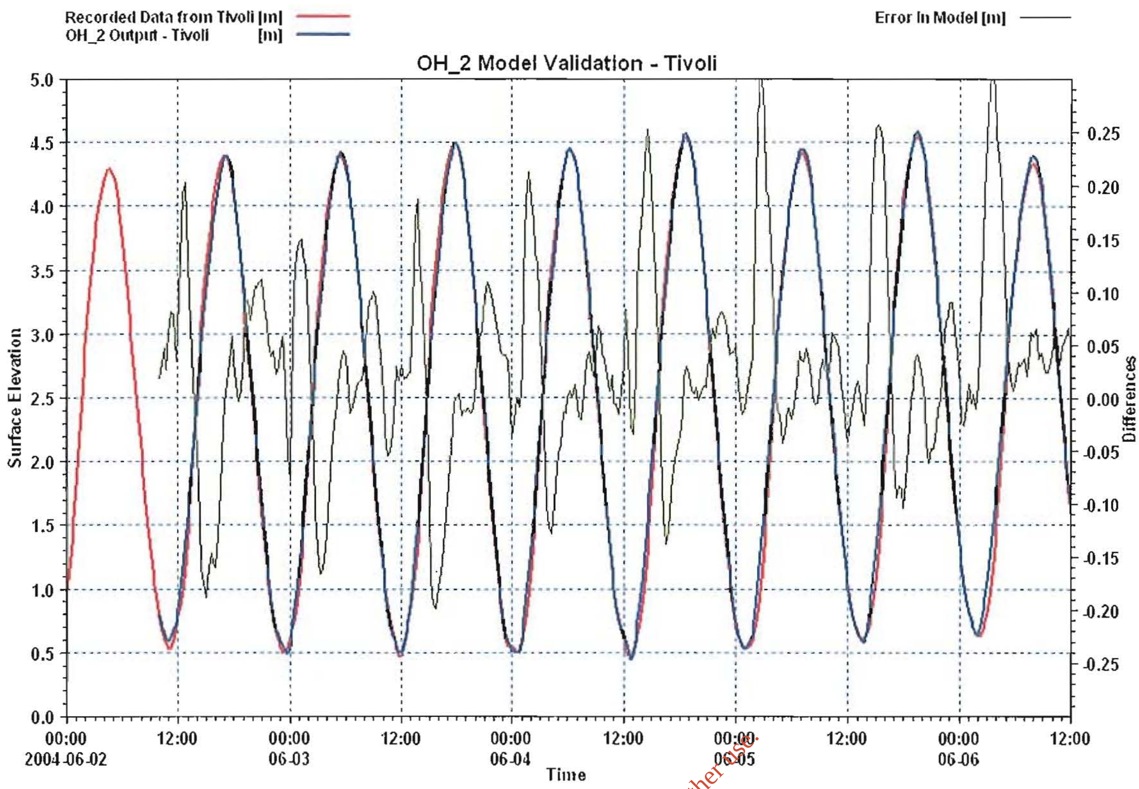

Fig. 3.4 Tivoli Spring Tide Water Level Validation

3.5 OH_2 model parameters

The OH_2 model has two parts. The first is the hydrodynamic model, which predicts the numerical variation in water level and the speed and direction of currents throughout Cork Harbour. Coupled with this is the Advection-Dispersion (AD) model, which describes the dispersal and decay of bacteria, *Norovirus* and Nitrogen discharged at any location in the Harbour. Numerous parameters are required for each model. Some of the values used were obtained through the calibration and validation as described in the previous section. Some were chosen based on experience and guidance from the literature.

3.5.1 Hydrodynamic Model Parameters

The main parameters in the RP model are listed as:

- **Δx – grid resolution**. 3 different resolutions were used in the OH_2 model as described in the last section.

- **Δt – timestep.** A timestep of 6 seconds was used for the model. This reasonably low value was found necessary to ensure the Advection Dispersion model remained stable.

- **Eddy Viscosity.** A flux-based formulation of the eddy viscosity, which varies over the entire grid, has been used. The eddy viscosity parameter is shown in the figure below. These values were determined by calibration.

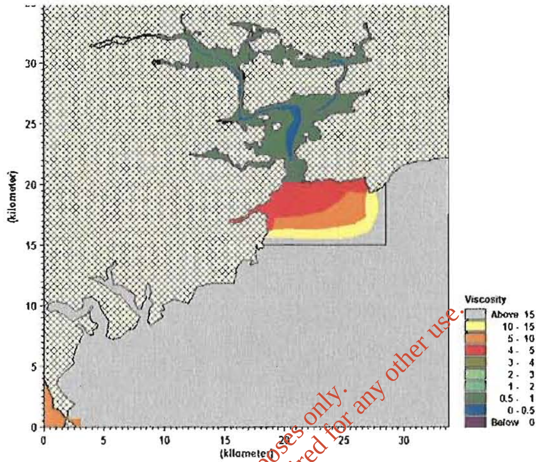

Fig. 3.5 Map of eddy viscosity values used for the OH_2 model

- **Bed Resistance.** The bed resistance was defined using the Manning's M number. The parameter varied over the entire grid as can be seen in the figure below. These values were determined from the calibration of the model.

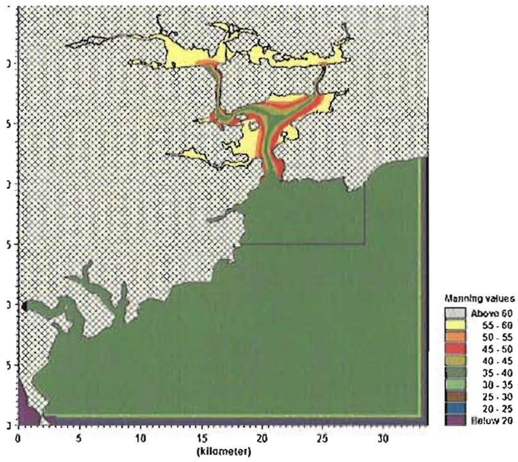

Fig. 3.6 Manning's M value used in model. Manning's M ($m^{1/3}/s$) is the reciprocal of Manning's n.

- **Flooding and Drying depths.** MIKE 21 allows the simulation of flow in areas that are subject to flooding and drying. When an area dries out the grid cells are removed from the computations. When the tide returns and floods the area the grid cells are included in the computations again. The flooding and drying depths control this inclusion and exclusion of computational points. The default values in MIKE 21 are 0.2m (drying) and 0.3m (flooding). Therefore when the depth of water in a grid cell is less than 0.2m the cell is removed from the computations. When the tide is on the flood and the water level is calculated to be above 0.3m, the grid cell is once again included in the computations. Values of 0.1m and 0.2m were used in this study.

3.5.2 AD Model Parameters

There are a number additional parameters required for the Advection dispersion model. These parameters are:

- **Initial conditions.** These were set to zero across the entire grid i.e. it was assumed that the concentrations of bacteria, *Norovirus* and Nitrogen were zero across the entire harbour at the start of the simulation.

- **Boundary Conditions.** The boundary conditions at the mouth were set to zero for the duration of the simulations.

- **Decay specification.** Bacteria and *Norovirus* decay exponentially with time. We have assumed that Faecal Coliforms have a T90 of 12 hours. We have also simulated the decay with a T90 of 24 hours as part of a sensitivity analysis. We have assumed that *Norovirus* has a T90 of 30 days. This applies to winter conditions which is a worse case scenario.

- **Dispersion Coefficient.** The dispersion coefficients in MIKE 21 may be defined as either independent of the current or proportional to the current. The results presented in this report use the independent option. A value of $1 m^2/sec$ in both the x- and y-direction has been used across all three grids in the OH_2 model.

- **Feedback.** By including the hydrodynamic (HD) density terms in the advection dispersion model, horizontal density gradients become another forcing function in the hydrodynamic model[19]. The influence of salinity and temperature may be included in this way. The results presented in this report do not include feedback[20] due to the unavailability of high-frequency measurements of salinity.

3.6 Discussion

The OH_2 model has been developed in MIKE 21 to simulate the discharge and transport of Bacteria, *Norovirus* and three species of Nitrogen from various outfalls in Cork Harbour. The first part of the OH_2, the hydrodynamic model, predicts the variation in water level and current speed from the Old Head to the Waterworks weir. The second part of the OH_2, the Advection-Dispersion model, describes the dispersal and decay of Faecal Coliforms, *Norovirus* and three species of Nitrogen for the same area.

The hydrodynamic parameters of the OH_2 model are based on the calibration and validation of a separate model, the RP_2 model, which covers an area from Roches Point to the Waterworks weir.

The boundary conditions of the OH_2 model are supplied by output from a numerical model of part of the North Atlantic Shelf which is maintained and run by the Proudman Oceanographic Laboratory in the UK. From the validation of the OH_2 model we may conclude that it reproduces the tides in Cork Harbour to an error within 20cm.

[19] In addition to the tide, wind and river flows.

[20] Significant horizontal density gradients are unlikely to occur in the areas of interest in the outer harbour and outside the mouth.

Chapter 4 Faecal Coliform Results

4.1 Introduction

This chapter presents the results of the faecal coliform modelling. We have assumed that there are $1.0*10^{11}$ faecal coliforms in every cubic metre of raw sewage which is equivalent to $1.0*10^{7}$ faecal coliforms in every 100ml[21]. This concentration, multiplied by the flow rate for each town (as listed in Chapter 1) gives the loading for each outfall. We have assumed that the proposed waste water treatment plant will remove 90% of the organic matter so that there are $1.0*10^{10}$ faecal coliforms in every cubic metre of treated effluent which is equivalent to $1.0*10^{6}$ faecal coliforms per 100ml.

We have used the results of the faecal coliform model to predict the concentrations of intestinal enterococci and *Escherichia coli* (sections 4.6 & 4.7).

A complete list of the production runs for the faecal coliform modelling is presented in the following table. Further production runs were simulated as part of a sensitivity analysis which is presented in sections 4.5. We examine the change in faecal coliform concentrations when a T90 of 24hours and different wind forcing are used.

	PR	Boundary	Forcing	T90
Case 1	**1**	Rpt Springs	Recorded wind & river flows	12hr
"	**2**	Rpt Neaps	Recorded wind & river flows	12hr
Case 2	**3**	Rpt Springs	Recorded wind & river flows	12hr
"	**4**	Rpt Neaps	Recorded wind & river flows	12hr
Case 3	**5**	Rpt Springs	Recorded wind & river flows	12hr
"	**6**	Rpt Neaps	Recorded wind & river flows	12hr
Case 4	**7**	Rpt Springs	Recorded wind & river flows	12hr
"	**8**	Rpt Neaps	Recorded wind & river flows	12hr

*Table 4-1 List of Production Runs **(PR)** for faecal coliform modelling. Recorded river flows were used for each run.*

[21] Tchobanoglous, G.; Burton, F.L. and Stensel, H.D.(2003). Wastewater Engineering: Treatment and Reuse/Metcalf & Eddy Inc. 4th Ed./Revised.

4.2 Spatially varying maps of Faecal Coliform concentration

This section presents the spatially varying maps of the maximum and averaged concentration for the entire model area. Over the course of the model run the number of faecal coliforms at each grid point will, at some specific moment, reach a maximum value. These maxima, at each and every grid point, may be extracted from the result files of a production run and plotted together on a single diagram. This diagram illustrates the spatially varying maximum concentrations for the simulation period for Cork Harbour. The times at which the concentrations reached their individual peak value are not considered.

In the same way there will be an average value in concentration for each grid point over the course of the simulation run. These averages, at each and every grid point, may be extracted from the result files of a model run and plotted together on a single diagram.

To aid the reader, the same colour palette is used for each plot and is shown on the right-hand side in each case. The full range of colours in the palette is used for the existing situation. Fewer colours are required for the proposed situation indicating a substantial relative reduction in concentration for faecal coliforms.

4.2.1 Repeating Spring Tides – Spatially Varying Maxima

Fig. 4.1 presents the maximum concentrations for Case 1, production run PR1. We can see from the figure that the highest concentrations are located just upstream of each of the outfalls. As the T90 is 12 hours in this run the bacteria decay rapidly upon being released from the outfall. There is a substantial drop in the maximum concentrations within a short distance of the outfall.

The bacterial plumes with concentrations in excess of 500 fc/100ml (red colour and above in the palette) do not extend[22] into the North Channel.

[22] The lowest value on the palette is 2 fc coliforms per 100ml. Values below this are not shown.

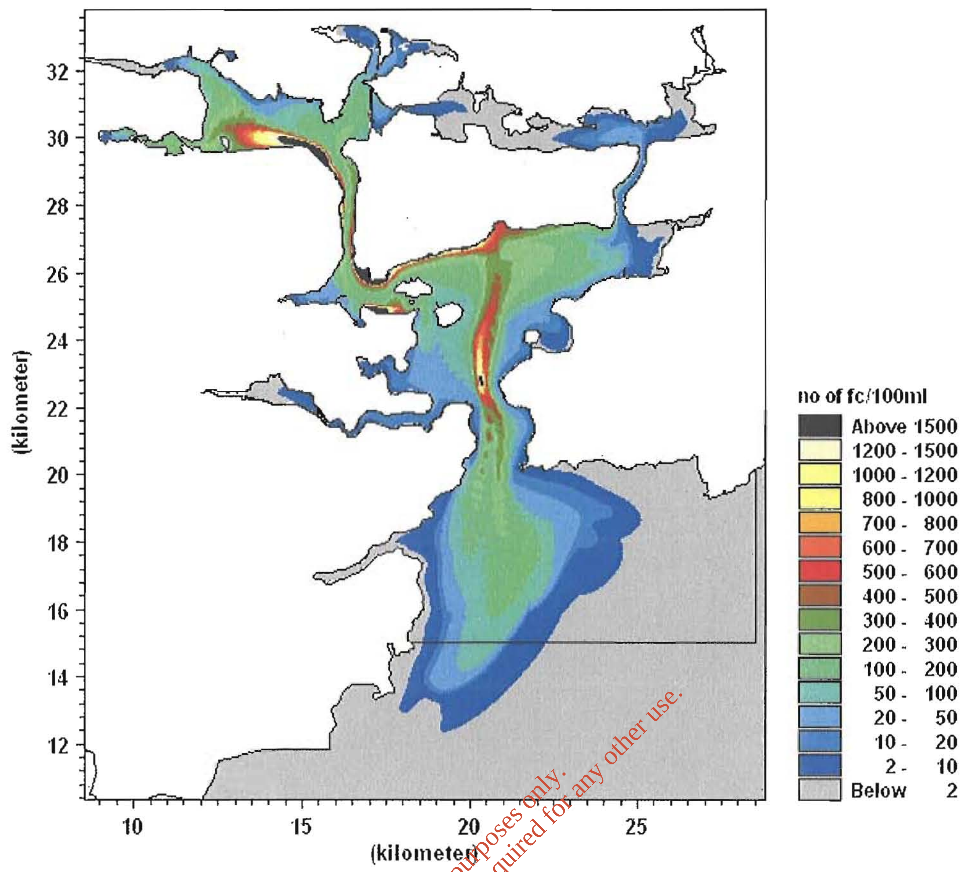

Fig. 4.1 Case 1, Production Run (PR) 1 – Maximum Concentrations[23]

Fig. 4.2 presents the maximum concentrations for Case 2, PR3. We can see how the maximum concentrations in the harbour increase as a result of the projected growth in population.

[23] We can see from Fig. 4.1 that discontinuities exist in the bands of concentration in the plot. If we follow a line due south from the location of the outfall we can see that patches of light red shading (400 -500 fc/100ml) are contained within the dark green shading (300 -400 fc/100ml). This is an artefact of the 15min sampling of the results generated by the model every 6 seconds (Δt = 6 seconds). Results were saved every 15 minutes and so some peak values were aliased in the writing of the result file. The error in the interpretation of the figure is not significant.

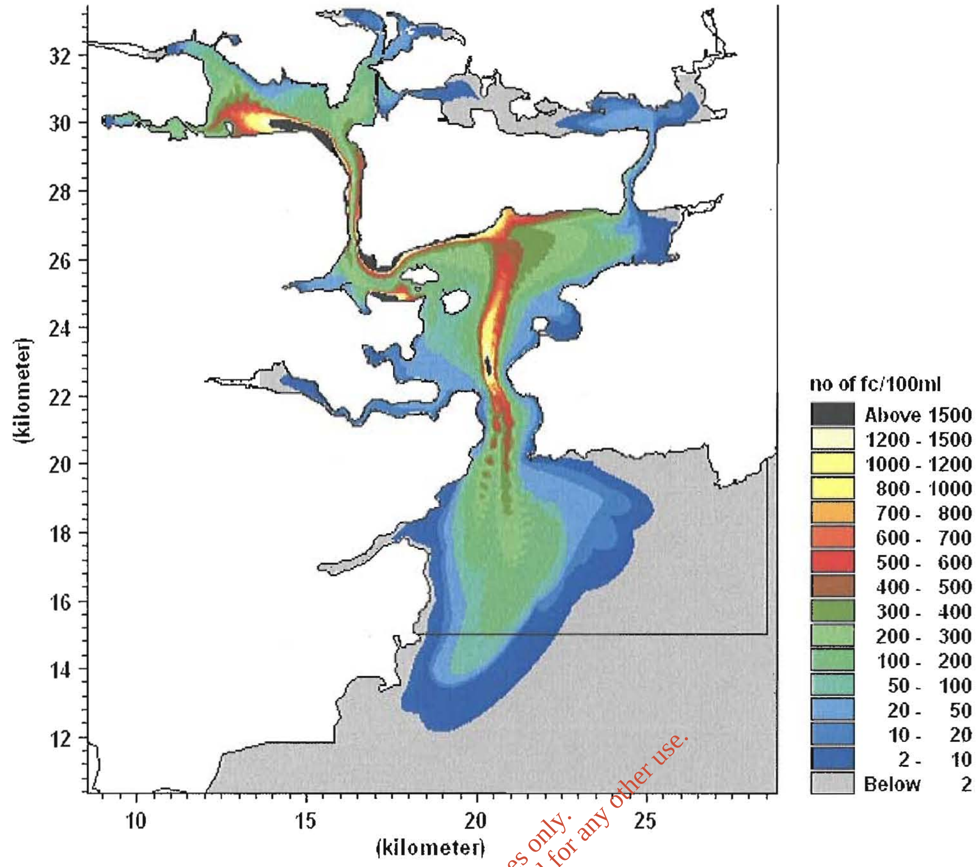

Fig. 4.2 Case 2, PR 3 – Maximum Concentrations

We have assumed that the 2010 design population is divided between the individual towns in the same way as it was for the 2001 situation. The individual flow rates are presented in the following table.

Outfall	CASE 1 (2001) – no teatment	CASE 3 (2010) - with treatment	**CASE 2 (2010) - no treatment**
Carrigaline/Crosshaven	4,075 m	10,371	**5624**
Passage West	547	0	**755**
Glenbrook	327	0	**451**
Monkstown	185	0	**255**
Pilots Pier Outfall	353	0	**487**
Corbett Outfall	178	0	**246**
Kings Quay Outfall	444	0	**613**
West Beach Outfall	668	0	**922**
White Point Outfall	634	0	**875**
Ringaskiddy Outfall	101	0	**139**
Total Flow Rate	7,515	10,371	***10,371***

Table 4-2 Design flow rates (m^3/day) for Case 2.

The flow rates for Case 2 given in the table above were obtained by multiplying the 2001 flow rates by 1.38. This scaling factor is obtained by dividing the combined flow rate for 2010 (10,371m3/day) by the combined flow rate for 2001

(7,515m3/day). Because the model is linear we may multiply all the concentrations in the harbour for 2001 by 1.38 to obtain 2010.

For both situations we can see that the concentrations of faecal coliforms, with the exception of the areas immediately surrounding the outfalls, range from 2 - 1200 fc/100ml.

Fig. 4.3 presents the maximum concentrations for Case 3, PR5. We can see from the figure that there has been a reduction in the number of faecal coliforms over the entire harbour with the introduction of the proposed treatment plant at Carrigaline. With the exception of the area immediately surrounding the proposed outfall the concentration of faecal coliforms is less than 300/100ml.

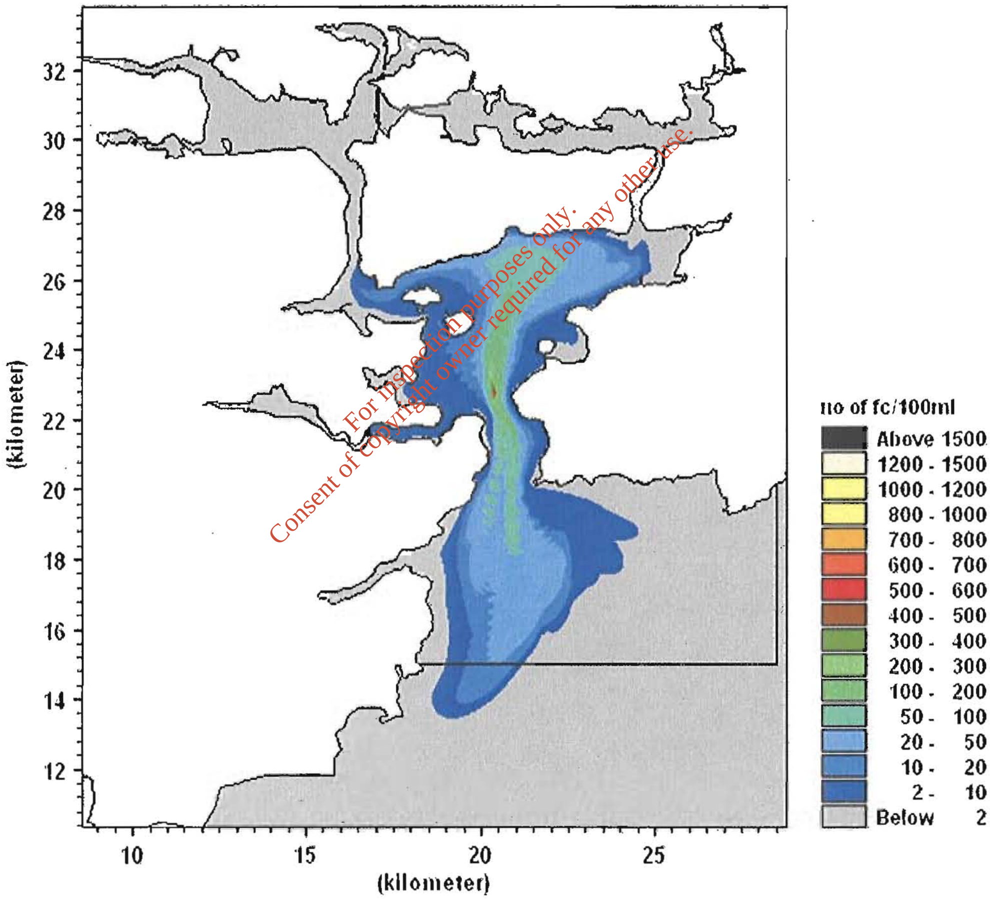

Fig. 4.3 Case 3, PR 5 – Maximum Concentrations

In order to quantify this improvement in water quality we can express the maximum concentrations with the treatment plant in place as a percentage of the maximum concentrations without the treatment plant in place (Fig. 4.4). We do this by dividing the maximum concentrations for Case 3 by the maximum concentrations for Case 2 and multiply the answer by 100. We can see from Fig. 4.4 that there has been a considerable relative reduction in the number of faecal coliforms across the entire harbour. The concentrations with the treatment plant in place are at least less than 20% of the concentrations without the treatment plant in place for the entire area i.e. there is an 80% relative reduction in the number of faecal coliforms. For the Inner harbour and the East and West Passages the concentrations are less than 5% i.e. there is a 95% relative reduction in the number of number of faecal coliforms. This represents a significant improvement in water quality.

Fig. 4.5 presents the maximum concentrations for Case 4, PR 7. As we can see from the figure there is an increase in the maximum concentration over the entire grid. The values in this plot are the values presented in Fig. 4.3 (Case 3, PR5) multiplied by 1.431. All of the models presented in this report obey the principles of superposition and scaling in both time and space. This allows us to scale up or down the results of a simulation run based on either an increase or decrease in the input concentrations or flow rates[24]. In this instance we have multiplied the values for PD5 by 1.431. This scaling factor is obtained by dividing the combined flow rate for 2030 (14,837m3/day) by the combined flow rate for 2010 (10,371m3/day). The principle of superposition allows us to multiply all the concentrations in the harbour for PR5 by 1.431 to obtain PR7.

[24] The necessary conditions for the theorem of superposition are (1) the boundary conditions must be zero, and (2) all carrier flows must be present in each individual case in both the hydrodynamic and water quality parts of the model. The proof of the theorem follows immediately from the linearity of the partial differential equation that describes the water quality dynamics.

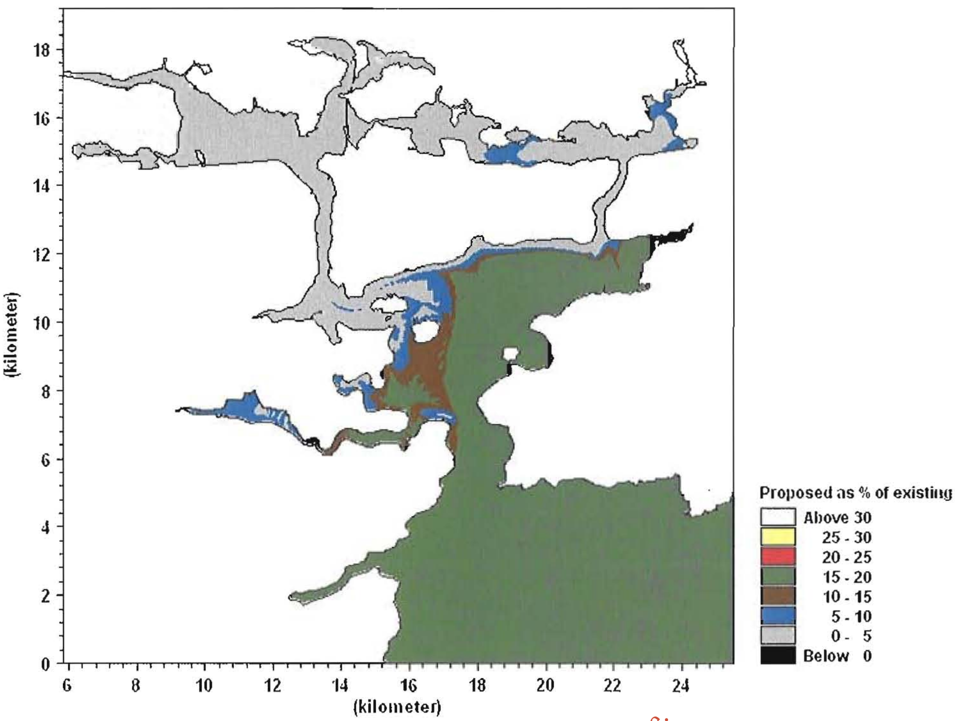

Fig. 4.4 PR3 as a % of PR5 - Maximum Concentrations

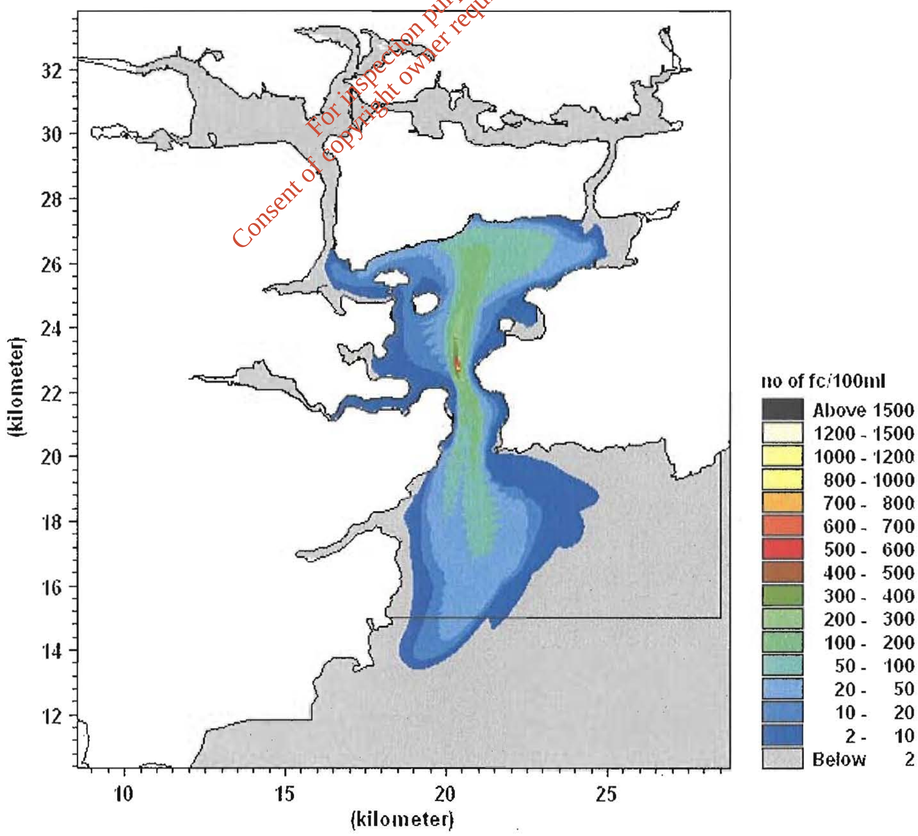

Fig. 4.5 Case 4, PR 7 – Maximum Concentrations

4.2.2 Repeating Spring Tides – Spatially Varying Averages

The spatially varying averages are now presented. The averaged concentrations for Case 2[25] are highlighted in Fig. 4.6. We can see from the figure that the concentrations are much less than the maximum concentrations presented in the previous section. With the exception of the areas adjacent to the outfalls the averaged concentrations are less than 200 fc/100ml.

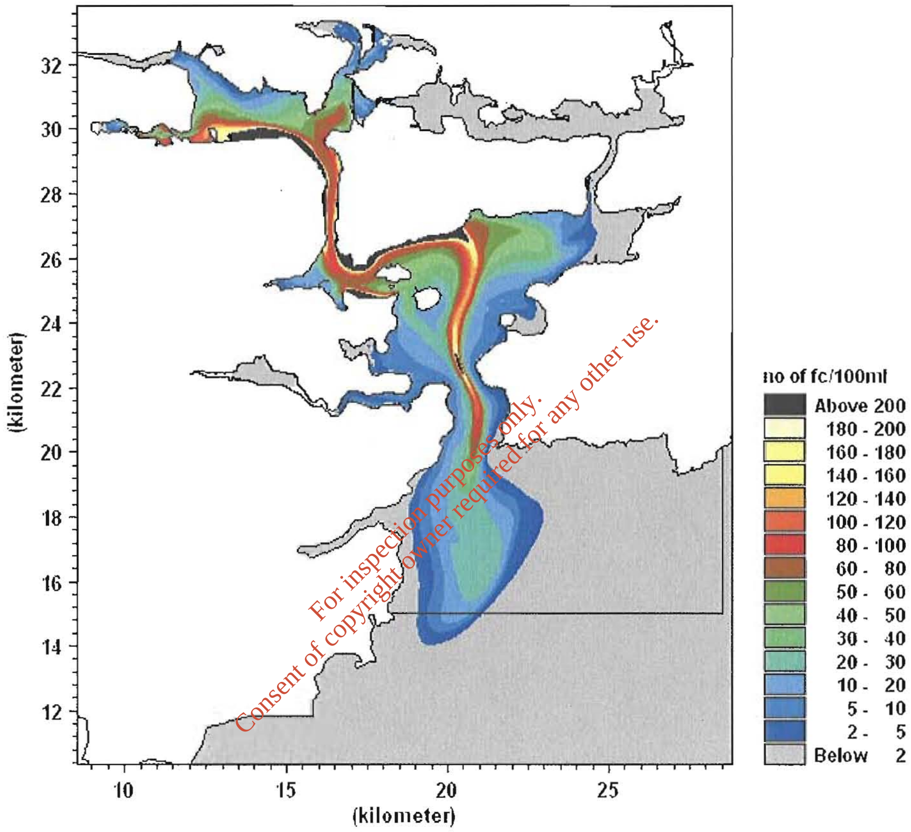

Fig. 4.6 Case 2, PR 3 – Averaged Concentrations

The averaged concentrations for Case 3, PR 5 are presented in Fig. 4.7. We can see from the figure that the averaged concentrations with the proposed treatment plant in place are greatly reduced. To quantify this improvement in water quality we can, as before, express the averaged concentrations for PR5 as

[25] The averaged concentration map for Case 1 has been omitted as it is visually very similar to this plot. The equivalent plots for the neap tides in the next two sections have also been omitted for the same reason.

a percentage of the averaged concentrations for PR3. This is shown in Fig. 4.8. We can see from the figure that the improvement for the averaged concentrations is similar to that of the maximum concentration (Fig. 4.4). The number of faecal coliforms is reduced by at least 80% for the entire model area. For the inner harbour they are reduced by at least 95%.

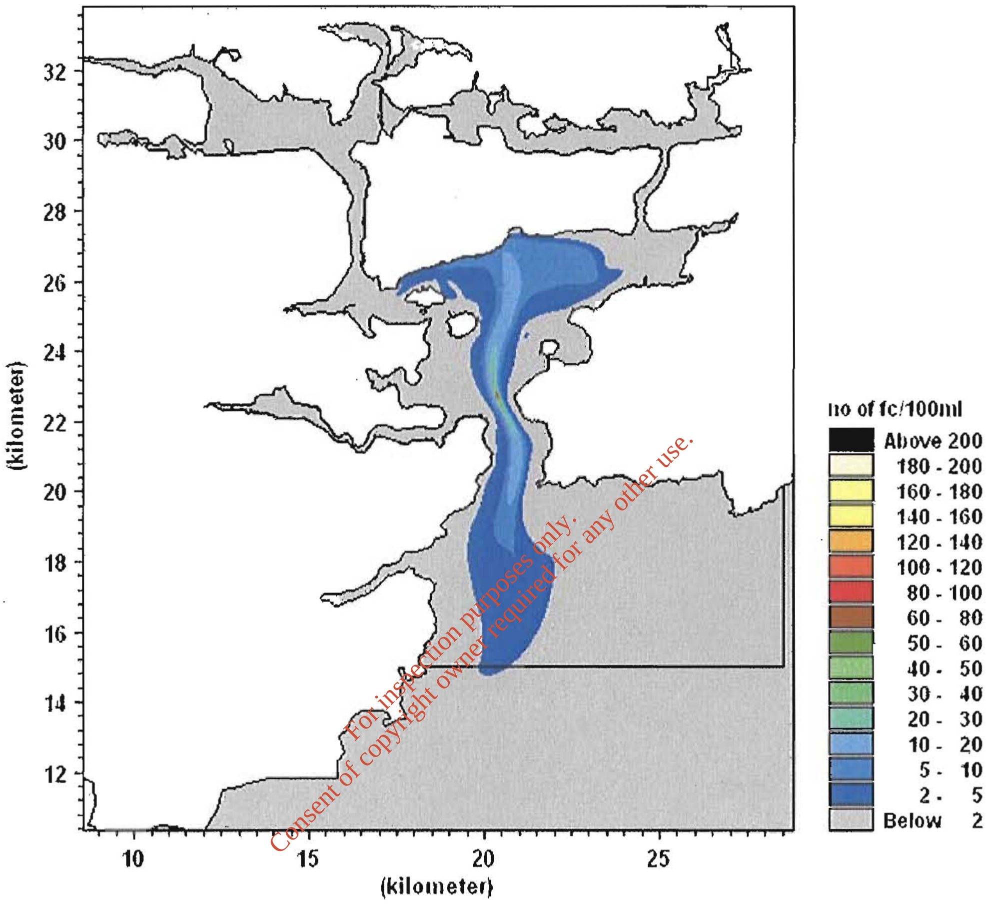

Fig. 4.7 Case 3, PR 5 – Averaged Concentrations

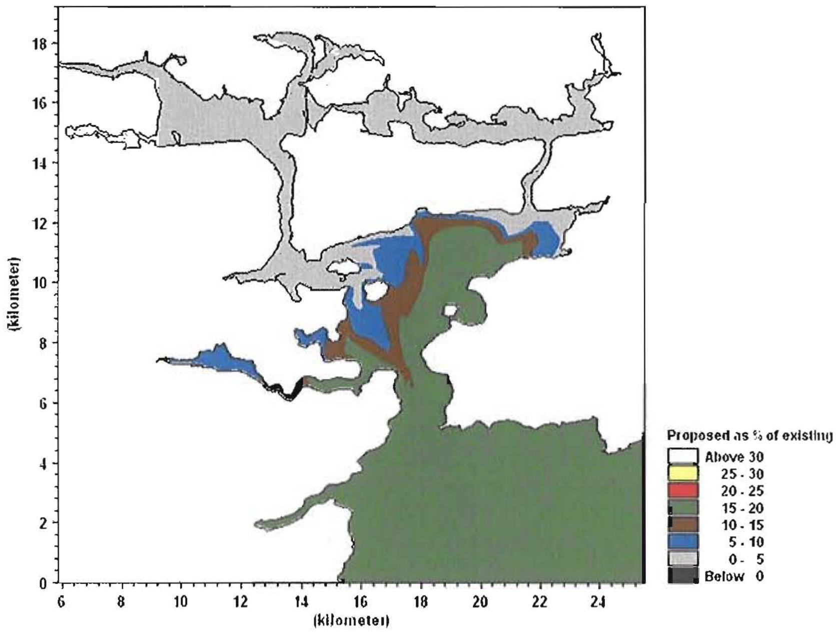

Fig. 4.8 PR3 as a % of PR5 - Averaged Concentrations

4.2.3 Repeating Neap Tides – Spatially Varying Maxima

The spatially varying maximums for the neap tides are presented in this section. We can see from Fig. 4.9 that the concentrations in the harbour are comparable to the equivalent spring tide simulation (Fig. 4.2). The extent of the plume however differs outside the harbour mouth.

For PR6 (Fig. 4.10) we can see that there is a reduction in the number of faecal coliforms per 100ml with the introduction of the proposed treatment plant. With the exception of the area immediately adjacent to the proposed outfall the number of faecal coliforms per 100ml ranges from 2 to 500 fc/100ml.

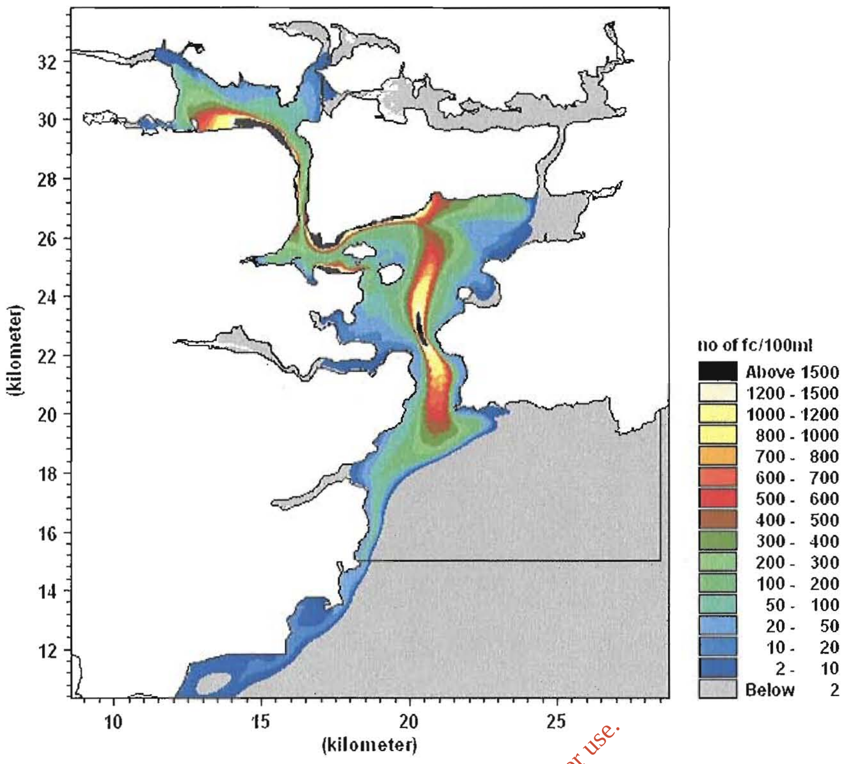

Fig. 4.9 Case 2, PR 4 – Maximum Concentrations

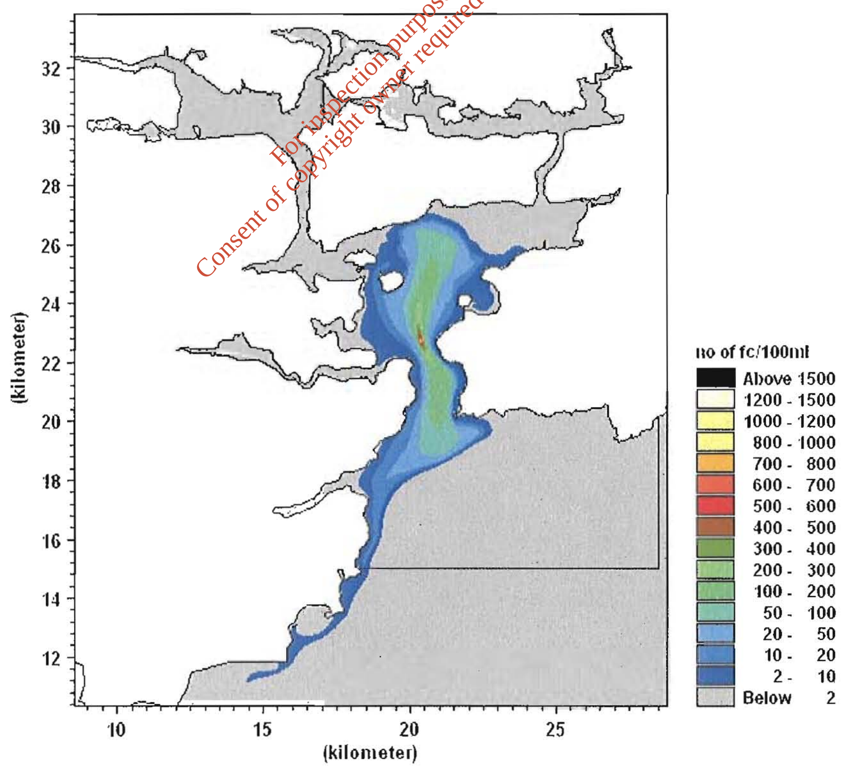

Fig. 4.10 Case 3, PR 6 – Maximum Concentrations

4.2.4 Repeating Neap Tides – Spatially Varied Averages

The spatially varying averages for the repeating neap tides are now presented. We can see from Fig. 4.11 (PR 4, Case 2) that the averages for the neap tides are similar to those of the spring tides. As with the equivalent maximum concentrations the extent of the plume is different outside the harbour mouth. PR 6 is presented in Fig. 4.12. We can see from the figure that the averaged concentrations are greatly reduced with the introduction of the proposed scheme.

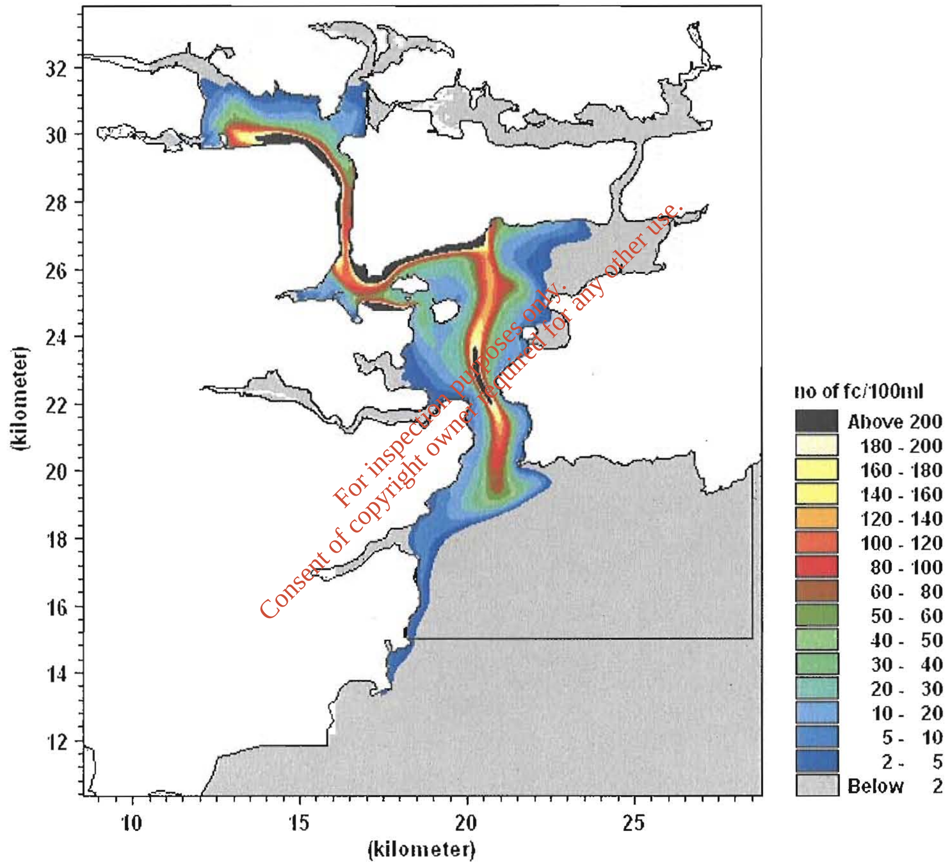

Fig. 4.11 Case 2, PR 4 – Averaged Concentrations

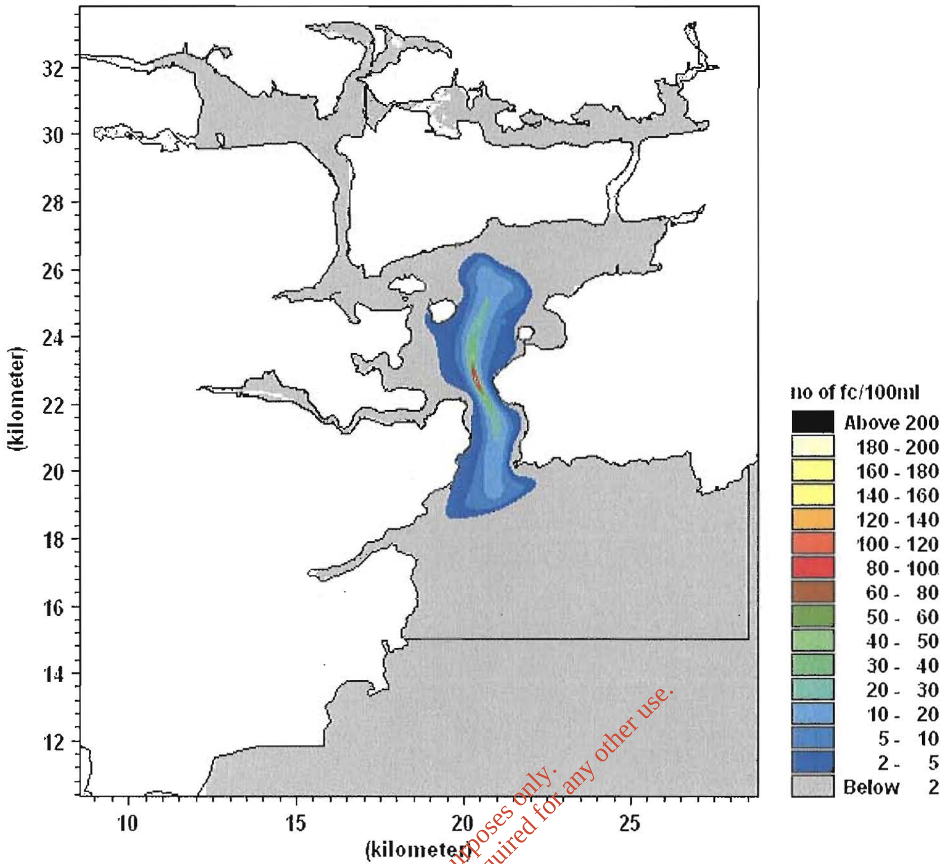

Fig. 4.12 Case 3, PR 6 – Averaged Concentrations

4.3 Time series of Faecal Coliform Concentrations

The previous section presented spatially varying maps of concentration across the entire harbour area. In order to evaluate the benefit of the proposed outer harbour drainage scheme at a particular point in the harbour we must extract the time series of concentration from that location in the model. For this Environmental Impact Statement 15 points of special interest have been identified. These are listed in the following table and plotted in Fig. 4.13.

The maximum and average value of faecal coliforms for each location is presented in Table 4-4 and Table 4-5.

Point No	Location	E (UTM)	N (UTM)
1	Fountainstown	547588	5736208
2	Myrtleville	548700	5737121
3	Roches Point	550651	5738138
4	Crosshaven	548497	5739695
5	Ringaskiddy Ferry	546466	5742772
6	Monkstown Creek	545166	5743316
7	Oyster Farm - North Channel	552712	5748103
8	Marlogue Point	554291	5745574
9	Oyster Farm - Outer Harbour	555451	5744826
10	Cobh - Recreational Area	548617	5744396
11	Spike Island - Proposed Heritage Area	549349	5742451
12	Shoreline Closest to Existing Outfall	547959	5741601
13	200m Upstream of Existing Outfall[26]	550203	5740759
14	West Passage	546223	5744496
15	Entrance to Lough Mahon	545505	5747784

Table 4-3 List of the sites of interest

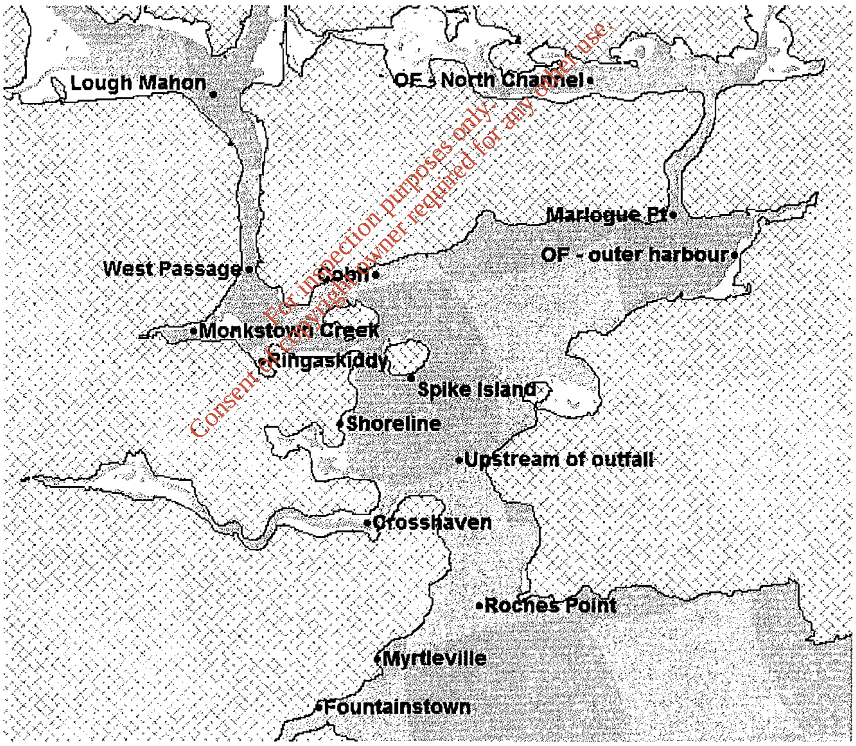

Fig. 4.13 Points of Special Interest to study

In order to make an assessment of the improvement in water quality resulting from the proposed wastewater treatment plant, time series for Case 2 and Case

[26] This point lies just outside the near field mixing zone

3 are presented in this section for both repeating spring and neap tides. The time series for Case 4 (2030) are not presented. The principle of superposition allows us to simply multiply the time series for Case 3 by 1.431 to get case 4.

Two plots are presented for each of the 15 points of special interest in this study. The first is the repeating spring tides for Case 3 (PR3) and Case 4 (PR3). The second is the repeating neap tide for Case 3 (PR5) and Case 4 (PR6).

For the repeating spring tide graphs, Case 2 is plotted with a blue line while Case 3 is plotted using red. The line is labelled "existing" in the legend indicating the existing scenario of no treatment.

For the repeating neap tide graphs Case 2 is plotted with a green line while Case 3 is plotted using dark red. This line is labelled "proposed" in the legend indicating the proposed treatment infrastructure.

The reader should be aware that the scale on the right-hand side, which indicates the number of faecal coliforms per 100ml, varies considerably for each of the 13 locations. The scale is the same however for the spring and neap graphs at each individual particular point. This allows us to determine what tidal conditions yield the highest concentration at each location.

The reader should also be aware that the dates labelled along the x-axis in all the time series refer to the period in 2004 which was chosen to simulate the model. Simultaneous measurements of wind, river flows and Proudman Data were available for this period. The simulated hydrodynamics are typical of any year and have been used for the three different cases considered in this report (2001, 2010 & 2030).

Year	**2001**		**2010**		**2010**		**2030**	
Treatment	none		none		Treated		Treated	
Repeating Tide	Spring MAX	Neap MAX	Spring MAX	Neap MAX	Spring MAX	Neap MAX	Spring MAX	Neap MAX
Fountainstown	0.8	0.4	1.0	0.5	0.2	0.1	0.3	0.1
Myrtleville	2.7	3.2	3.7	4.5	0.7	0.8	1.0	1.2
Roches Point	256.7	402.7	354.3	555.8	65.3	102.5	93.5	146.7
Crosshaven	17.7	4.1	24.5	5.6	3.8	1.5	5.4	2.2
Ringaskiddy	42.8	17.2	59.1	23.8	0.0	0.0	0.0	0.0
Monkstown Ck	24.5	85.6	33.8	118.1	0.0	0.0	0.0	0.0
Oyster F - NC	5.7	0.0	7.8	0.0	0.1	0.0	0.2	0.0
Marlogue Point	26.1	0.8	36.1	1.1	0.3	0.0	0.4	0.1
Oyster F - Outer	2.7	0.0	3.7	0.0	0.6	0.4	0.9	0.6
Cobh	346.7	344.5	478.4	475.4	23.2	0.9	33.2	1.4
Spike Island	47.8	70.3	66.0	97.0	9.1	17.8	13.1	25.5
Shoreline	11.0	10.1	15.2	14.0	2.2	1.2	3.2	1.8
Upstream Outfall	1332.5	1662.5	1838.8	2294.3	333.7	423.0	477.6	605.3
West Passage	140.9	178.0	194.4	245.6	1.0	0.0	1.5	0.0
Lough Mahon	155.8	136.5	215.0	188.4	0.1	0.0	0.2	0.0

All concentrations are expressed in no of fc per 100ml

Table 4-4 Maximum faecal coliform concentrations for locations of interest

Year	**2001**		**2010**		**2010**		**2030**	
Treatment	None		None		Treated		Treated	
Repeating Tide	Spring AVG	Neap AVG	Spring AVG	Neap AVG	Spring AVG	Neap AVG	Spring AVG	Neap AVG
Fountainstown	0.21	0.06	0.29	0.09	0.05	0.02	0.07	0.02
Myrtleville	0.57	1.27	0.79	1.75	0.14	0.32	0.20	0.46
Roches Point	46.22	55.97	63.79	77.24	11.65	14.25	16.67	20.39
Crosshaven	4.32	0.93	5.96	1.28	0.95	0.19	1.36	0.28
Ringaskiddy	13.67	5.72	18.86	7.89	0.01	0.00	0.02	0.00
Monkstown Ck	8.38	6.14	11.56	8.47	0.01	0.00	0.01	0.00
Oyster F - NC	0.01	0.00	0.02	0.00	0.00	0.00	0.00	0.00
Marlogue Point	2.62	0.03	3.62	0.04	0.04	0.00	0.06	0.00
Oyster F - Outer	0.03	0.00	0.04	0.00	0.00	0.00	0.00	0.00
Cobh	81.96	111.59	113.10	153.99	5.32	0.05	7.62	0.07
Spike Island	10.31	13.94	14.22	19.24	1.55	3.16	2.21	4.53
Shoreline	3.33	0.65	2.71	0.89	0.56	0.10	0.47	0.15
Upstream Outfall	83.78	209.64	115.62	289.31	20.12	53.32	28.79	76.30
West Passage	56.00	81.47	77.28	112.43	0.08	0.00	0.11	0.00
Lough Mahon	45.94	42.47	63.40	58.61	0.01	0.00	0.01	0.00

All concentrations are expressed in no of fc per 100ml

Table 4-5 Average faecal coliform concentrations for locations of interest

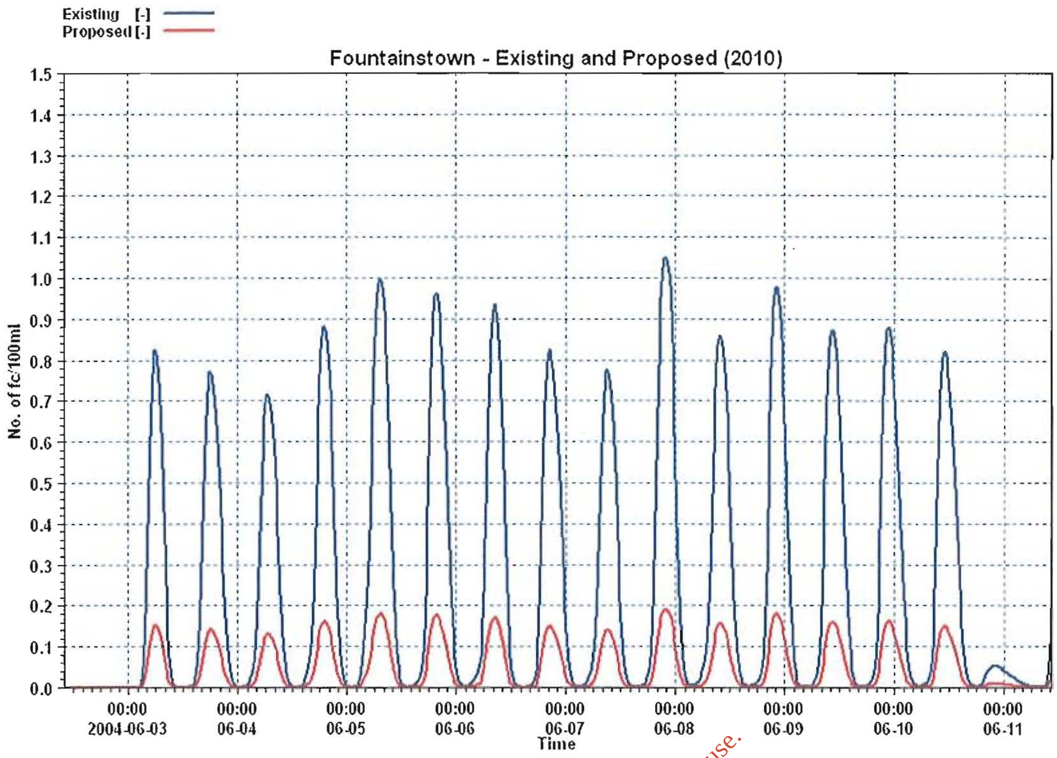

Fig. 4.14 Fountainstown – Repeating Spring Tide

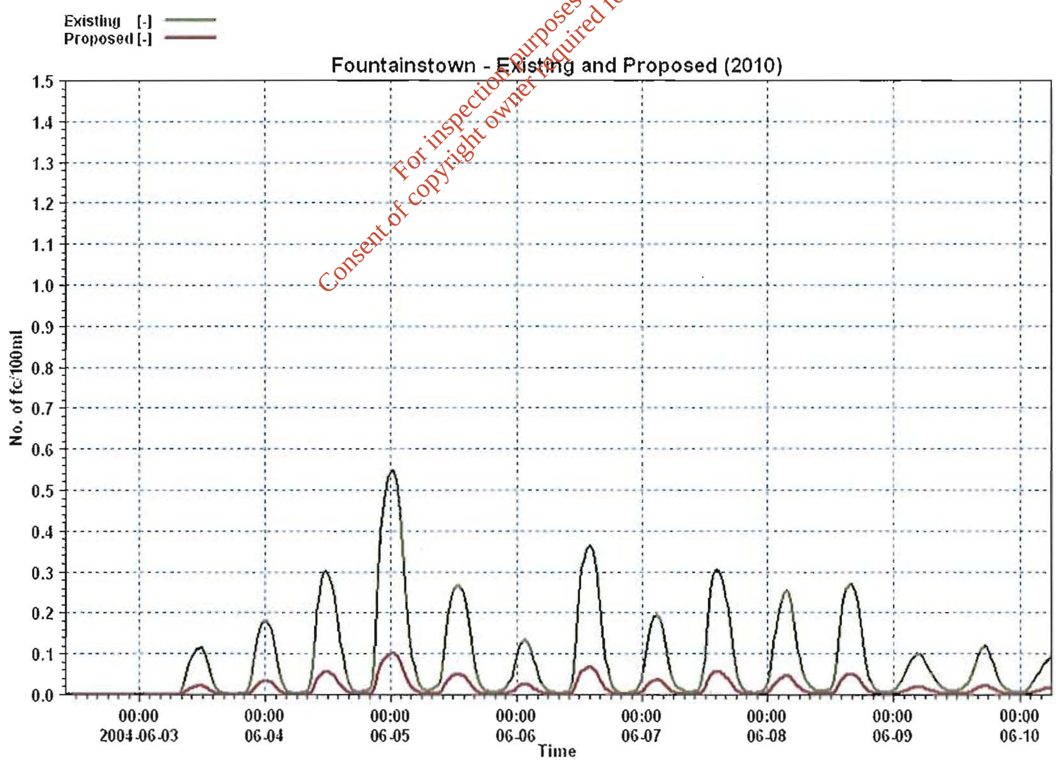

Fig. 4.15 Fountainstown – Repeating Neap Tide

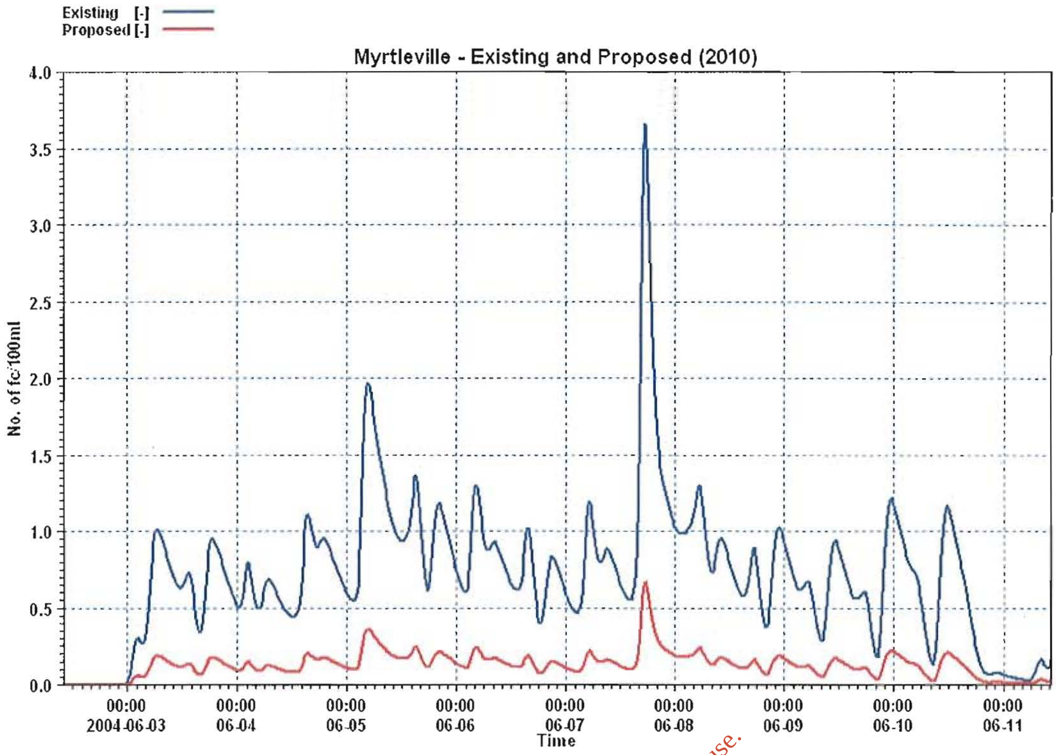

Fig. 4.16 Myrtleville – Repeating Spring Tide

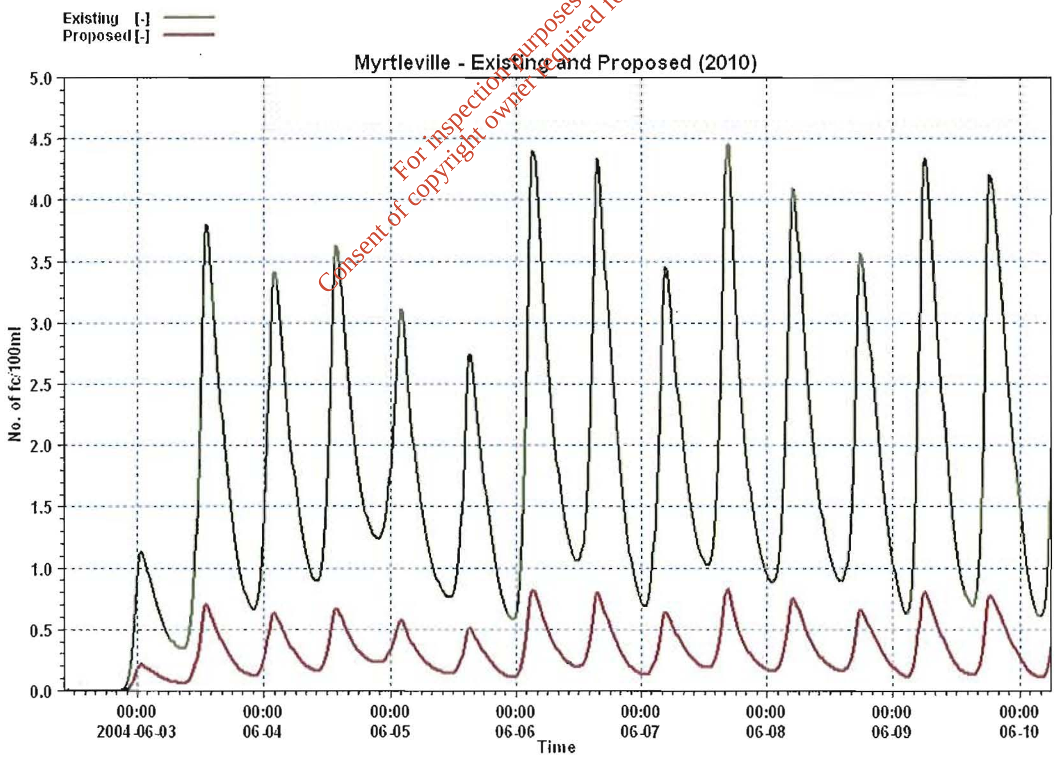

Fig. 4.17 Myrtleville – Repeating Neap Tide

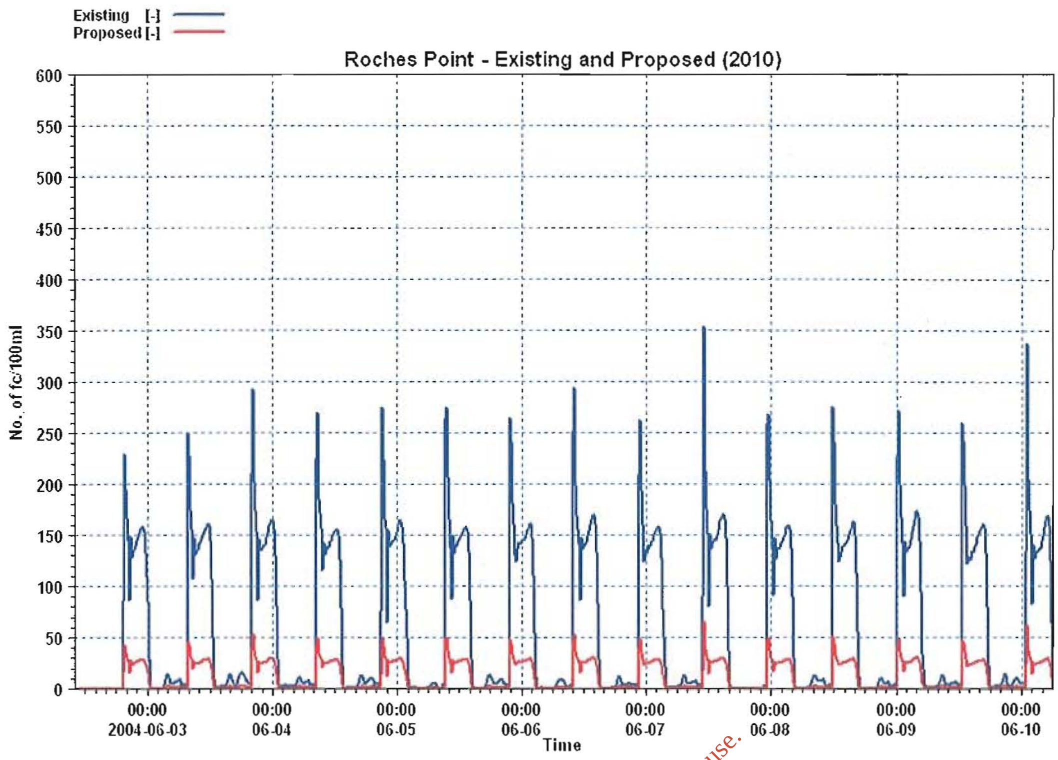

Fig. 4.18 Roches Point – Repeating Spring Tide

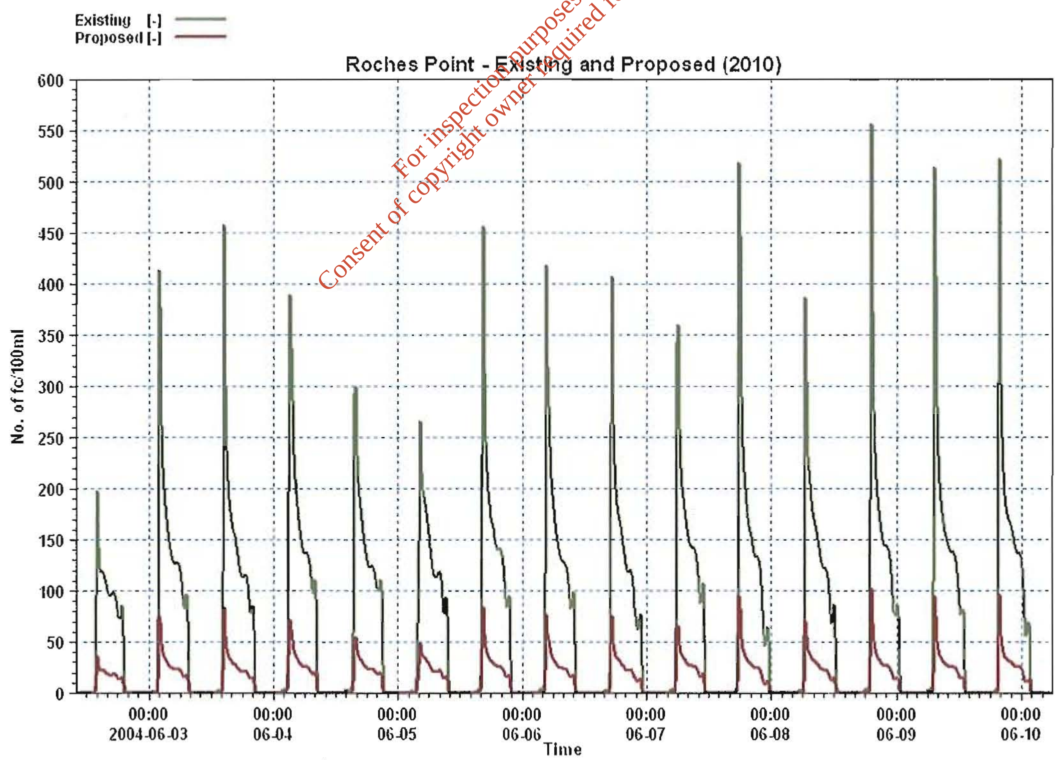

Fig. 4.19 Roches Point – Repeating Neap Tide

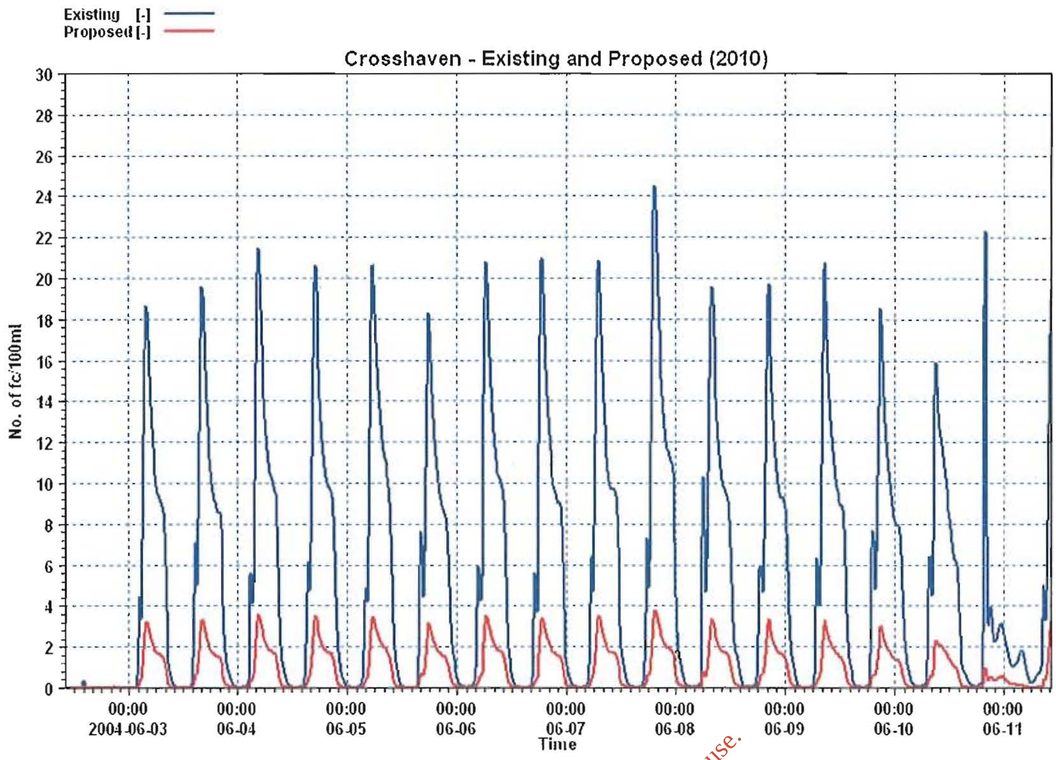

Fig. 4.20 Crosshaven – Repeating Spring Tide

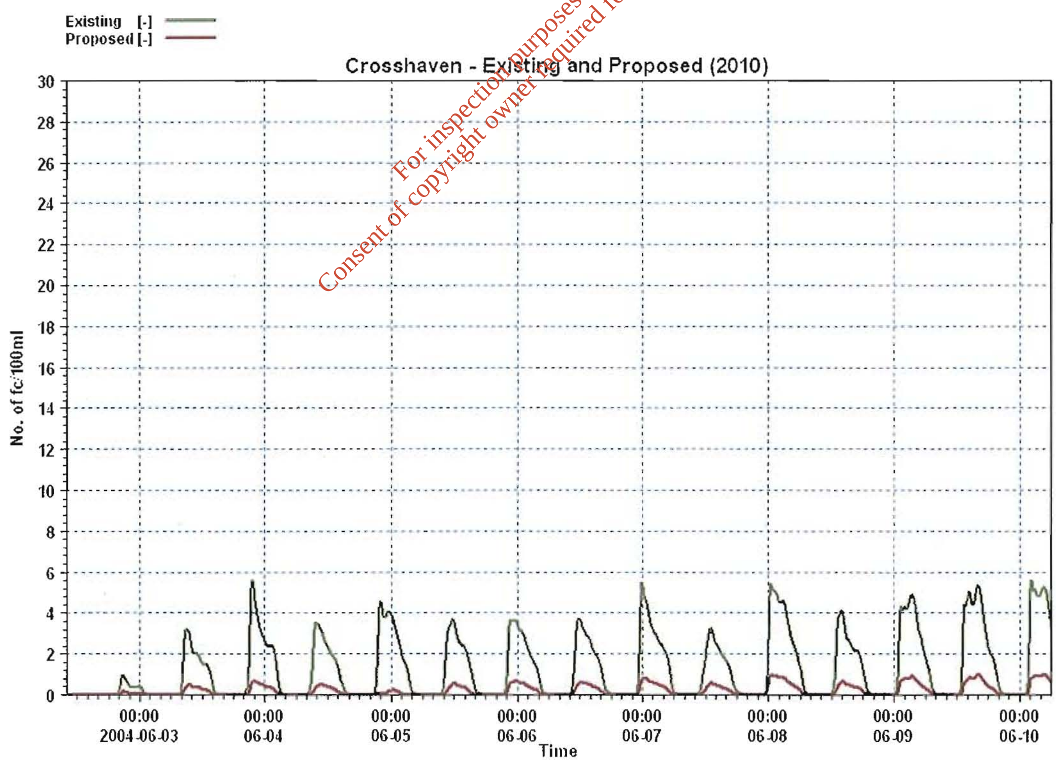

Fig. 4.21 Crosshaven – Repeating Neap Tide

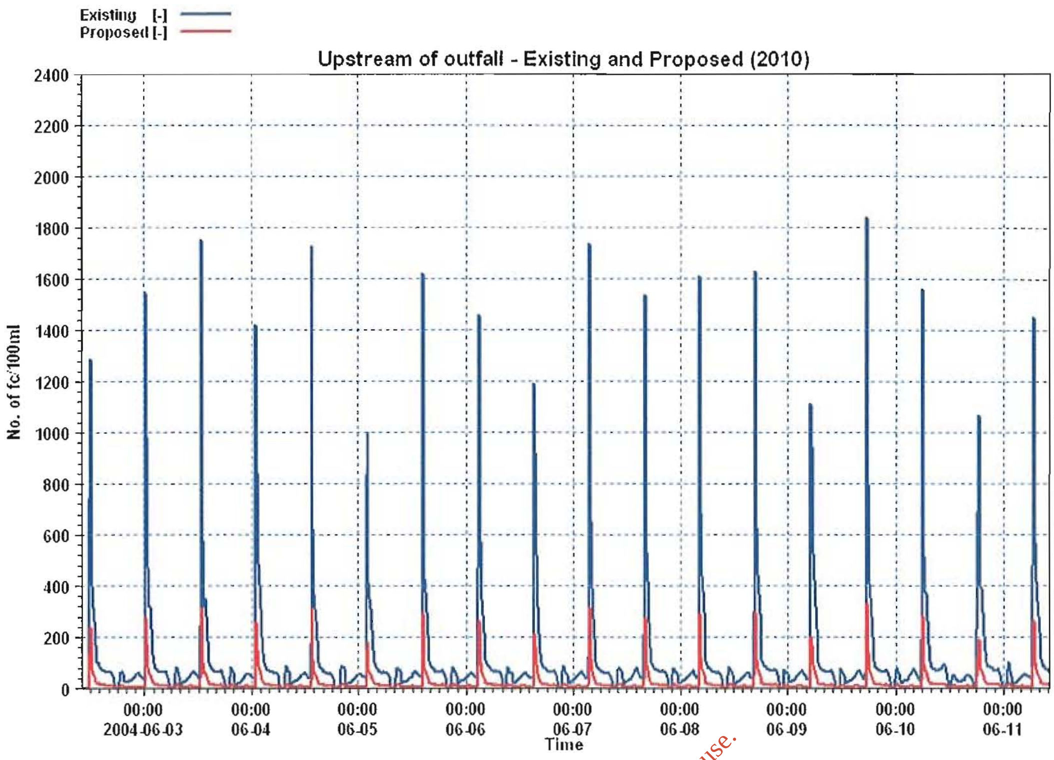

Fig. 4.22 200m upstream of outfall – Repeating Spring Tide

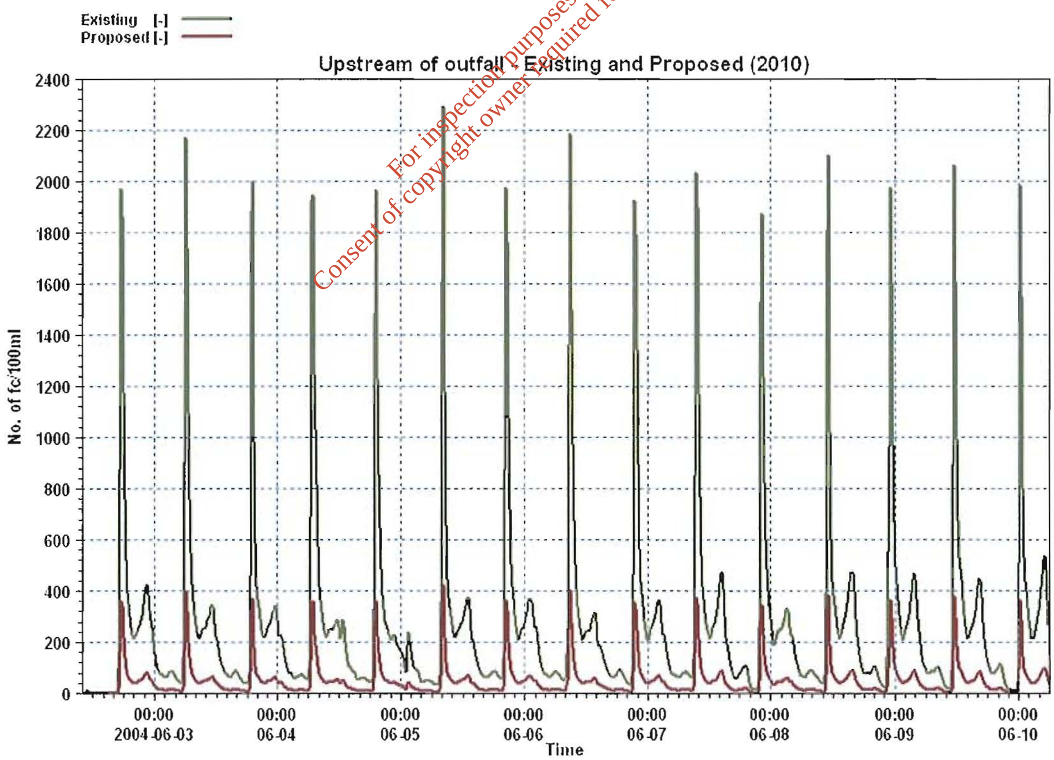

Fig. 4.23 200m upstream of outfall – Repeating Neap Tide

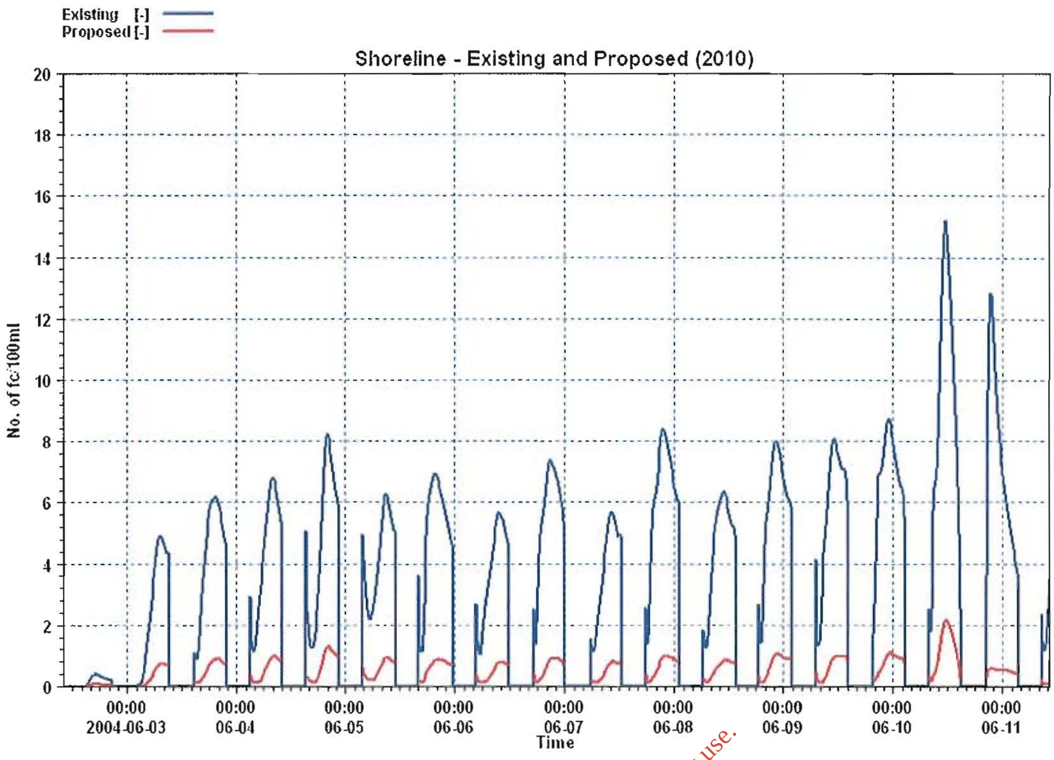

Fig. 4.24 Shoreline closest to outfall – Repeating Spring Tide

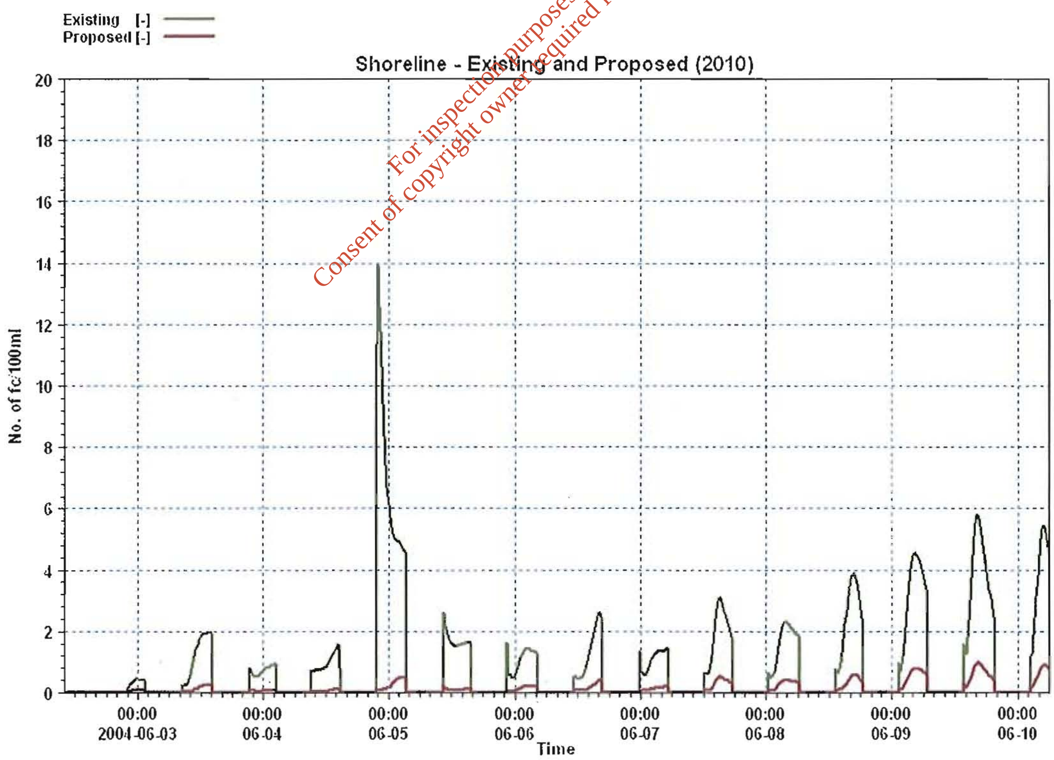

Fig. 4.25 Shoreline closest to outfall – Repeating Neap Tide

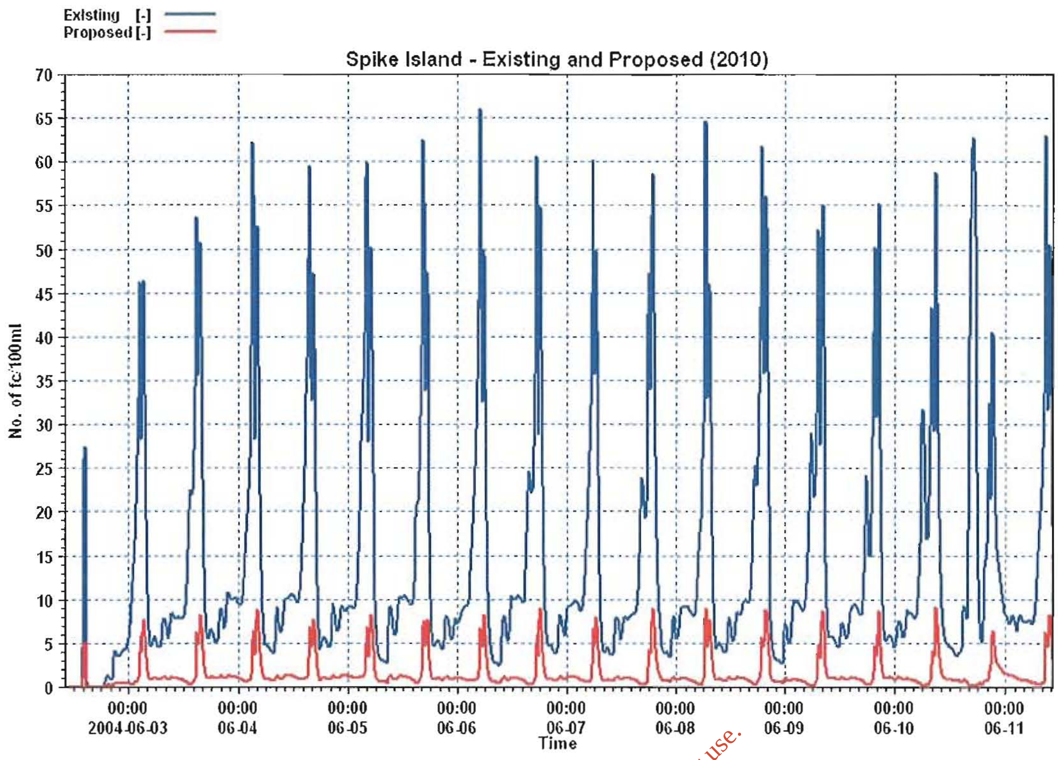

Fig. 4.26 South of Spike Island – Repeating Spring Tide

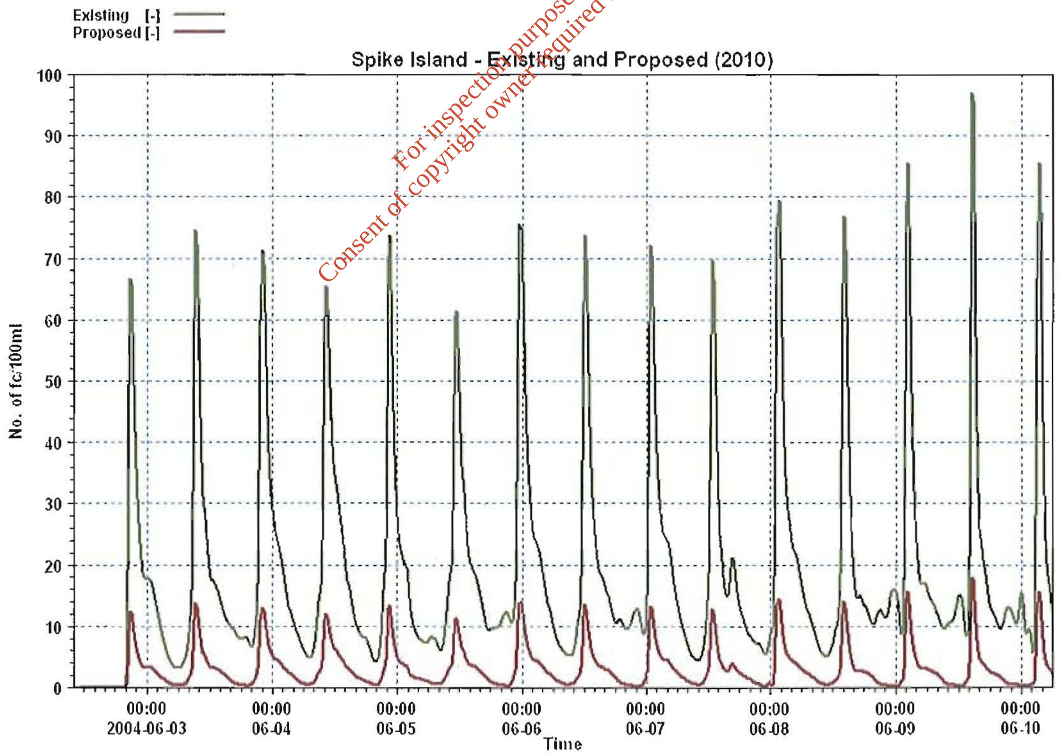

Fig. 4.27 South of Spike Island – Repeating Neap Tide

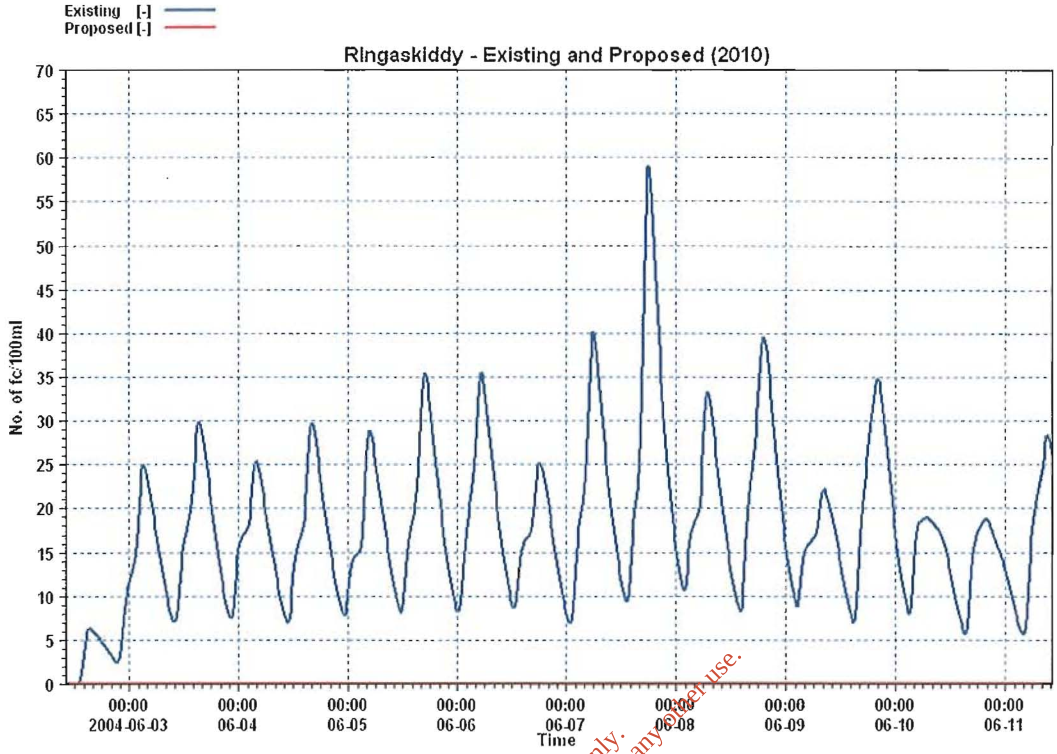

Fig. 4.28 Ringaskiddy – Repeating Spring Tide

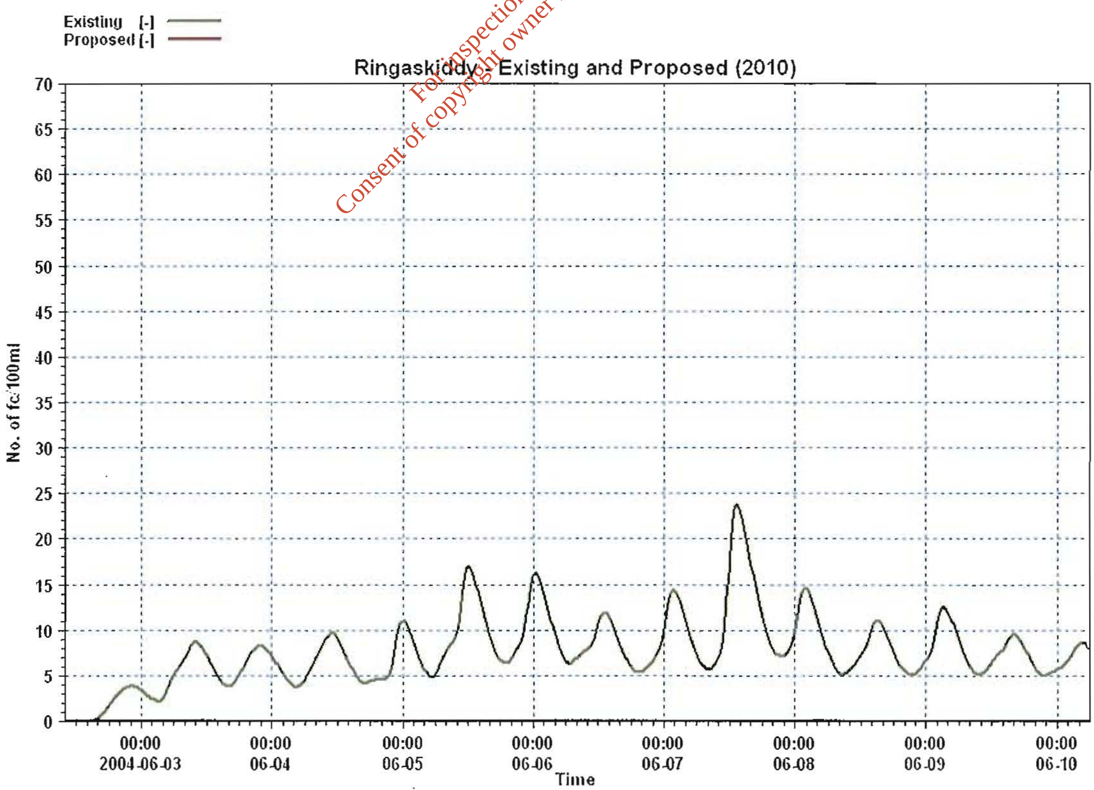

Fig. 4.29 Ringaskiddy – Repeating Neap Tide

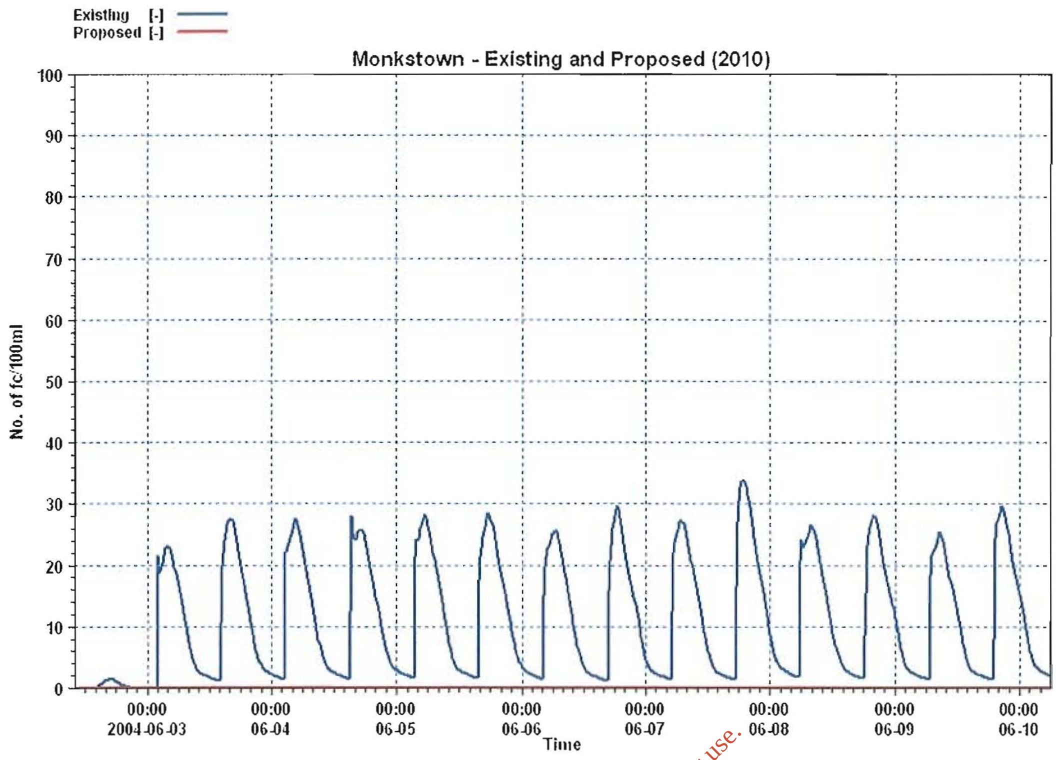

Fig. 4.30 Monkstown Creek – Repeating Spring Tide

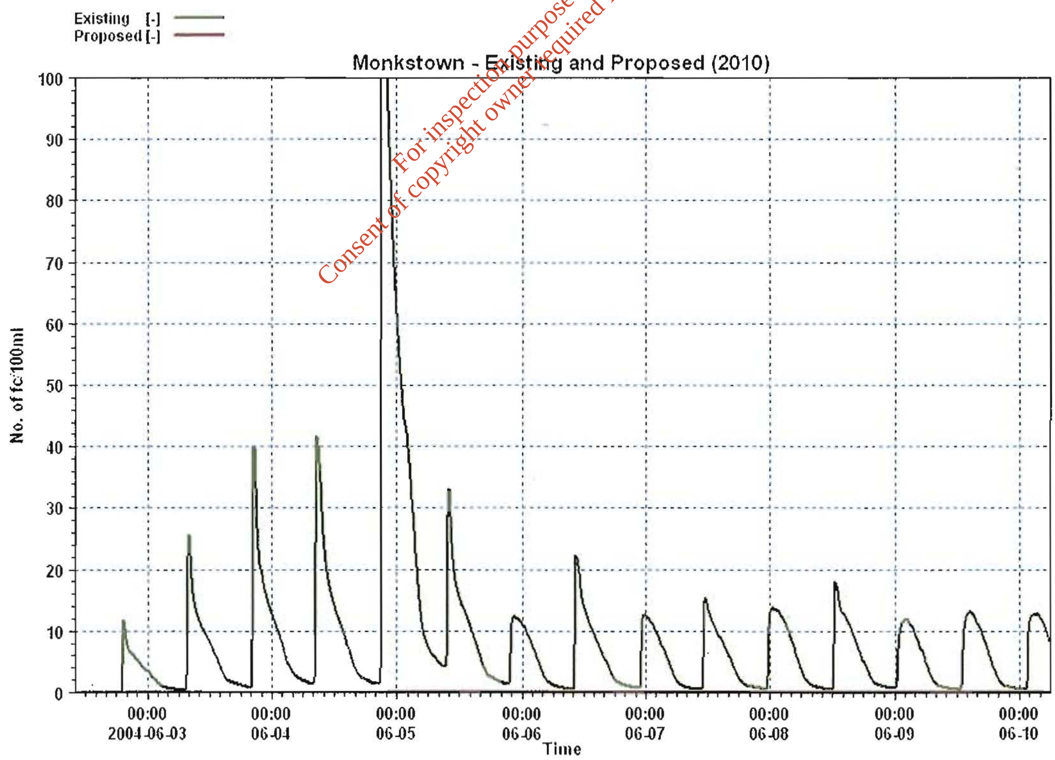

Fig. 4.31 Monkstown Creek – Repeating Neap Tide

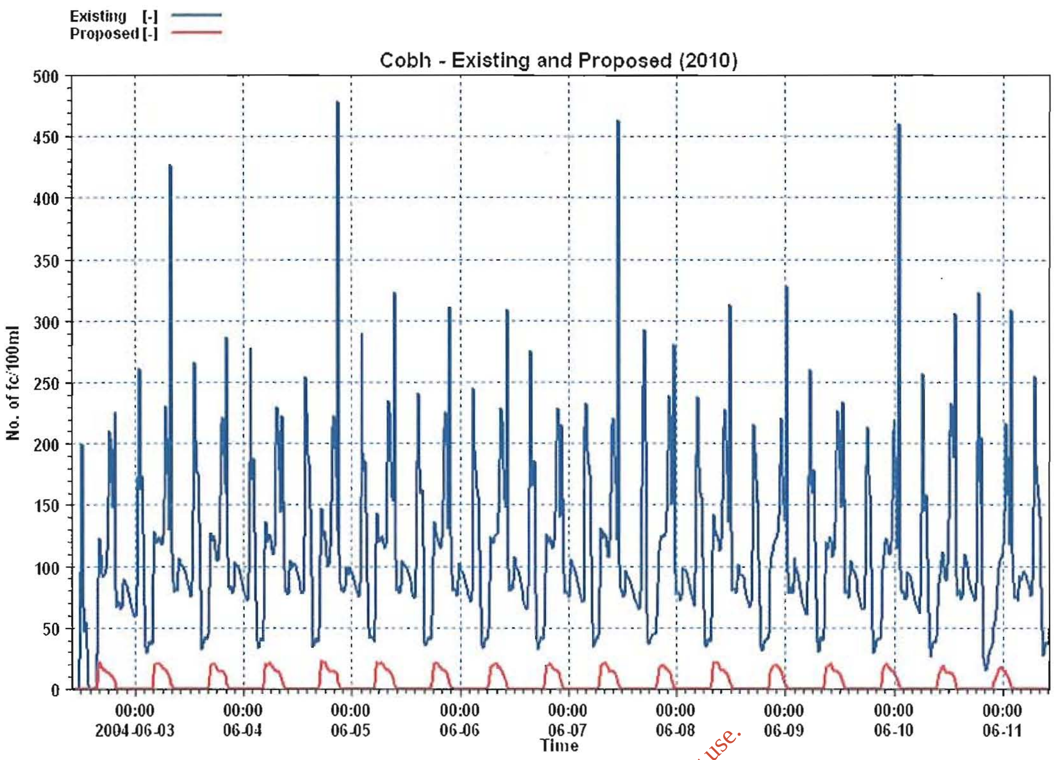

Fig. 4.32 Cobh – Repeating Spring Tide

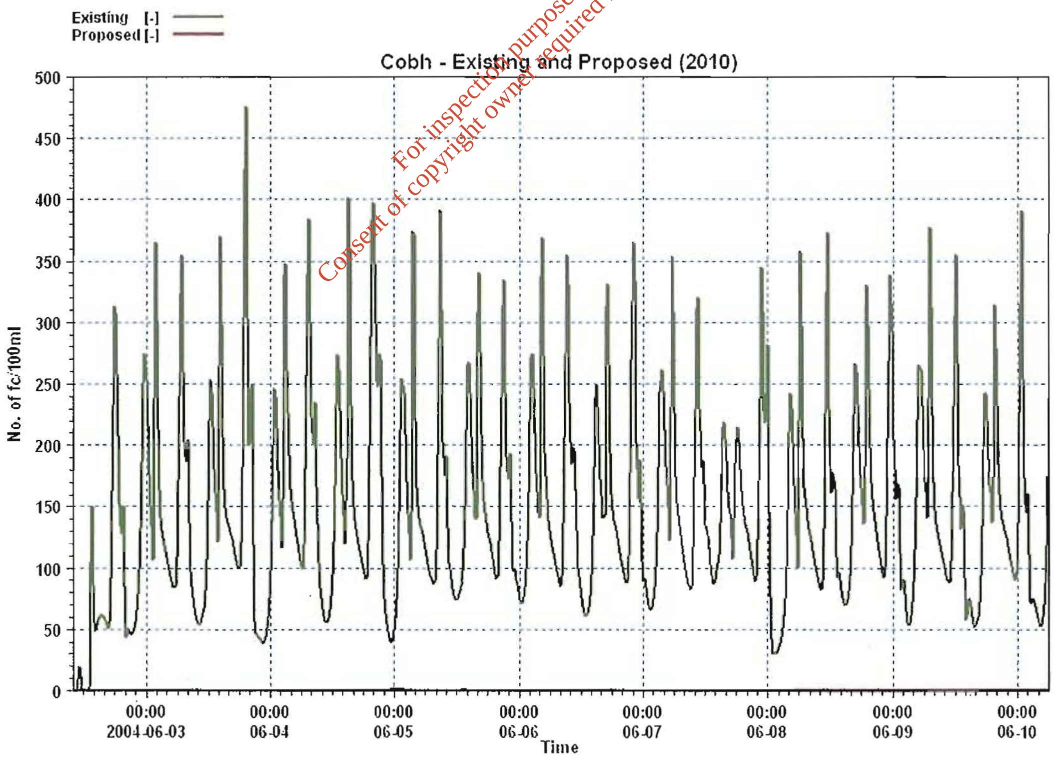

Fig. 4.33 Cobh – Repeating Neap Tide

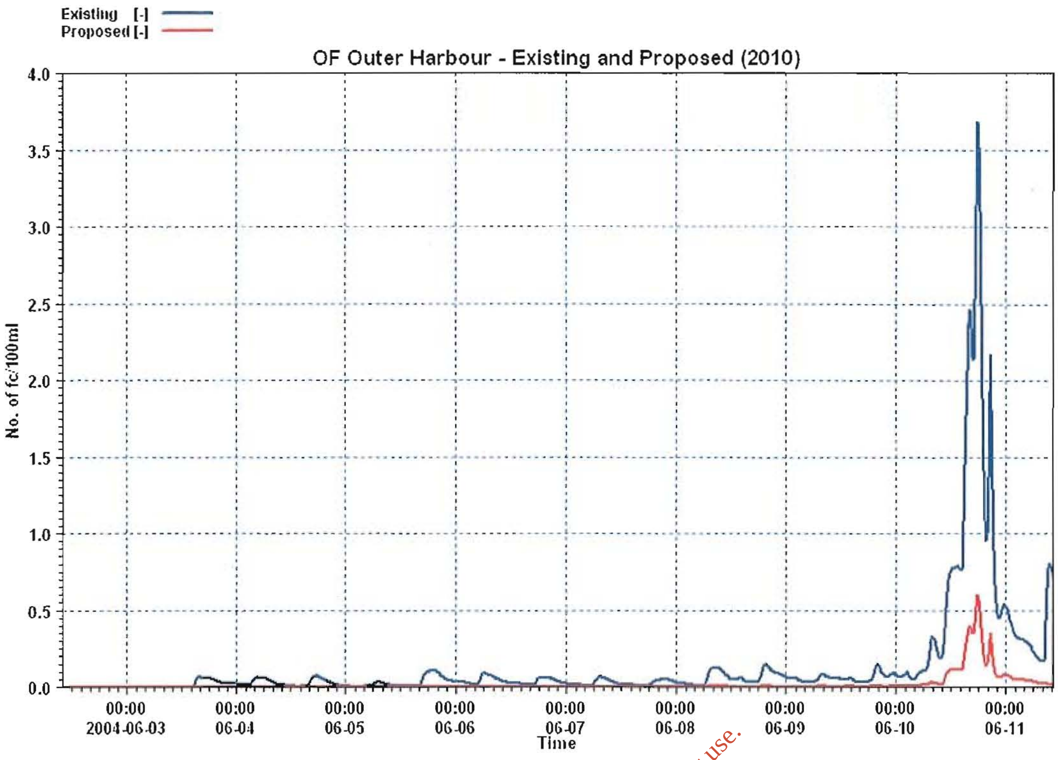

Fig. 4.34 OF - Outer Harbour – Repeating Spring Tide

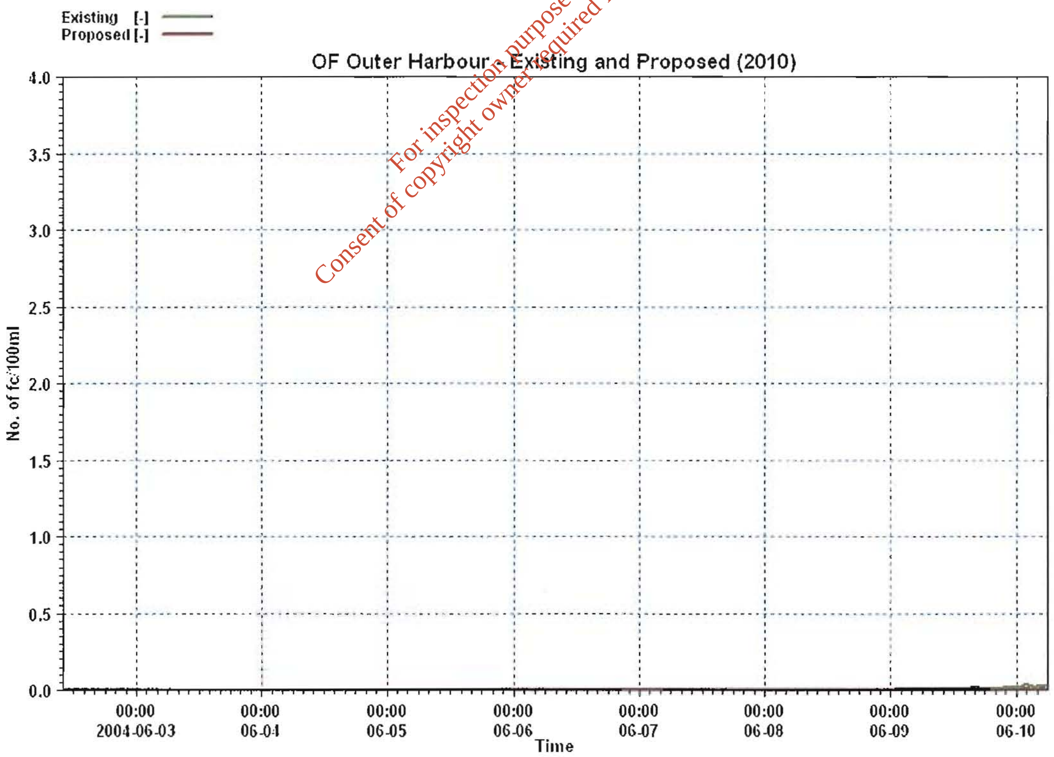

Fig. 4.35 OF - Outer Harbour – Repeating Neap Tide

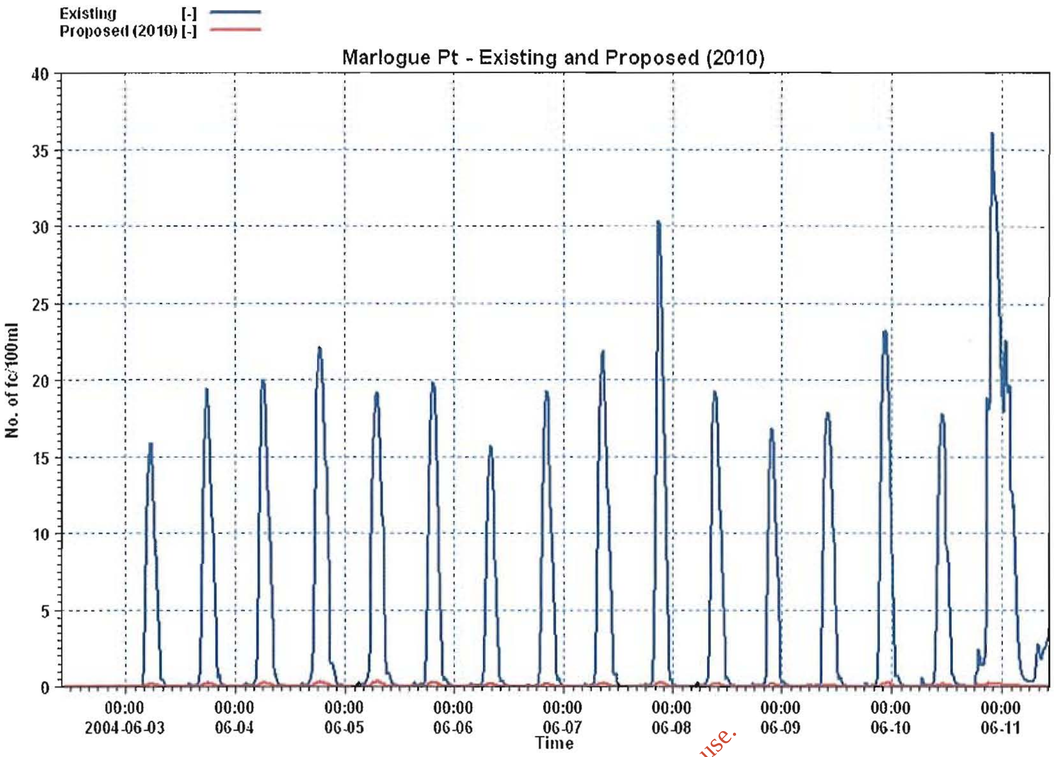

Fig. 4.36 Marlogue Point – Repeating Spring Tide

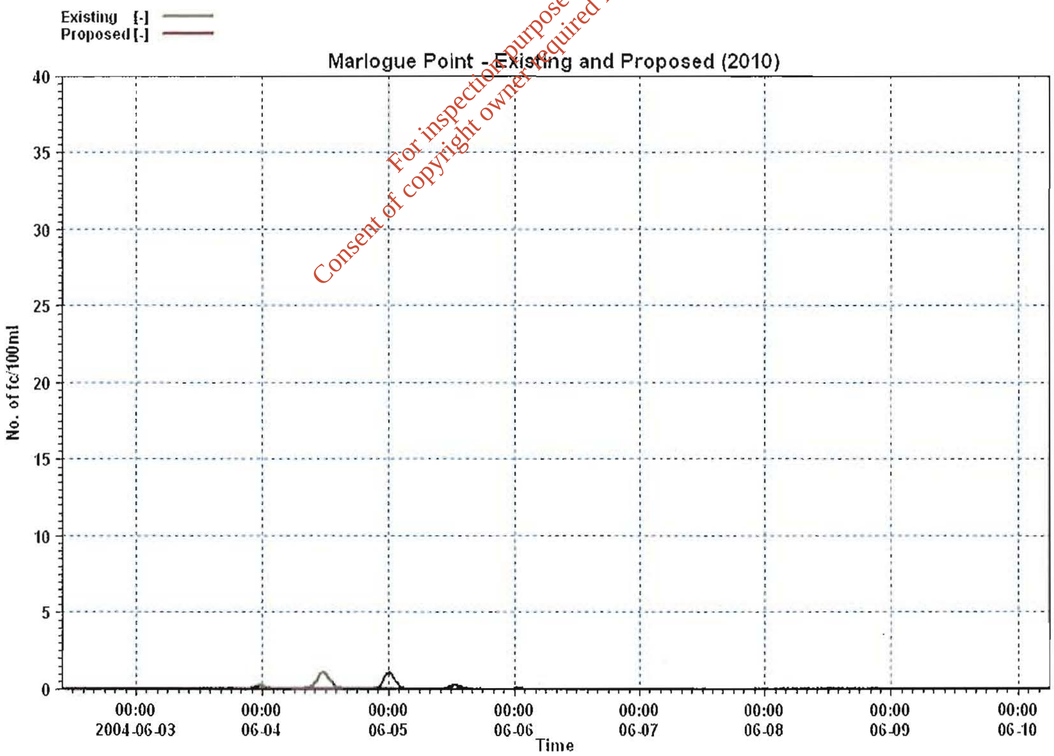

Fig. 4.37 Marlogue Point – Repeating Neap Tide

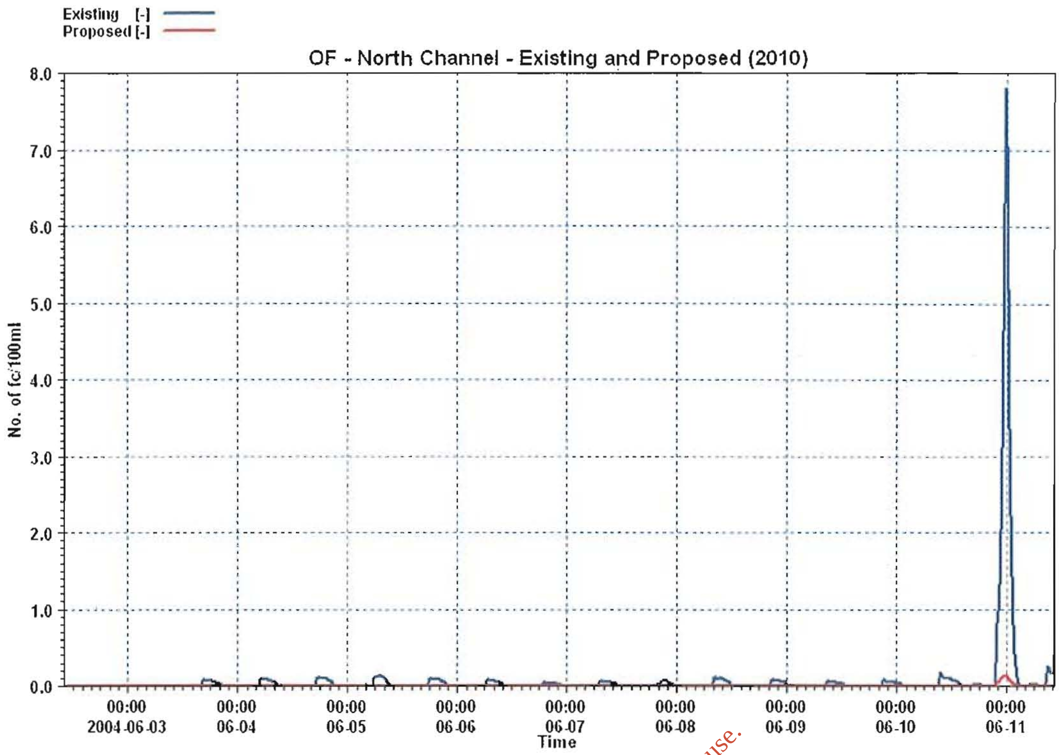

Fig. 4.38 OF – North Channel – Repeating Spring Tide

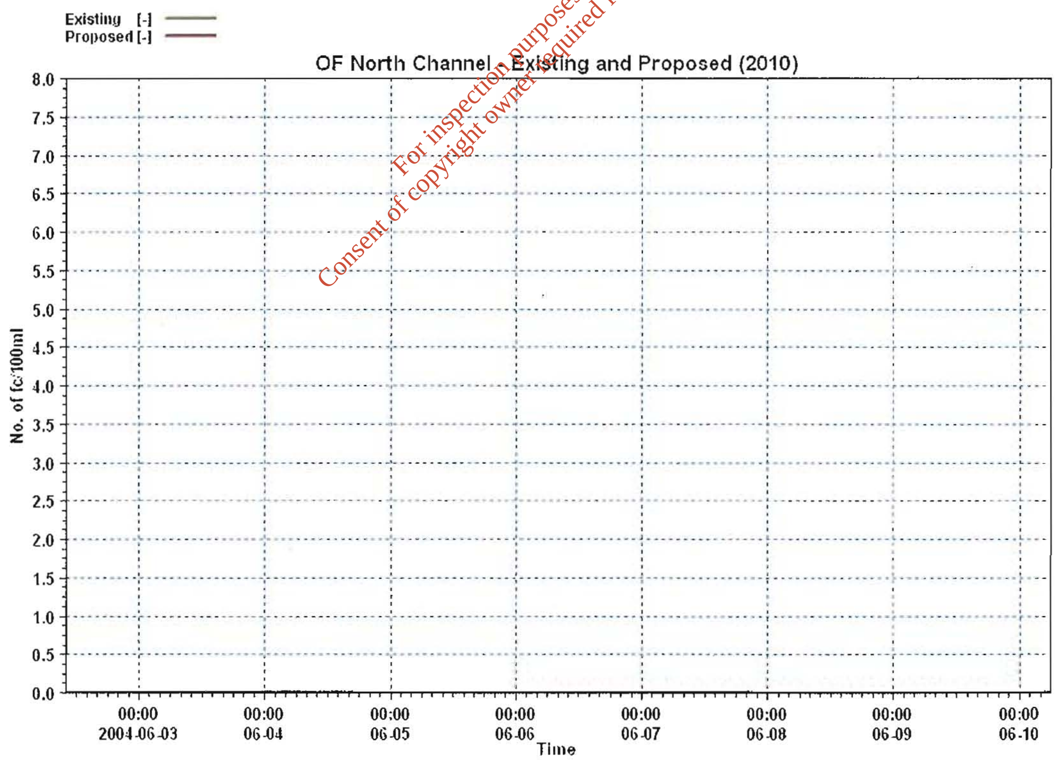

Fig. 4.39 OF – North Channel – Repeating Neap Tide

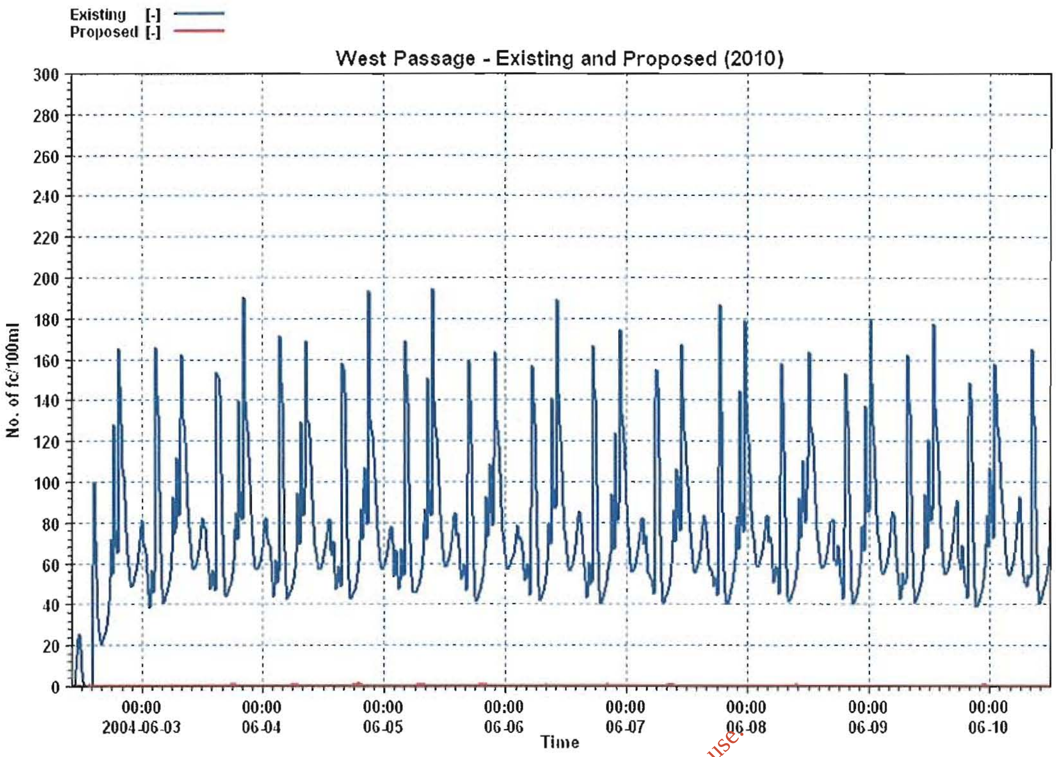

Fig. 4.40 West Passage – Repeating Spring Tide

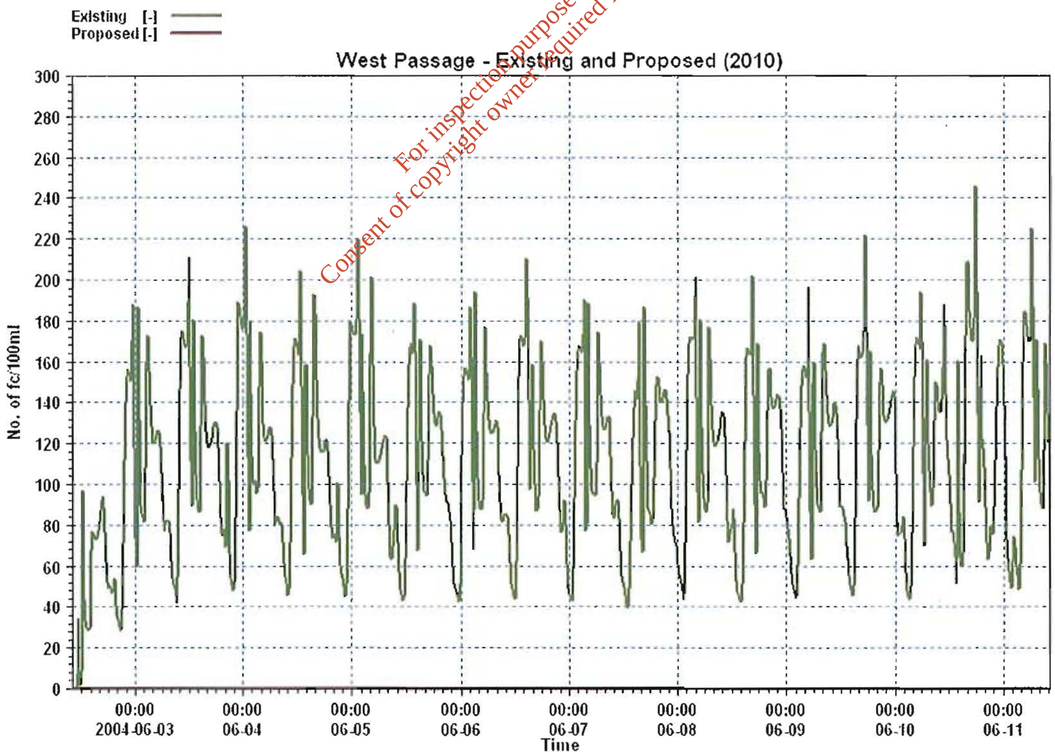

Fig. 4.41 West Passage – Repeating Neap Tide

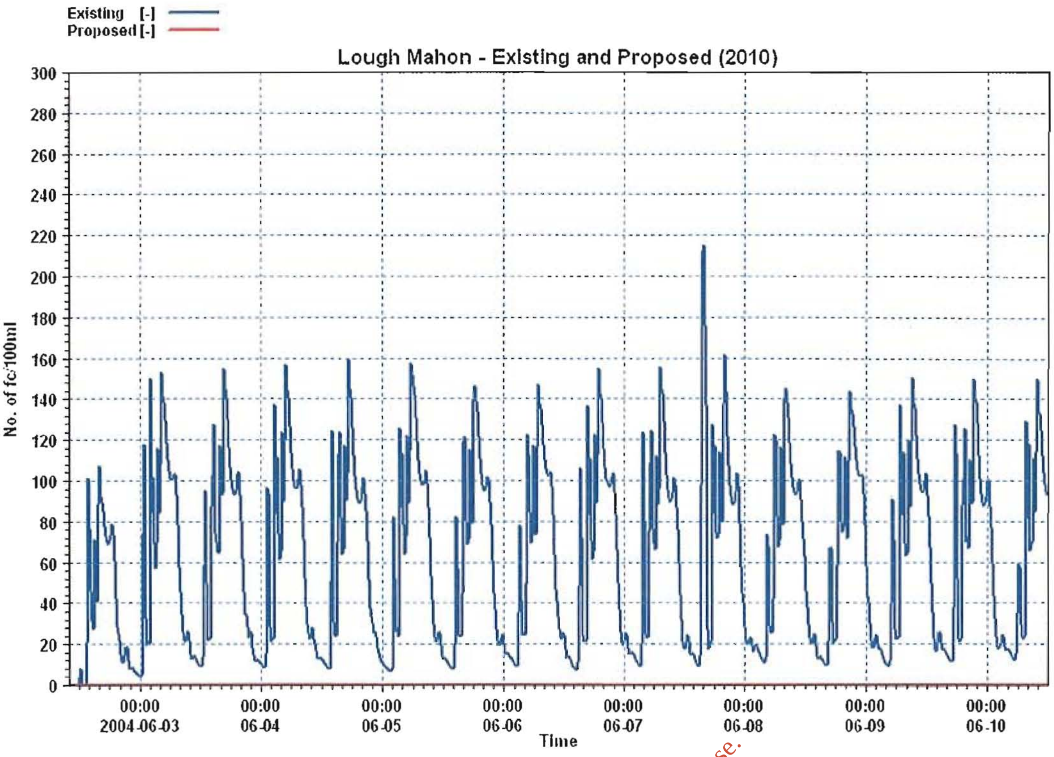

Fig. 4.42 Lough Mahon - Repeating Spring Tide

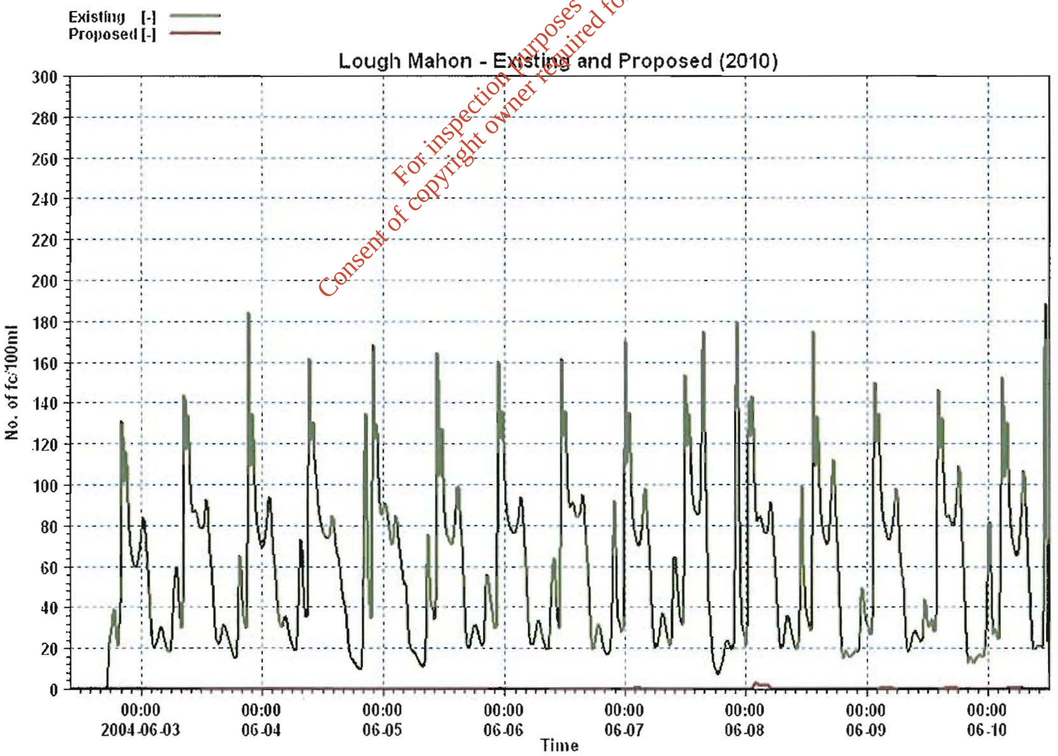

Fig. 4.43 Lough Mahon - Repeating Neap Tide