ENVIRONMENTAL IMPACT STATEMENT

for the

Clonakilty Wastewater Treatment Plant Upgrade

at

Clonakilty, Co. Cork

September 2006

VOLUME II - Appendices

Prepared by:

White Young Green Ireland Ltd.
Unit 2, University Technology Centre
Curraheen Road
Cork

APPENDIX 6.1

Table 6.1 Surface Water Quality Data (SW1) Feagle River

River:	**Fealge**								
Sample Location	**Bridge near square in Clonakilty (SW-1)**								
Location Easting	137698								
Location Northing	41306								
	Max Limit	25	0.5	0.05	0.03	-	15	150	9
	Min Limit						5	50	6
Sample Reference	**Sample Date**	**Nitrate (NO3)**	**Ammonium (NH4)**	**Nitrite (NO2)**	**Molybdate Reactive Phosphorous (P)**	**Temperature**	**Dissolved Oxygen (O2)**	**Dissolved Oxygen % Saturation**	**pH**
		mg/l	**mg/l**	**mg/l**	**mg/l**	**Degrees C**	**mg/l**	**% O2**	
2004/0163	05-Feb-04	**31.9**	0.06	0.046	**0.031**				
2004/0348	10-Mar-04	**26.12**	< 0.026	0.021	0.021				
2004/0518	01-Apr-04	**31.41**	0.054	0.031	0.022	10	10	123	
2004/0740	06-Apr-04	**28.43**	**1.115**	**0.081**	**0.041**	10.3	12.3	110	
2004/1035	03-Jun-04	23.63	0.061	**0.125**	**0.056**	14.4	10.1	97	
2004/1307	07-Jul-04	24.17	0.031	0.037	**0.055**	15.5	10.9	106	
2004/1504	04-Aug-04		< 0.026	0.031	**0.034**	15.5	11.8	115	
2004/1810	02-Sep-04	23.23	< 0.026	0.02	**0.046**	15.2	10.5	104.5	
2004/2074	06-Oct-04	**28.17**	0.036	**0.115**	**0.041**	12.4	10.7	100	
2004/2282	03-Nov-04	**33.45**	0.032	**0.055**	**0.034**	11.6	10.1	93	
2004/2474	01-Dec-04	**28.21**	0.056	**0.08**	**0.053**	9.5	10.5	92	
2005/0018	05-Jan-05	**31.41**	< 0.026	0.045	0.029	9.3	12.1	106	
2005/0368	02-Mar-05	**30.74**	< 0.026	0.032	0.02	6.5	12.4	101	
2005/0569	06-Apr-05	22.12	0.059	0.049	**0.047**				
	Sample Count	13	14	14	14	11	11	11	1
	Maximum	**33.45**	**1.115**	**0.125**	**0.056**	15.5	12.4	123	0
	Minimum	22.12	< 0.026	0.02	0.02	6.5	10	92	0
	Mean	**27.92**	0.17	**0.055**	**0.04**	11.84	11.04	104.32	7.60
	Median	28.21	0.056	0.0455	0.0375	11.6	10.7	104.5	
	Std. Deviation	3.77	0.36	0.03	0.01	3.02	0.93	9.30	

Data from Cork County Council Laboratory, Inniscarra Waterworks, Inniscarra, Co.Cork.

APPENDIX 6.2

Document Ref.: 979/1F/2005

CLONAKILTY SEWERAGE SCHEME MARINE OUTFALL STUDY

FINAL REPORT

Inchydoney Hotel and Beaches: Courtesy National Coastal Survey 2000

Submitted to:-	Prepared by:-
White Young Green, Consulting Engineers, Cork.	**Irish Hydrodata Limited,** Rathmacullig West, Ballygarvan, Co. Cork.

2nd February 2006

Document Ref.: 979/1F/2005

CLONAKILTY SEWERAGE SCHEME
MARINE OUTFALL STUDY

FINAL REPORT

Submitted to:-

White Young Green,
Consulting Engineers,
Cork.

Prepared by:-

Irish Hydrodata Limited,
Rathmacullig West,
Ballygarvan,
Co. Cork.

Tel (021) 4311255
Fax (021) 4311740

2nd February 2006

CONTENTS

1. INTRODUCTION

This report presents the findings of studies conducted to assess the impacts of treated effluent discharges from the town of Clonakilty to the sea. At present, municipal wastes discharge from an outfall located near the treatment plant adjacent to the town. Proposals for upgrading this facility are being investigated and consideration is being given to relocating the outfall further downstream. The three potential discharge locations are:

i) the existing site;
ii) a site below the proposed tidal barrage;
ii) a site near Ring Pier.

In assessing these outfall sites the requirements of the Urban Waste Water Treatment Directive, the Bathing Water Directive and the voluntary requirements of the Blue Flag Scheme were considered.

Figure 1.1 – Potential Discharge Locations.

1.1 Summary Of Study Works

Following a full review of available data and bearing in mind the objectives of the brief the following study works were undertaken:

- *various measurements in Clonakilty Harbour and the sea area off Inchydoney Beach;*
- *set up of a two dimensional circulation model of Clonakilty Harbour and the sea area off Inchydoney Beach;*
- *set up of a two dimensional dispersion model of the sea area outlined above to compare the impacts of discharges from various outfall points.*

The results of these works are described in the following sections of the report.

2. CHARACTERISTICS OF AREA

2.1 Bathymetry

The town of Clonakilty is situated some 50 kilometres southwest of Cork City and rates as a significant urban centre on the southern seaboard of Ireland. The coast is exposed to the Celtic Sea and the shoreline is fringed by low cliffs and rocky headlands interspersed with sandy beaches. Clonakilty Harbour is a shallow tidal inlet with a surface area of approximately 2 km^2. The Fealge river enters the harbour at the northern end. It has an estimated mean flow of 0.9m^3/s and a 95 percentile low flow of 0.075m^3/s. These flows were derived by scaling data (factor based on catchment areas) from the nearby Argideen river, which has been gauged since 1977.

Information relating to water depths and general bathymetric features of the wider area were obtained from Admiralty Chart No.1765 [1]. These were supplemented by local bathymetric surveys conducted within the harbour during the course of a previous study [2]. The harbour bathymetry is indicated in Figure 2.1. At low tide the majority of the harbour empties of water and Figure 2.2 compares low tides on spring and neaps.

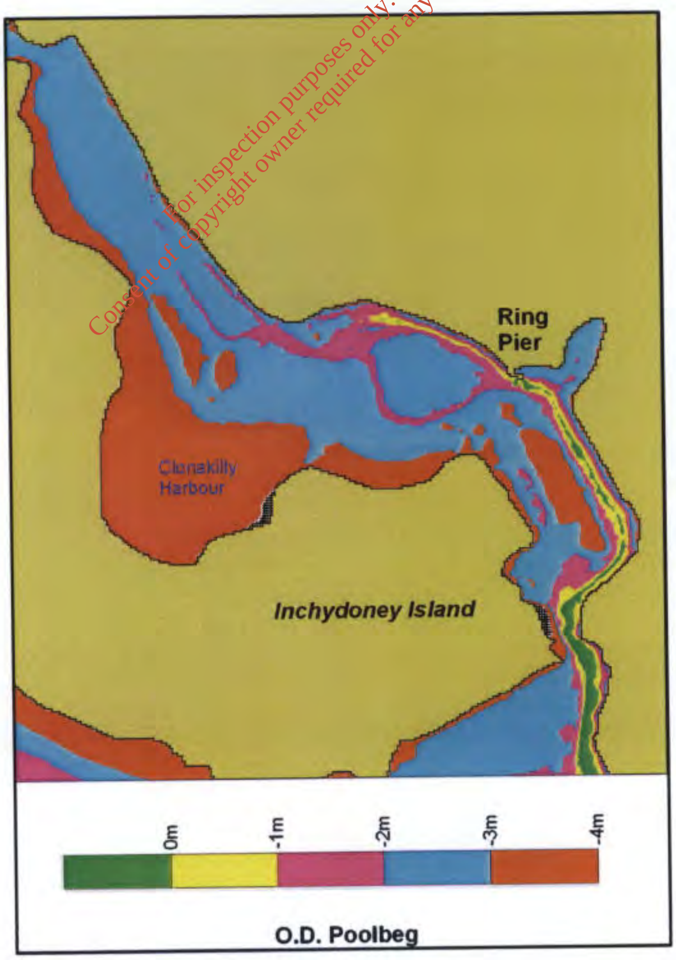


Figure 2.1 – Harbour Bathymetry.

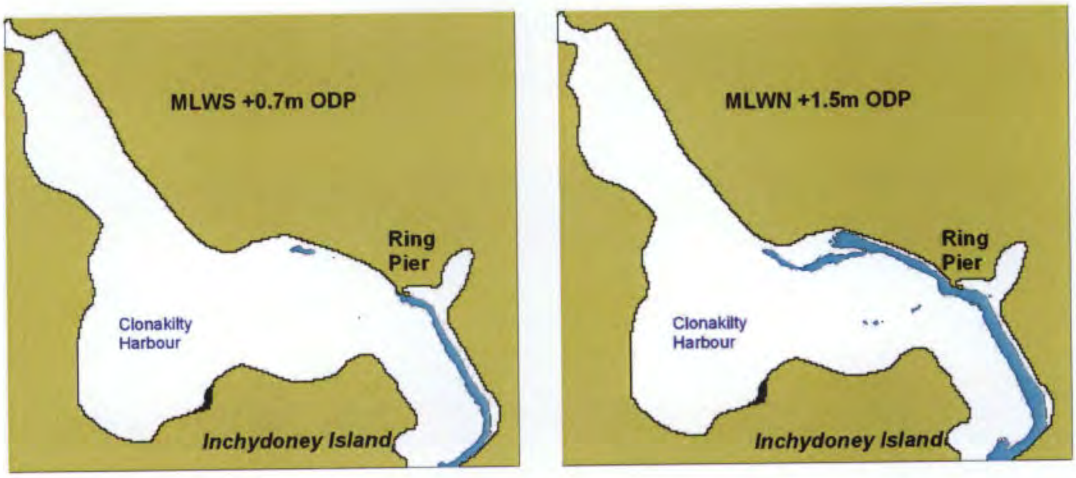


Figure 2.2 – Water Areas at Low Water Spring and Low Water Neap Tide.

2.2 Tide Levels

The tidal regime off the south coast is semidiurnal with two high waters (HW) and two low waters (LW) each day (24.8 hours) [3]. Tidal statistics, based on measurements made at the Wind Rock near the entrance to the harbour [2], are summarised in Table 2.1.

Tide Level	Level to OD Poolbeg
Mean High Water Spring Tide	4.1m
Mean High Water Neap Tide	3.3m
Mean Tide Level	2.4m
Mean Low Water Neap Tide	1.5m
Mean Low Water Spring Tide	0.7m

Table 2.1 - Tidal Levels at Wind Rock.

2.3 Coastal Tidal Streams and Currents

The general circulation patterns of the Celtic Sea have been well documented through the years. Admiralty charts and tidal atlases [1,4] provide an indication of the current patterns. The offshore tidal currents generally run parallel to the coast. Speeds are relatively low reaching maximum values of about 1.0 knots (.5m/s) during spring tides. Off Inchydoney the circulation patterns are less pronounced with weaker speeds and more varied directions.

3. FIELD STUDIES METHODOLOGY

Survey measurements were made to obtain sufficient information for calibration of numerical models. The various activities are summarised briefly below.

3.1 Current Metering

Two current meters were deployed at mid depth on U-moorings outside the Harbour to record water movements over several tides. The units, Interocean S4's, recorded speed and direction data at 10 minute intervals. The mooring locations are indicated in Figure 3.1.

3.2 Tide Gauge

A recording tide was deployed at Ring pier for the duration of the current metering works.

3.3 Drogue Tracking

Cruciform shaped drogues were released from various locations within Clonakilty Harbour on three dates to establish the general water movements. These drogues were set to track the water mass at a depth of 0.5m below the water surface. Positions were recorded by steaming the survey launch close to the drogue at frequent intervals and recordings its position with an onboard DGPS unit and HYPACK software.

3.4 Dye Releases

The effects of the proposed treated effluent discharges were simulated by releasing Rhodamine WT tracer dye at different locations on four occasions. The dye was diluted with water to provide a patch which would mix readily with the surrounding waters in the manner of municipal effluent. The volume of dye released varied from 200ml to 500ml depending on the location. Tracking was accomplished with a continuously recording Cyclops 7 fluorometer towed 0.5m below the water surface. This unit measures the increased water fluorescence resulting from the dye slug. As the patch dilutes fluorescence levels decrease and eventually reach the natural background of water.

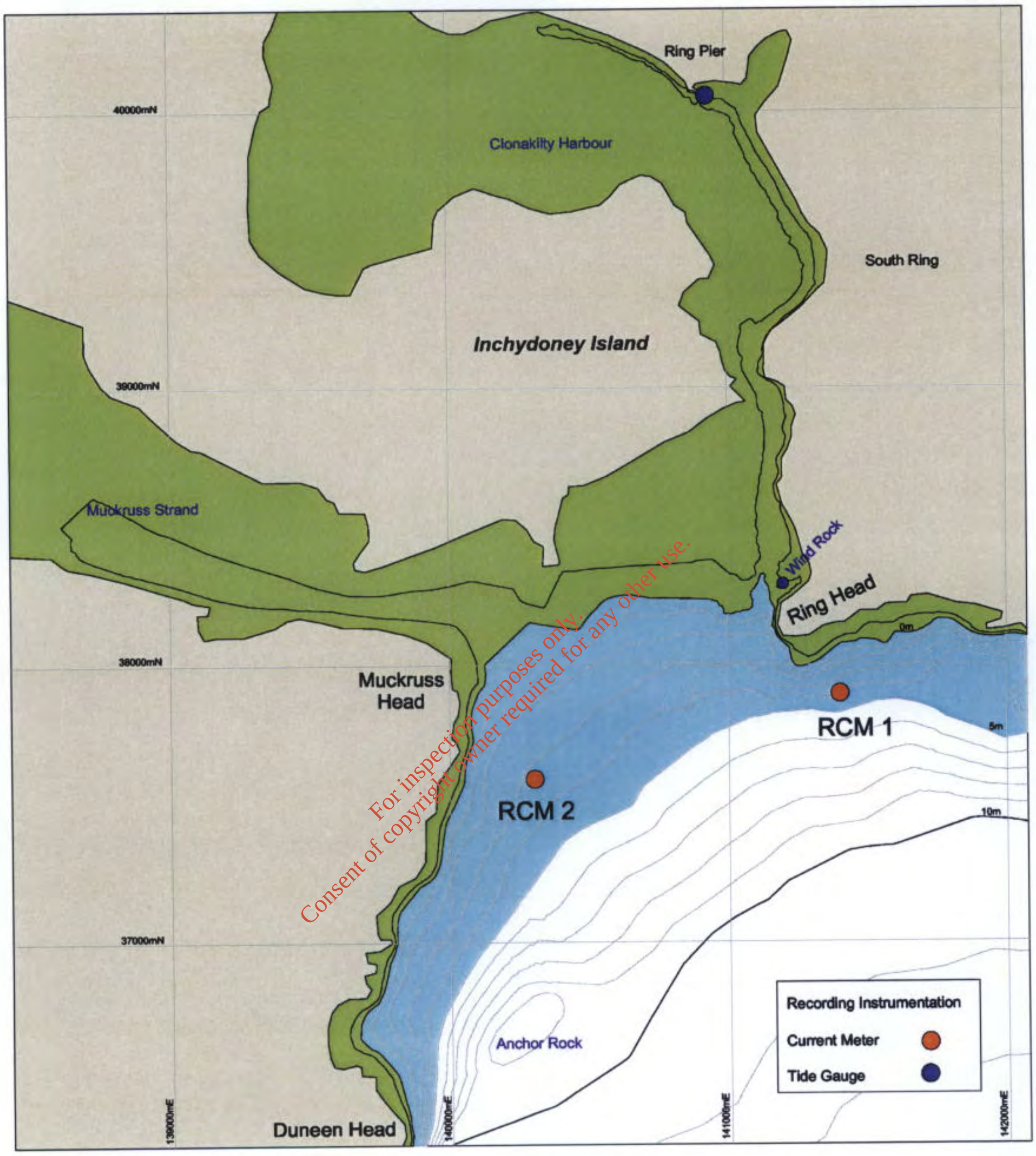


Figure 3.1 – Recording Instrument Locations.

4. FIELD SURVEY RESULTS

4.1 Current & Tide Measurements

Current speed and direction data recorded at Sites 1 and 2 are presented in Figure 4.1. The meter at Site 1 logged data for 4 days while that at Site 2 stopped after 2 days due to a faulty battery cell. Peak speeds reached approx 0.14m/s at both sites. Directions were variable at both sites, indicating a significant wind impact. Directional distributions of the data are presented in Figure 4.2. At Site 1 the current has an east west bias while at Site 2 the northerly flood direction dominates. Tidal levels during the deployment are included in the figure. The tidal ranges for the deployment correspond to mean springs.

4.2 Drogue & Dye Tracking

Day No.1 - 5/12/05

Two surface drogues and a 200ml slug of dye were released at approx 1 hour after high water (HW+1h) from a point just upstream of Ring Pier. Both dye and drogues moved rapidly seawards and by HW+1.75h had passed Wind Rock and were travelling southwards. A further drogue was released in the channel off the eastern tip of Inchydoney Island at HW+1.5h.

Drogue trajectories are indicated in Figure 4.2a. At the time of recovery, approx HW+4h, the three drogues were about 1km south of Ring Head and were spread out over a distance of 700m.

The dye patch trajectory was very similar to that of the drogues, passing Ring Head at HW+2h and becoming widely dispersed at HW+3.5h as shown in Figure 4.3b.

Weather conditions were good on the day with light westerly winds (1m/s) for the duration of the track. Tidal conditions corresponded to a spring tide.

Day No.2 - 6/12/05

For this release two surface drogues and a 200ml dye slug were again released at a point upstream of Ring Pier at the slightly later time of HW+2.5h.

The two drogues travelled quickly downstream, one exited the harbour and continued to the south southwest while the other passed to the east of Wind Rock and grounded. Two further drogue releases took place in mid-channel at Wind Rock. One release was made at HW+3.25h, the drogue travelling southwards and then to the southeast before recovery at HW+6h, and the other at LW-1h (Low Water -1h). On the latter release two drogues were deployed. Initially both travelled to the south before swinging to the east. One drogue went aground on the rocks below Ring Head while the other drogue was recovered at LW+1.5h still travelling to the east (Figure 4.4a).

Two dye releases were made in conjunction with the drogue tracking (Figure 4.4b). The first release took place at HW+2.5h off Ring Pier and the second at LW-1h off Wind Rock. The first patch followed the channel downstream from Ring Pier and as it passed Ring Head continued to the south-southwest. Concentrations were low at this time (HW+4.5h) and just above background. The patch from the second release, made at 1 hour before low water, was lying to the south of Ring Head at low water and it continued to move to the east and disperse. The last track was at LW+1.5h by which time concentrations were just above background levels.

Weather conditions were good on the day with light north westerly winds (1m/s) for the duration of the track. Tidal conditions corresponded to a spring tide.

Day No.3 - 13/12/05

These dye and drogue track exercises were undertaken in the outer bay to obtain information on trajectories around low water. Results are presented in Figure 4.5a & 4.5b. The drogues/dye slug were released at LW-2.25h and tracked for 4.25 hours until LW+2h. During that time they travelled first to the south and later around LW+1h began to move to the east.

Weather conditions were good on the day with light north westerly winds (1.5m/s) for the duration of the track. Tidal conditions corresponded to a spring tide.

Day No.4 - 14/12/05

On this occasion a slug of 0.5l of dye was released at the existing outfall location at HW+3h. Dye concentrations were monitored at Ring Pier and the dye slug arrived at this location at HW+6h, approx 3 hours after release. The time series of dye concentration is shown in Figure 4.6. The river discharge on this date was estimated to be 0.5m^3/s.

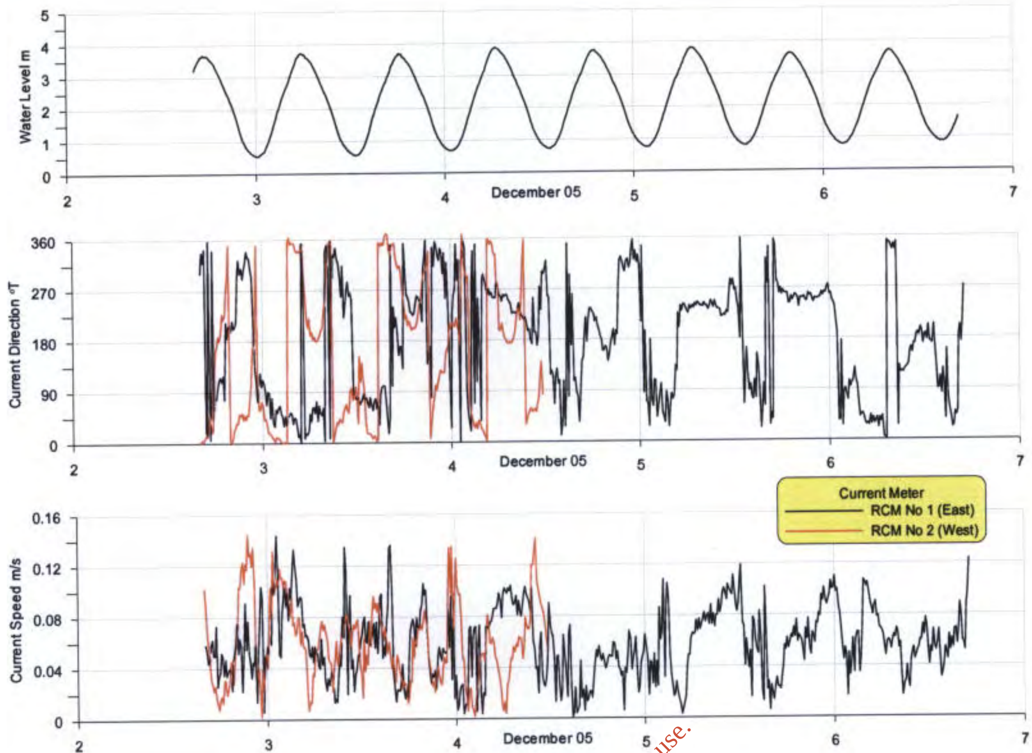


Figure 4.1 - Current and Tide Gauge Time Series.

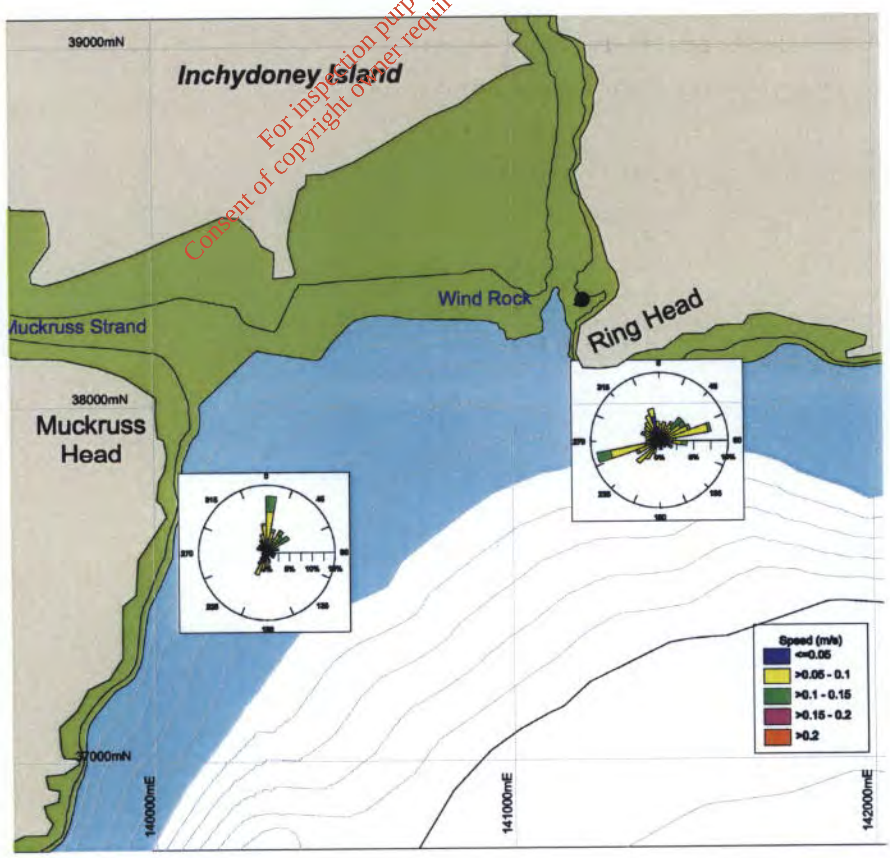


Figure 4.2

Current Rose Data.

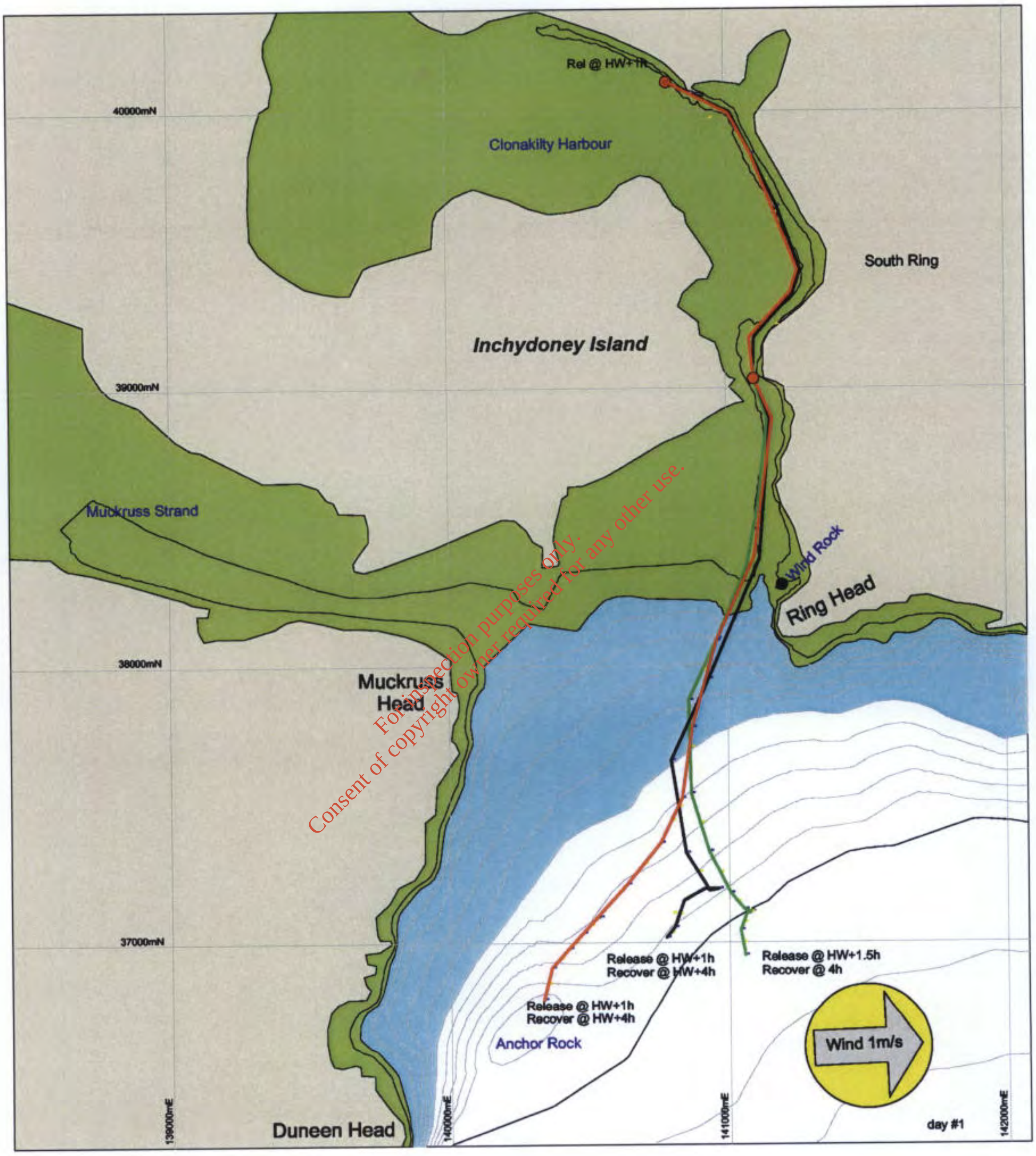


Figure 4.3a – Drogue Track, Day No 1.

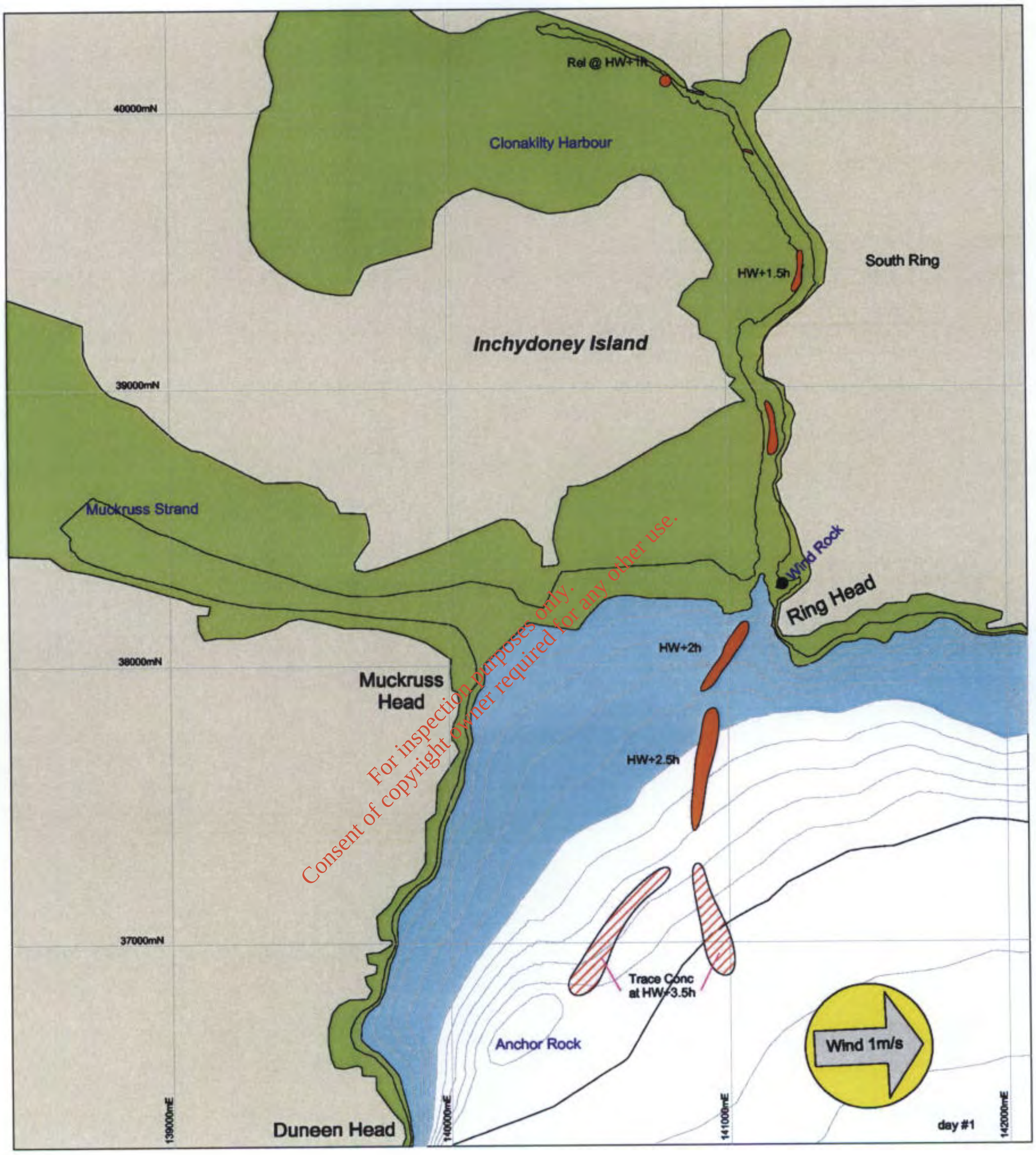


Figure 4.3b – Dye Track, Day No 1.

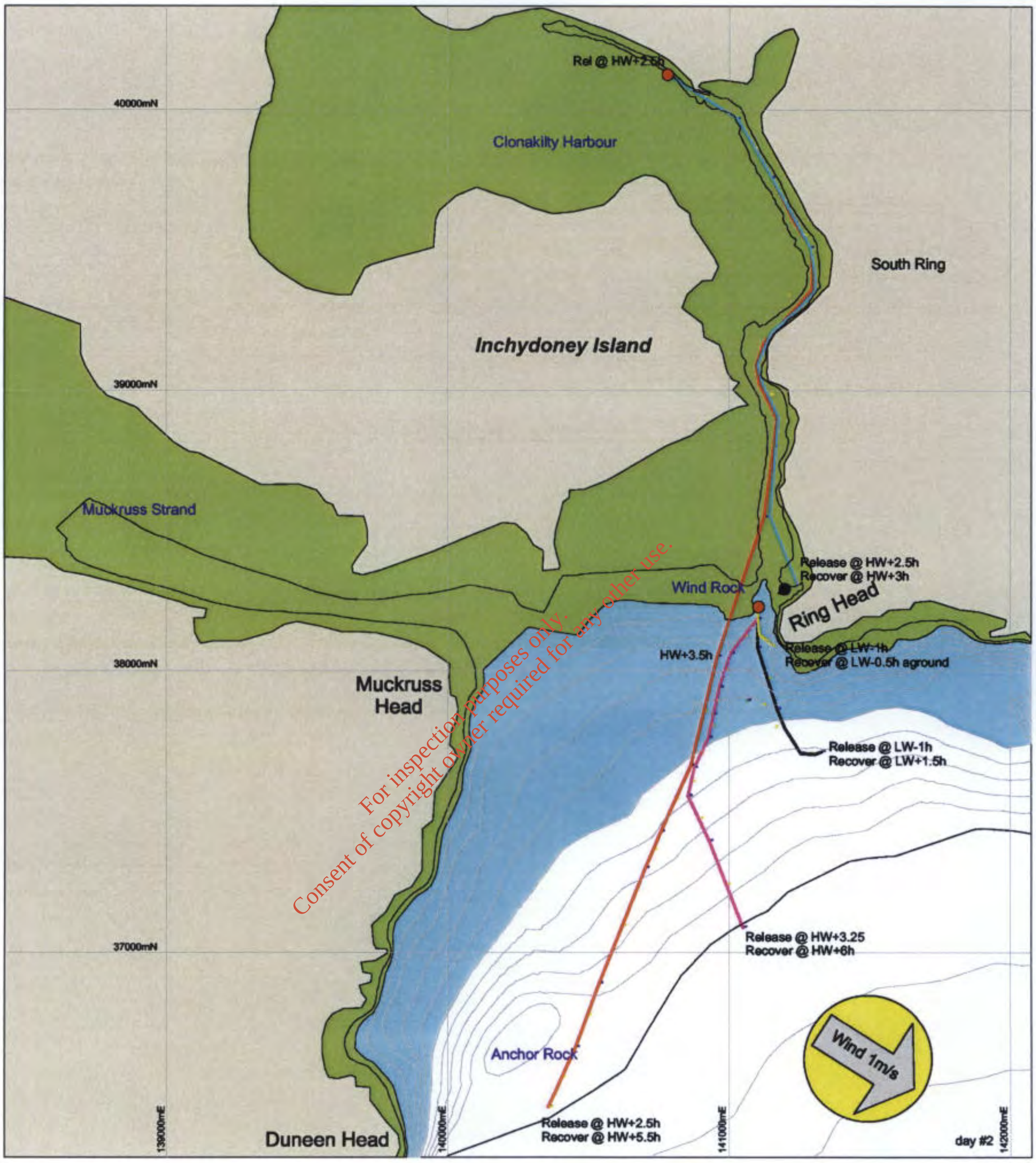


Figure 4.4a – Drogue Track, Day No 2.

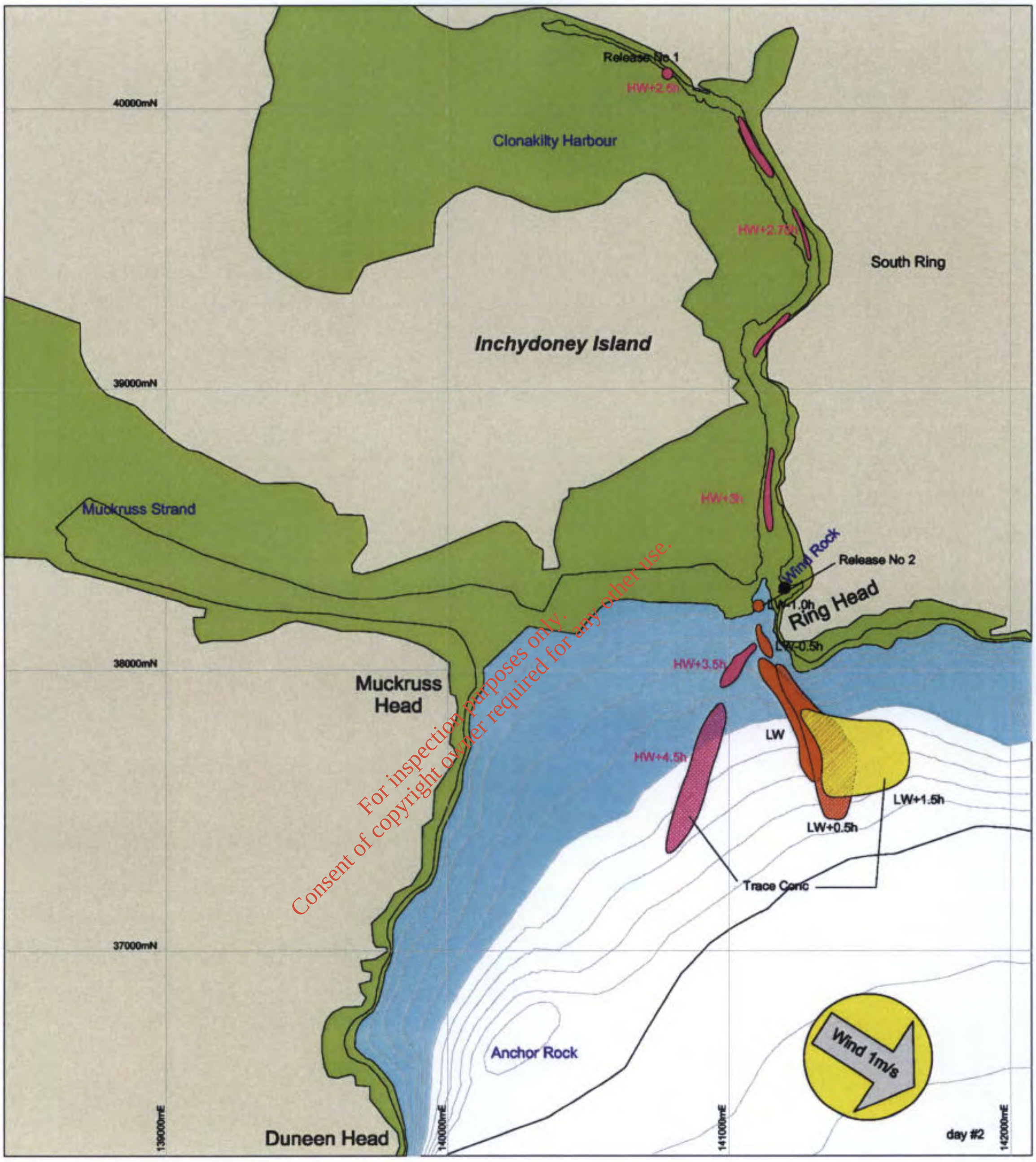


Figure 4.4b – Dye Track, Day No 2.

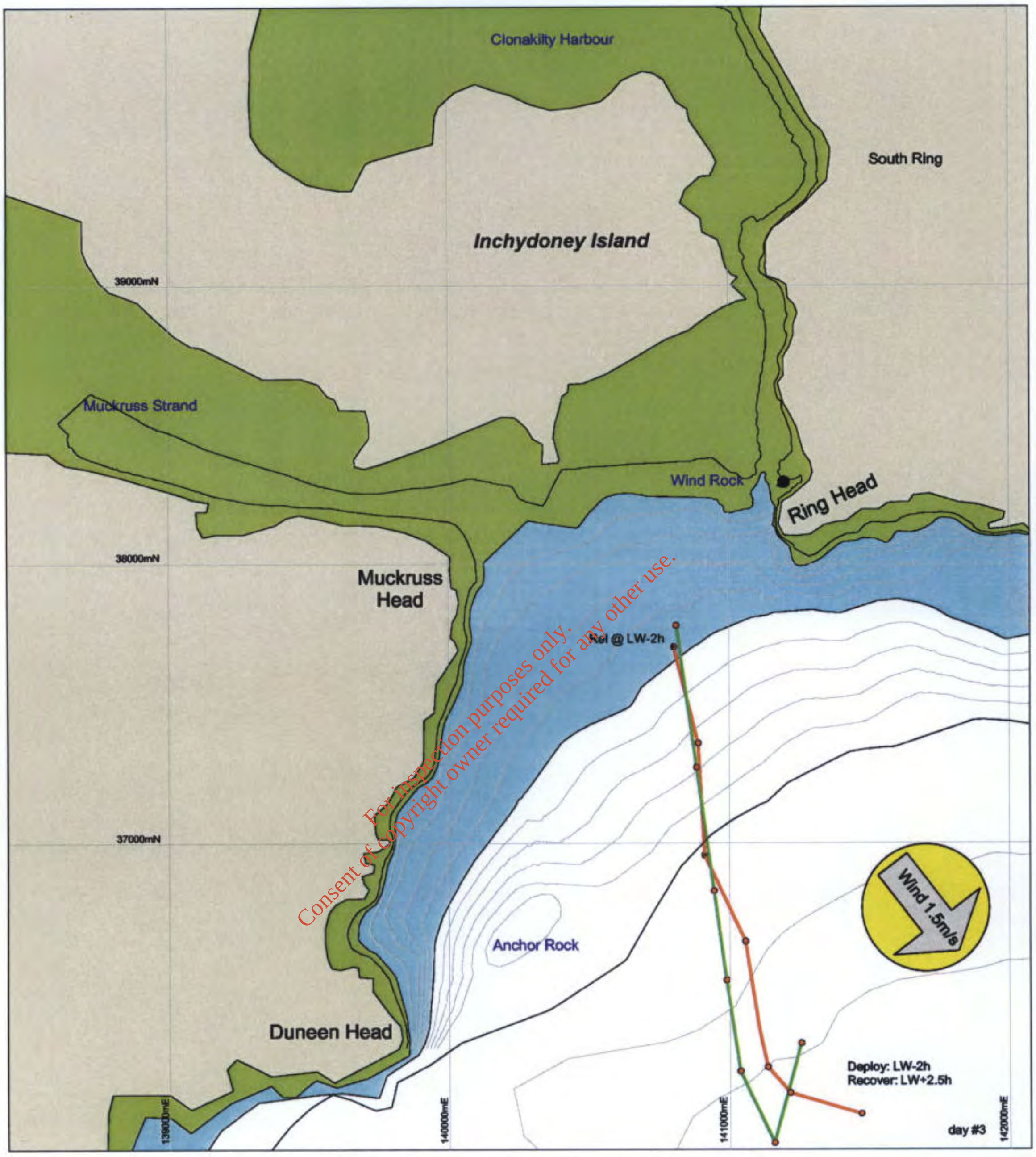


Figure 4.5a – Drogue Track, Day No 3.

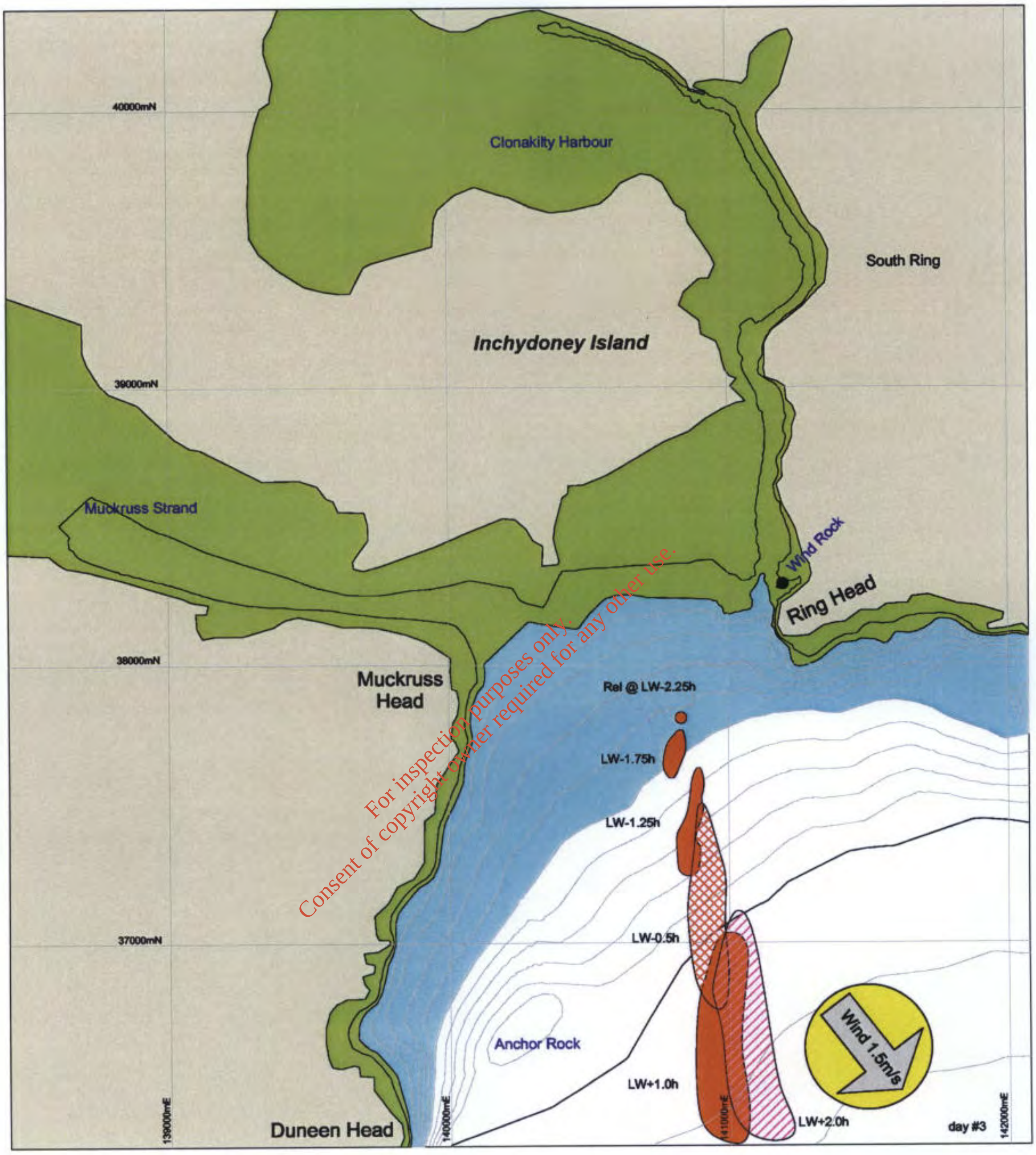


Figure 4.5b – Dye Track, Day No 3.

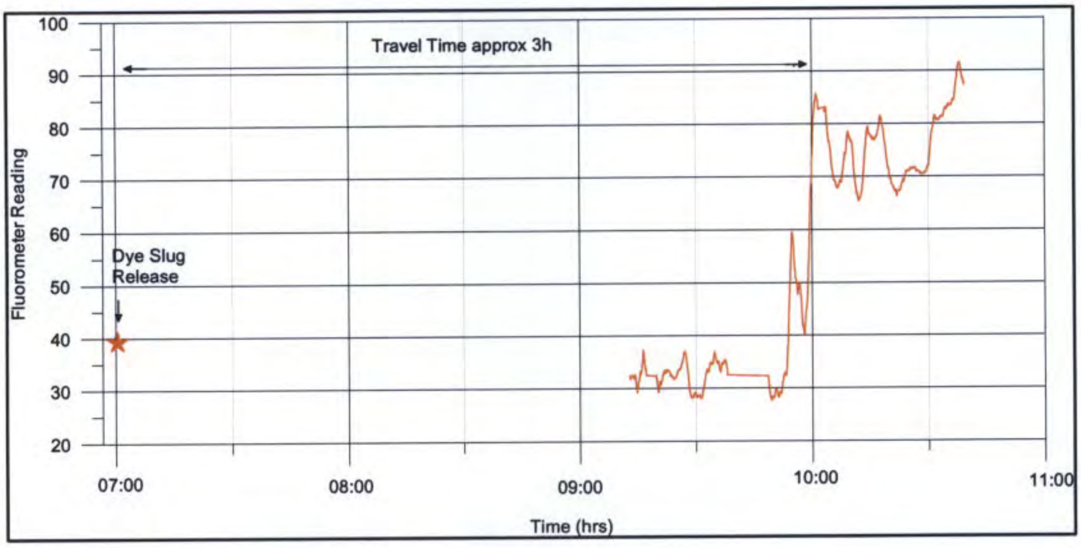


Figure 4.6 – Dye Track, Day No 4.

5 PREDICTIVE MODEL OF WATER QUALITY

5.1 Modelling Approach and Methods

A two dimensional depth averaged flow model (M2D, [5]) was used to simulate the tidal circulation in the study area and provide an hourly flow pattern for both the spring and neap tidal cycles. The effects of wind were included in terms of enhanced horizontal mixing.

A particle track model was used for predicting the effluent dispersion patterns (TRACK, [6]). With this technique, a cloud of discrete particles simulates the continuum of dispersing contaminant. The model operates on the same grid as that employed in the flow model.

5.2 2d - Flow Model

In the 2D circulation model the bathymetry was defined on a rectangular grid with cells of horizontal dimension 10m x 10m. The model included both Clonakilty Harbour and Muckruss Strand and extended seawards beyond Ring Head for a distance of approx. 2km. The large model area was necessary to ensure model boundary effects did not adversely impact the flows in the region of interest and that water exchanges were consistent with estuary volumes.

Boundary conditions were initially taken from co-tidal charts [7] and other references. These indicate spring tidal amplitudes of approximately 3.4m and a small phase difference along the southern boundary. Several model runs were then conducted for a range of boundary parameters to obtain simulations equivalent to mean neap and mean spring tides.

Calibration of the model was first achieved by comparing predicted current speeds with field measurements and adjusting model coefficients as required Further calibration was achieved by simulating drogue releases with the model and comparing results to those observed in the field.

Simulated surface drogue trajectories corresponding to the spring tide release of 5/12/05 (Figure 4.3a) are shown in Figures 5.1. Excursions and trajectories are simulated to a good accuracy.

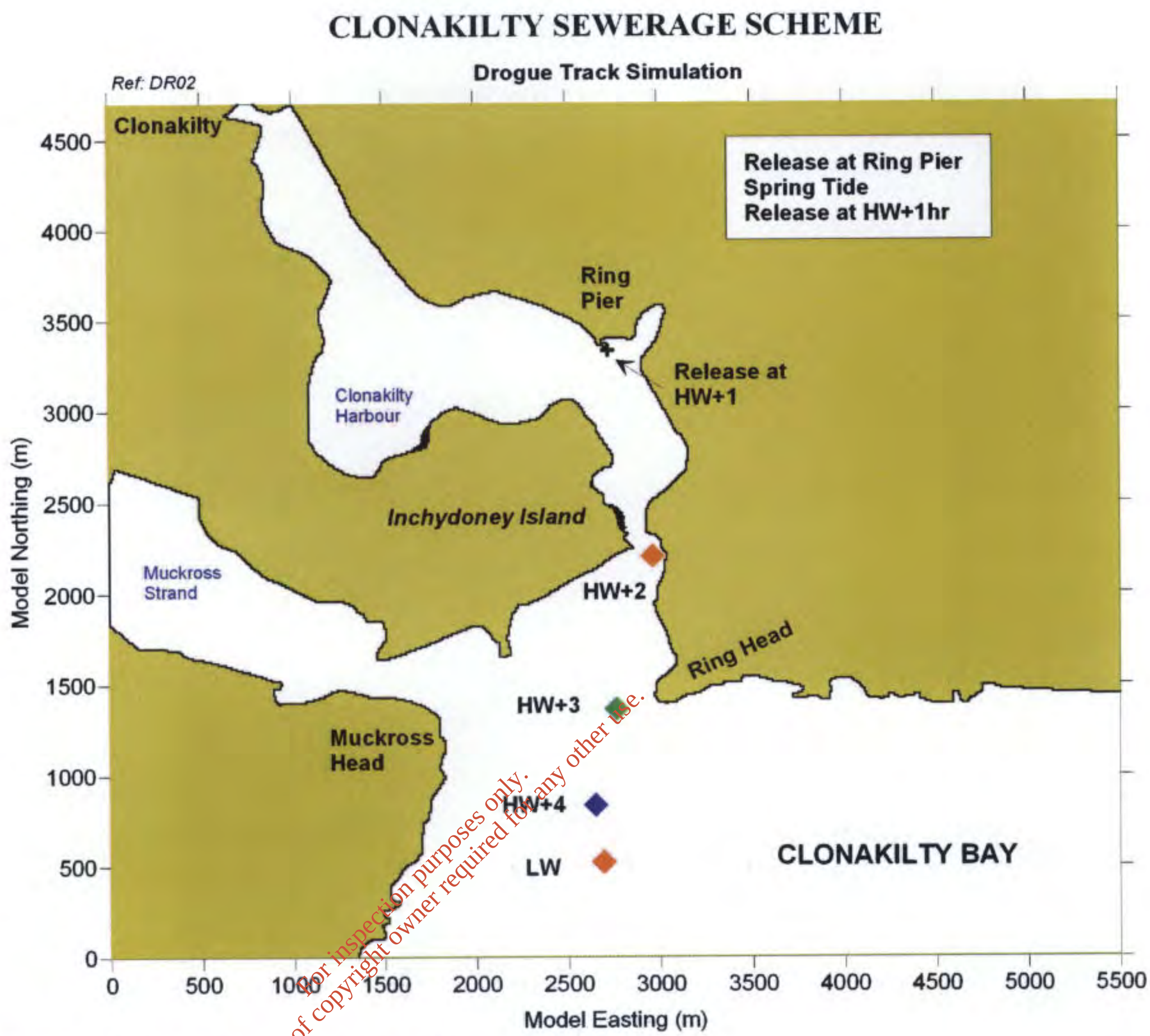


Figure 1.1 – Simulated Drogue Track

5.3 2d - Dispersion Model

Dispersion was simulated using the particle tracking model TRACK. In the model the discharge of effluent material is represented by a number of discrete particles. As the simulation progresses through time a series of particles are released at the outfall location. During each time step the particles are moved horizontally by the current flows. In addition to these advective steps, each particle is moved by random steps in order to simulate the effects of diffusion. The particle step length which simulated diffusion in the models was selected randomly in the range +/- infinity according to an appropriate Gaussian probability density function.

In shallow coastal waters dispersion results from a combination of physical mechanisms. These principally relate to the current and the manner in which it varies both vertically and laterally. The greater the 'velocity shear' the more rapid will be the dilution of the effluent. The horizontal diffusion coefficient was estimated from the dye test results. The effect of wind is to promote more rapid mixing and this was simulated by an increased diffusion coefficient.

The process of bacterial decay was included in the model by evaluation of the probability of decay for each particle during each time step. This was expressed as a function of T_{90} where T_{90} is the time for 90 percent decay. In the simulations produced for this study, the decay time was defined to be 12 hours and so the results achieved equilibrium within 10% after one tide and within 1% after two tides.

The simulations made by the model were of 24.8 hours duration (two tides) using a time step of 20 seconds.

Concentrations of effluent were estimated by counting the number of particles in each model grid cell (10m x 10m). This produced the number of model particles in a volume of water which was determined by the horizontal cell dimensions and the water depth at that point. If the water depth exceeded 5m then the vertical dimension was set to 5m to allow for incomplete vertical mixing as sometimes occurs in coastal/estuary waters

Verification of the combined flow/particle track dispersion model was achieved by comparing simulated drogue tracks with field data and similarly dye patch releases with dye track data.

6. SIMULATION OF EFFLUENT DISCHARGES

Following evaluation of the field work data modelling was undertaken to simulate effects of the treated wastewater discharges. Three effluent release points (Figure 1.1) were simulated. Model simulations for each outfall location were made for spring and neap tides.

6.1 Effluent Characteristics

Effluent parameters used in the simulations were:

Flow Rate:	53 litres per second;
Faecal Coliform Concentration:	1×10^6 fc/100ml;
Decay Time:	$T_{90} = 12$ hours;
BOD	25 mg/l;
Nitrate	30 mg/l;
Ortho-phosphate	8.0 mg/l.

The bacterial decay time of 12 hours represents a conservative value, typically adopted for coastal waters.

The Fealge river entering at the head of Clonakilty Harbour was assumed to have a discharge corresponding to the 95 percentile condition (i.e. approx 0.075m^3/s).

6.2 Quality Criteria

The Blue Flag scheme sets the acceptable limit for faecal coliform bacteria on a bathing beach at 100fc/100ml based on 80% sample compliance. Inchydoney beach is a designated Blue Flag beach and therefore must comply with this standard.

Nutrient enrichment of estuary waters is considered to occur when median dissolved Nitrogen levels exceed 1.4mg/l and Ortho-phosphate levels exceed 0.06mg/l (60µg/l). Inner Clonakilty Harbour, i.e. upstream of Ring, could potentially become enriched unless these standards are met.

6.3 River Flows and Inner Harbour Dilutions

Flows in the Fealge River have been estimated from EPA statistics for the nearby Argideen river. Typical values are outlined in Table 6.1.

Based on these flows it is possible to compute effluent dilutions in the inner reaches of the harbour. As noted in Section 2.1 most of the estuary dries at low water leaving a small wet area downstream of Ring Pier. Figure 6.1 shows a profile of the channel extending upstream from Ring Pier to the existing town outfall. It can be seen that for about 50% of the time the channel will contain only river waters. Concentrations of the various parameters in the channel at this time have been calculated and are presented in Table 6.1

River Flow m^3/s	Type	Dissolved Nitrogen mg/l	Ortho-Phosphate mg/l	Faecal Coliform fc/100ml
0.075	95%ile	11	3.3	444×10^3
0.9	Mean Flow	2.3	0.46	55×10^3

Table 6.1 – Computed contaminant concentrations in river channel at low tide.

Based on: Effluent Flow: 53 l/s, River backgrounds N: = 1mg/l, OP = 0.02mg/l and FC = 0.

6.4 Model Results – Bacterial Simulations

The model simulations were run for two tidal cycles and outputs generated at 1 hour intervals. Outputs are presented in two formats:

(a) contour plots of bacterial concentration;

(b) time series plots of bacterial concentration at selected sampling strips.

The contour plots, output type (a), show the movement of the effluent plume as it is advected and dispersed over the tidal cycle. Examples for the three outfall cases are contained in Appendices A - C. These show high and low water plume excursions for spring and neap tides during calm conditions.

The time series plots, output type (b), provide a more comprehensive method of comparing the impacts of differing discharge locations. The output contains the peak plume bacterial concentration where it enters a sampling strip and is derived from model output at all stages in the tidal cycle. The peak concentration is the highest value recorded in any 10m x10m cell of the sampling strip. The chosen sampling strip locations are indicated in Figure 6.2. Locations 2 and 3 correspond to the 'Blue Flag' bathing areas while Location 1 is popular with surfing enthusiasts. Locations 4 & 5 within the harbour are chosen for comparative purposes and are not of any special significance.

Time series output plots for each sampling strip are included in Figures 6.3 to 6.7 with a summary of predicted maximum bacterial concentrations extracted from these plots presented in Tables 6.2 to Table 6.5.

The results clearly show that in terms of minimising the bacterial contamination on the Blue Flag beach at Inchydoney the optimum solution is to retain the outfall at the existing location. This applies to both calm and windy conditions.

This result is in keeping with what is intuitively expected for a location such as Clonakilty Harbour where contamination levels at the downstream end of the estuary are of interest. Effluent released from an outfall located furthest upstream has the longest travel time to reach the downstream end and benefits most from the natural mortality of bacteria.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1	440	560	955
2	40	135	175
3	20	50	30
4	780	1170	1650
5	2350	3350	5000

Table 6.2 - Predicted Maximum Faecal Coliform Concentration (fc/100ml) for Spring Tide and Calm Conditions.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1	300	890	1050
2	15	20	70
3	0	20	50
4	1200	1600	1875
5	2080	3000	4150

Table 6.3 - Predicted Maximum Faecal Coliform Concentration (fc/100ml.) for Neap Tide and Calm Conditions.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1	700	1020	830
2	60	80	125
3	40	60	60
4	1200	1300	1400
5	2600	2950	4550

Table 6.4 - Predicted Maximum Faecal Coliform Concentration (fc/100ml.) for Spring Tide and Onshore Wind Conditions.

Model Strip Sampling Location	Outfall Location		
	Existing	Below New	At

	(Town)	Barrage	Ring Pier
1	540	825	1160
2	40	80	85
3	30	60	90
4	1220	1750	1600
5	2600	3000	4200

Table 6.5 - Predicted Maximum Faecal Coliform Concentration (fc/100ml.) for Neap Tide and Onshore Wind Conditions.

6.5 Model Results – Nutrient Simulations

Simulations of nitrate and orthophosphate contaminants were made for calm weather conditions. The modelling procedures were similar to those adopted for bacterial predictions except that a longer 5 day decay time was applied. Model results are summarised in Table 6.6 and 6.7 and show that both nitrate and ortho-phosphate levels will be well below the levels which would indicate nutrient enrichment.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1	0.08	0.10	0.10
2	0.04	0.04	0.05
3	0.02	0.02	0.02
4	0.15	0.17	0.17
5	0.20	0.20	0.20

Table 6.6 - Predicted Maximum Nitrate Concentration (mg/l) for a Spring or a Neap Tide and Calm Conditions.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1	23	28	30
2	12	14	16
3	3	9	9
4	7	52	54
5	56	59	57

Table 6.7 - Predicted Maximum Ortho-phosphate Concentration (μg/l) for a Spring or a Neap Tide and Calm Conditions.

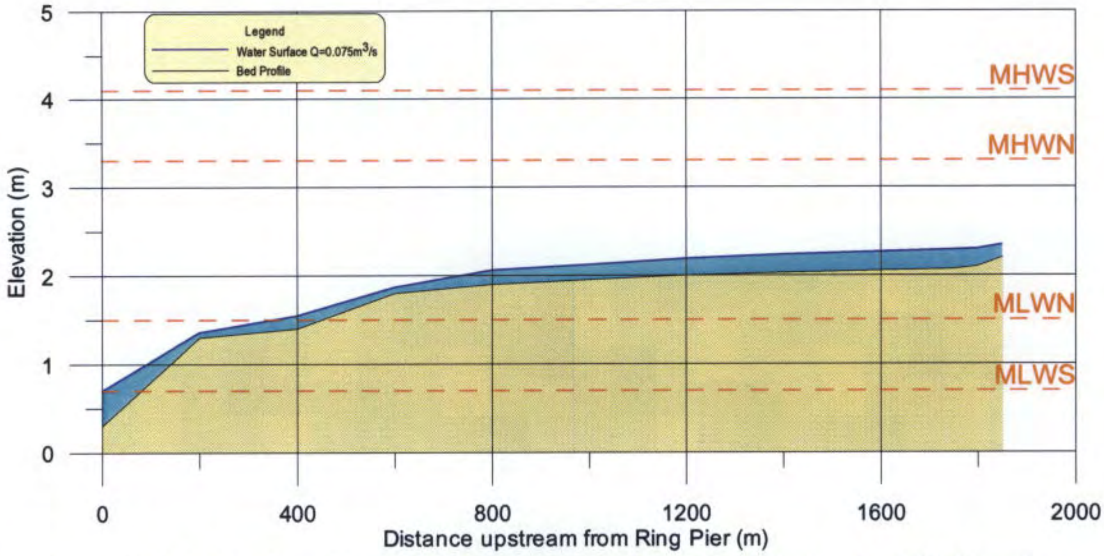


Figure 6.1 – Bed Profile from Ring Pier Upstream to Town Outfall showing tidal levels.

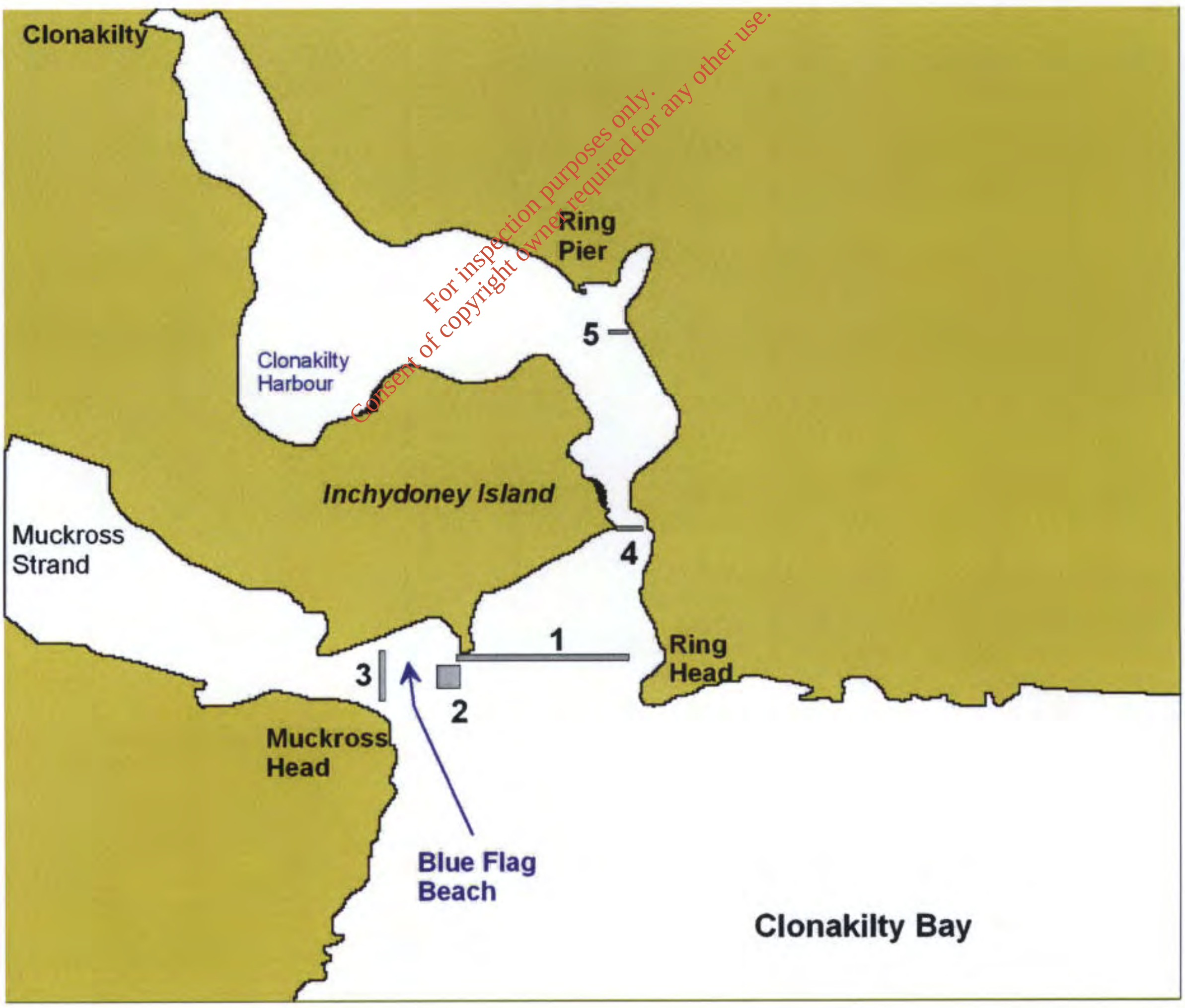


Figure 6.2– Model Sampling Strips.

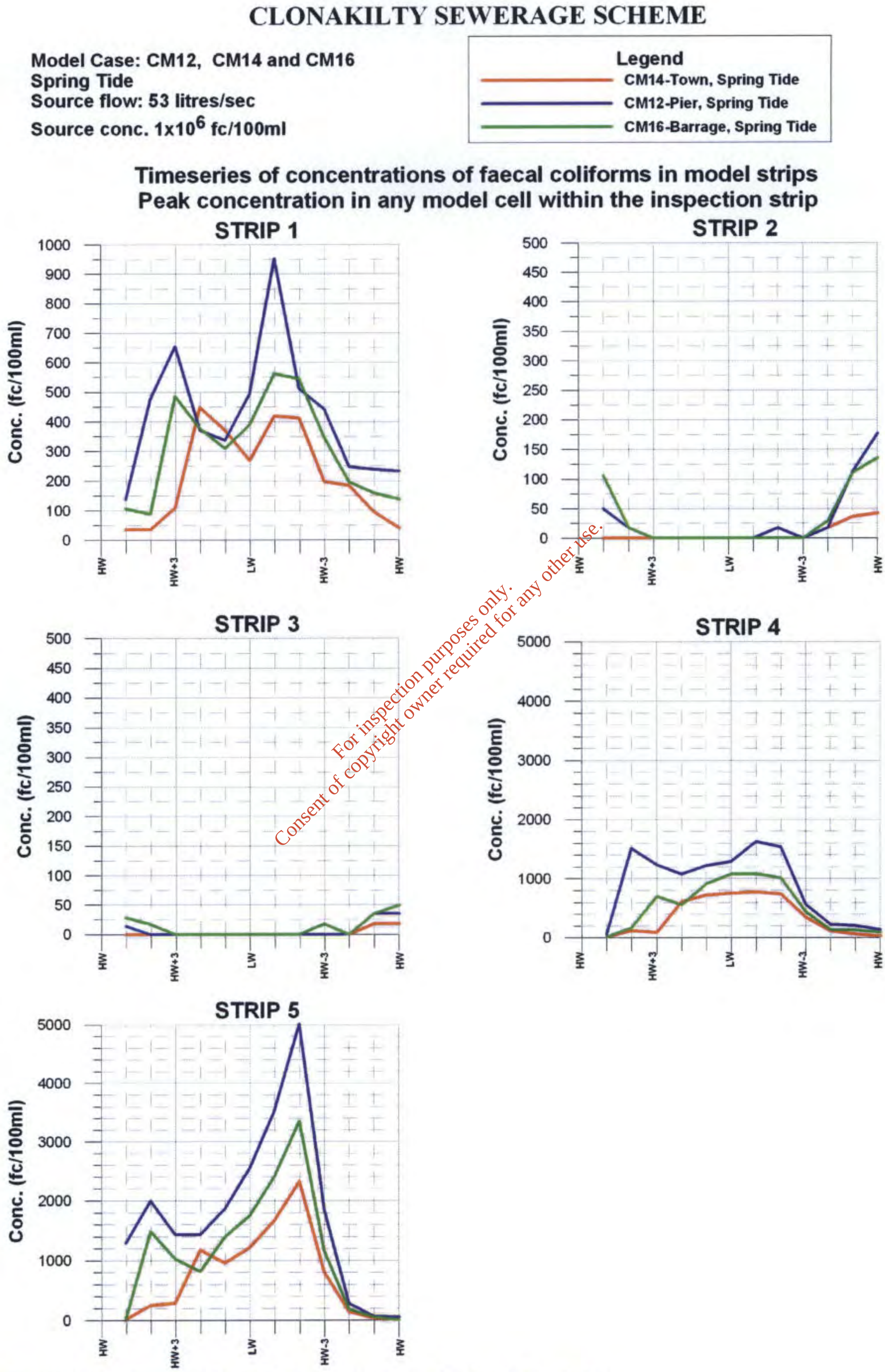


Figure 6.2 – Time Series of Concentration – Spring Tide Calm.

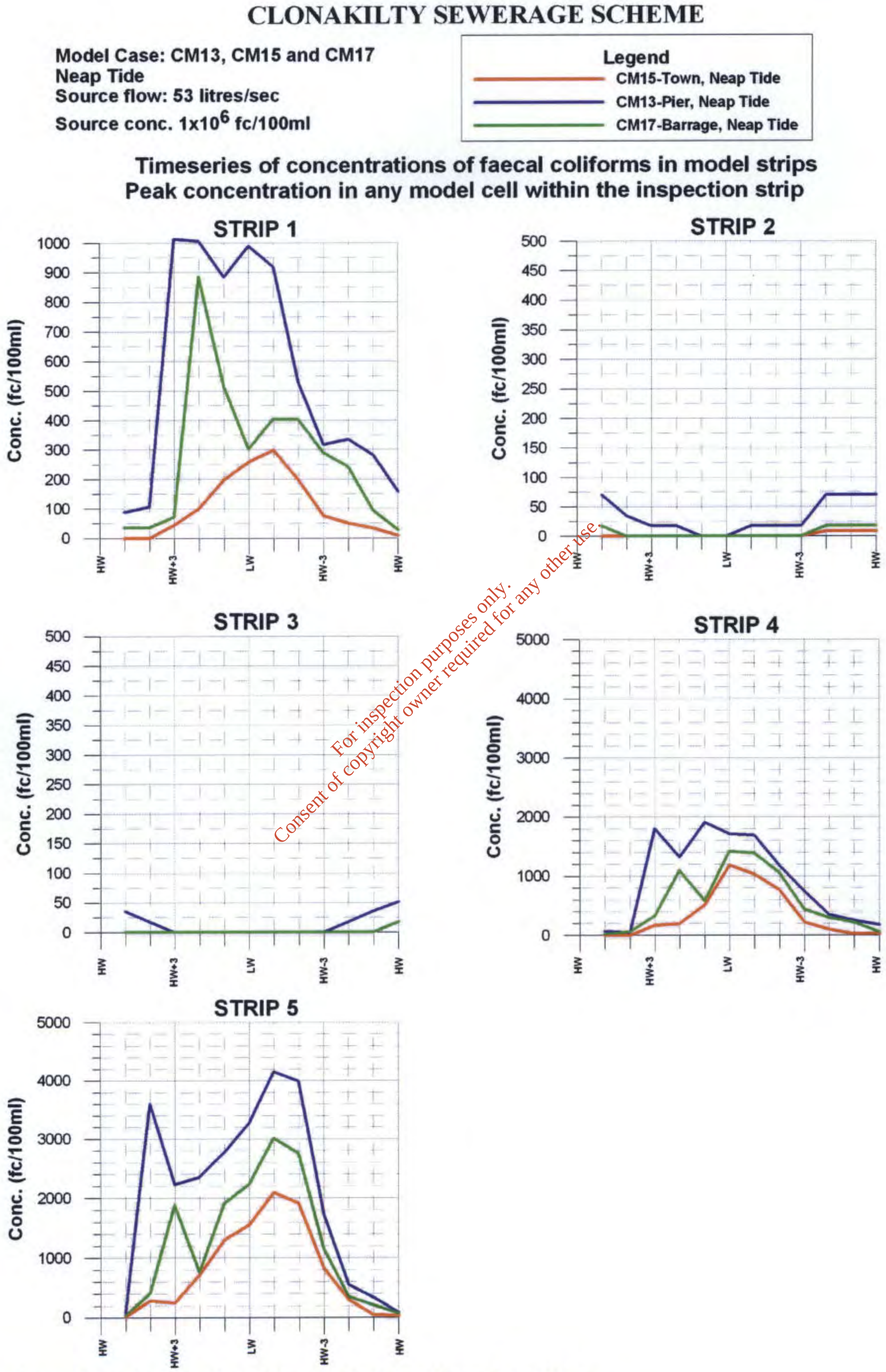


Figure 6.3 – Time Series of Concentration – Neap Tide Calm.

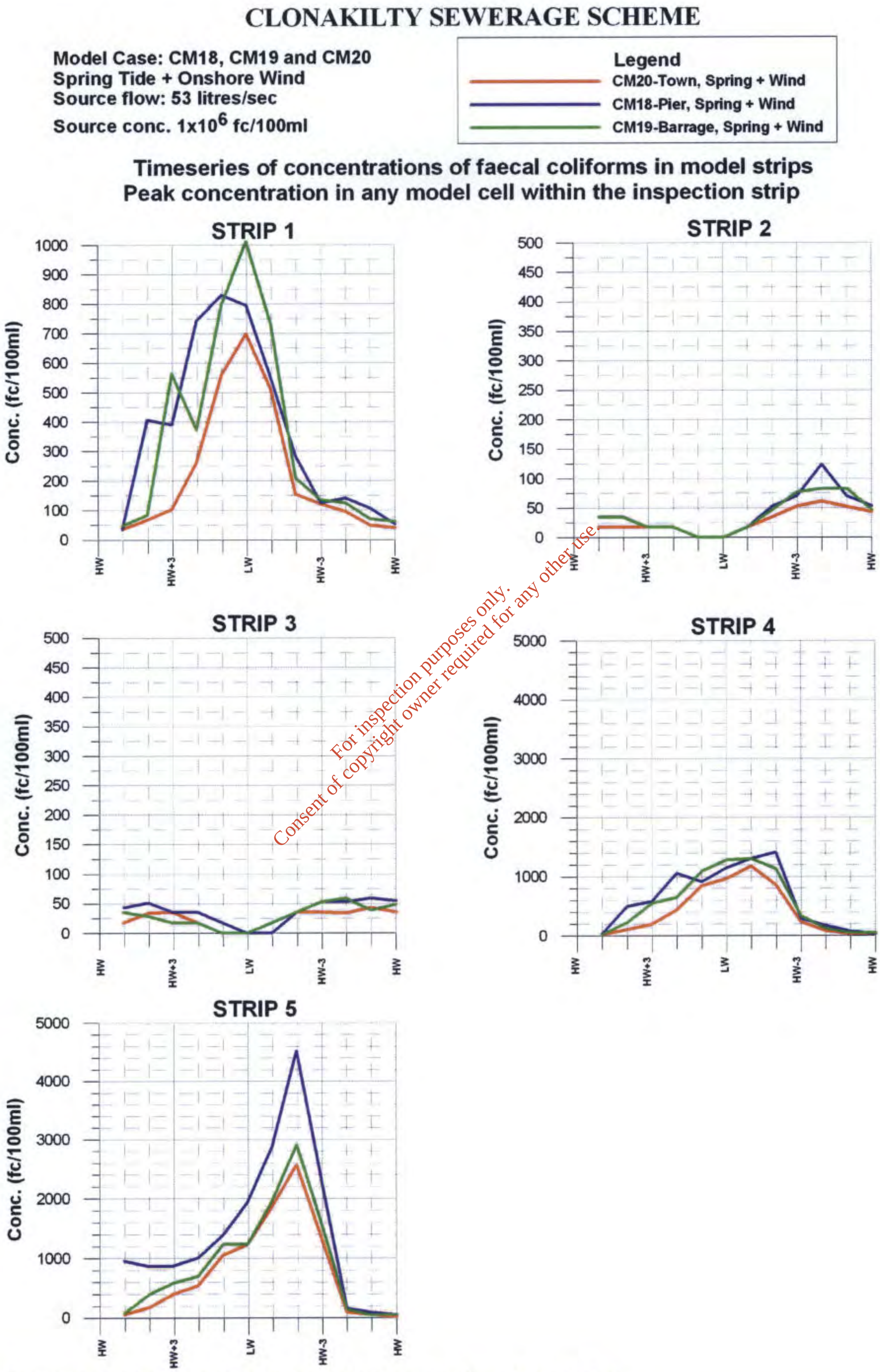


Figure 6.4 – Time Series of Concentration – Spring Tide & Wind.

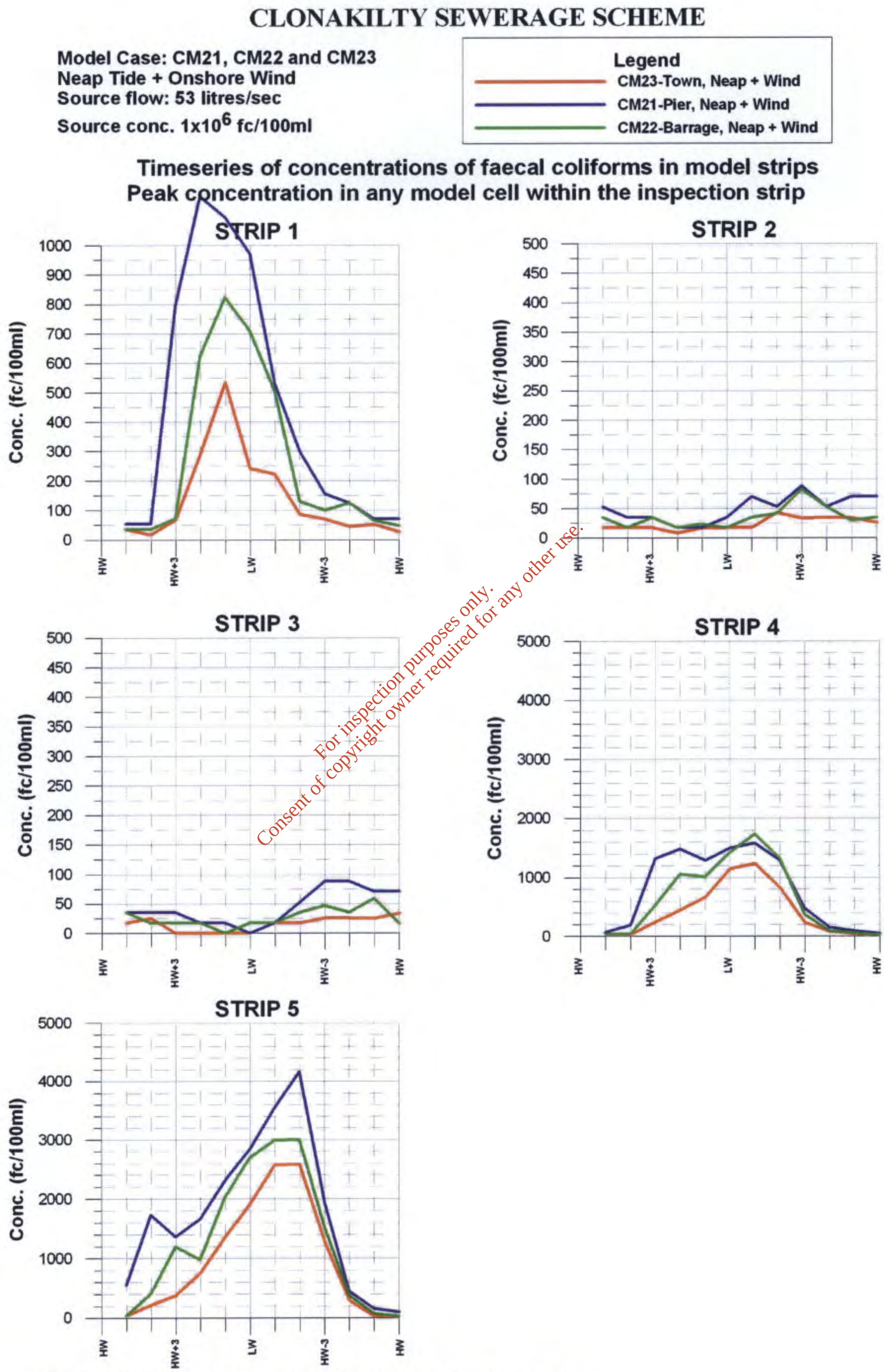


Figure 6.5 – Time Series of Concentration – Neap Tide & Wind.

7. CLOSURE

7.1 This report presents the findings of a marine study of the proposed treated waste water discharges from the town of Clonakilty. The study assesses the dispersive characteristics of the coastal area and comments on the siting of the outfall with regard to possible impacts, particularly on the 'Blue Flag' bathing beaches at Inchydoney.

7.2 The oceanography of the region is typical of coastal sites, though with added complexities due to the shallow inner harbour and the local topographic features. Outside the harbour the surface tidal currents are generally weak and are influenced by prevailing winds. Peak values observed here were about 0.14m/s. Dispersion characteristics are good as shown by dye and drogue data. Tidal ranges in the area are approx. 3.4m on springs and 1.8m on neaps.

7.3 A two dimensional flow model together with a particle track dispersion model was used to simulate the discharges. Recorded data from current meter, drogue and dye releases were used for calibration and validation purposes. Twelve separate simulations were conducted.

7.4 Five representative sampling sites within the model domain were selected to aid assessment of outfall options (Figure 6.1). The predicted maximum bacterial concentrations at these locations are presented as time series in Figures 6.2-6.5 and listed in Tables 6.2 to 6.5. Tables 7.1 and 7.2 present a further simplification, listing maximum values on either a spring or a neap condition for calm and windy conditions respectively.

7.5 The results show that the optimum location for the outfall in terms of minimising bacterial contamination at Inchydoney beach is the existing site adjacent to the town.

7.6 Simulations of nitrate and ortho-phosphate (OP) levels show that for the area downstream of Ring there is little difference between the impacts of the three outfall options. In the inner estuary and the channel area upstream of Ring the town outfall will have the greatest impact on nutrient levels as for about 50% of the time only river water will be present. Dilution calculations show that for mean river flows of $0.9m^3/s$ the nitrate and OP levels will be 2.3 and 0.46 mg/l respectively. For the lower 95 percentile flow condition these levels increase to 11 and 3.3 mg/l.

7.7 A build-up of nutrients is not expected from the wastewater treatment plant as almost full flushing of the estuary occurs on each tide. The incoming tide predominantly floods along the western shoreline past Duneen Head, bringing in cleaner uncontaminated waters.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1 – Surfing Area	440	890	1050
2 – Blue Flag Beach	40	135	175
3 – Bathing Area	20	50	50
4	1200	1600	1875
5	2350	3350	5000

Table 7.1 - Predicted Maximum Faecal Coliform Concentration (fc/100ml) for either a Spring or a Neap Tide and Calm Conditions.

Model Strip Sampling Location	Outfall Location		
	Existing (Town)	Below New Barrage	At Ring Pier
1 – Surfing Area	700	1020	1160
2 – Blue Flag Beach	60	80	125
3 – Bathing Area	40	60	90
4	1220	1750	1600
5	2600	3000	4550

Table 7.2 - Predicted Maximum Faecal Coliform Concentration (fc/100ml) for either a Spring or a Neap Tide and Onshore Wind Conditions.

REFERENCES

[1] ***Chart 1765***
UK. Hydrographic Office, 2005.

[2] ***Clonakilty Harbour Tidal Barrage Study***
Irish Hydrodata Limited, 2000.

[3] ***Admiralty Tide Tables Vol 1, 2005***
UK. Hydrographic Office, 2005.

[4] ***Tidal Stream Atlas for the Celtic Sea, NP256***
UK. Hydrographic Office, 1974.

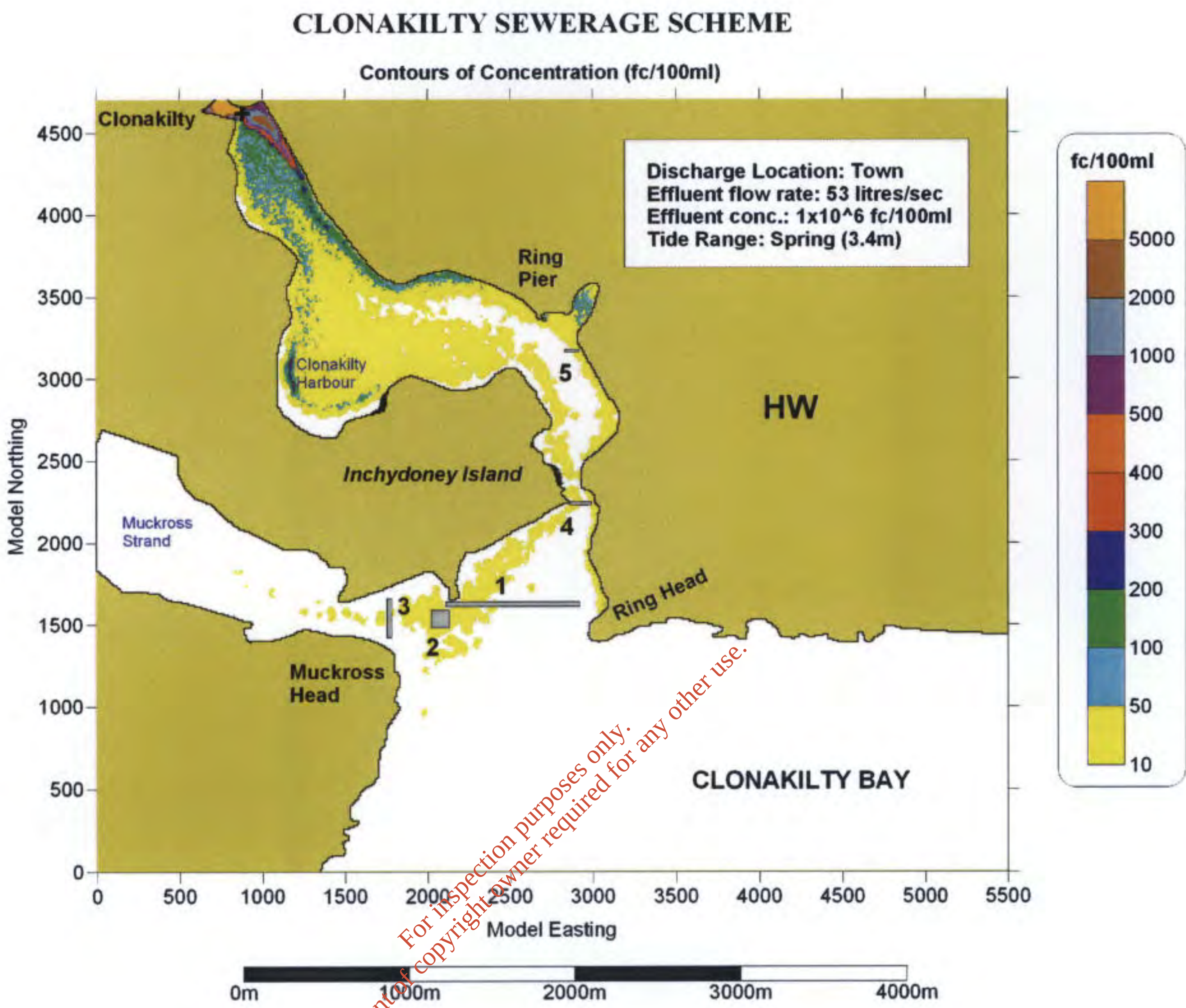
[5] ***User Manual for Numerical Hydrodynamic Models of Marine Systems and Associated Plotting Package.***
J.R. Hunter, 1987

[6] ***User Manual for Two-Dimensional Lagrangian Particle Tracking Model.***
J.R. Hunter, 1987

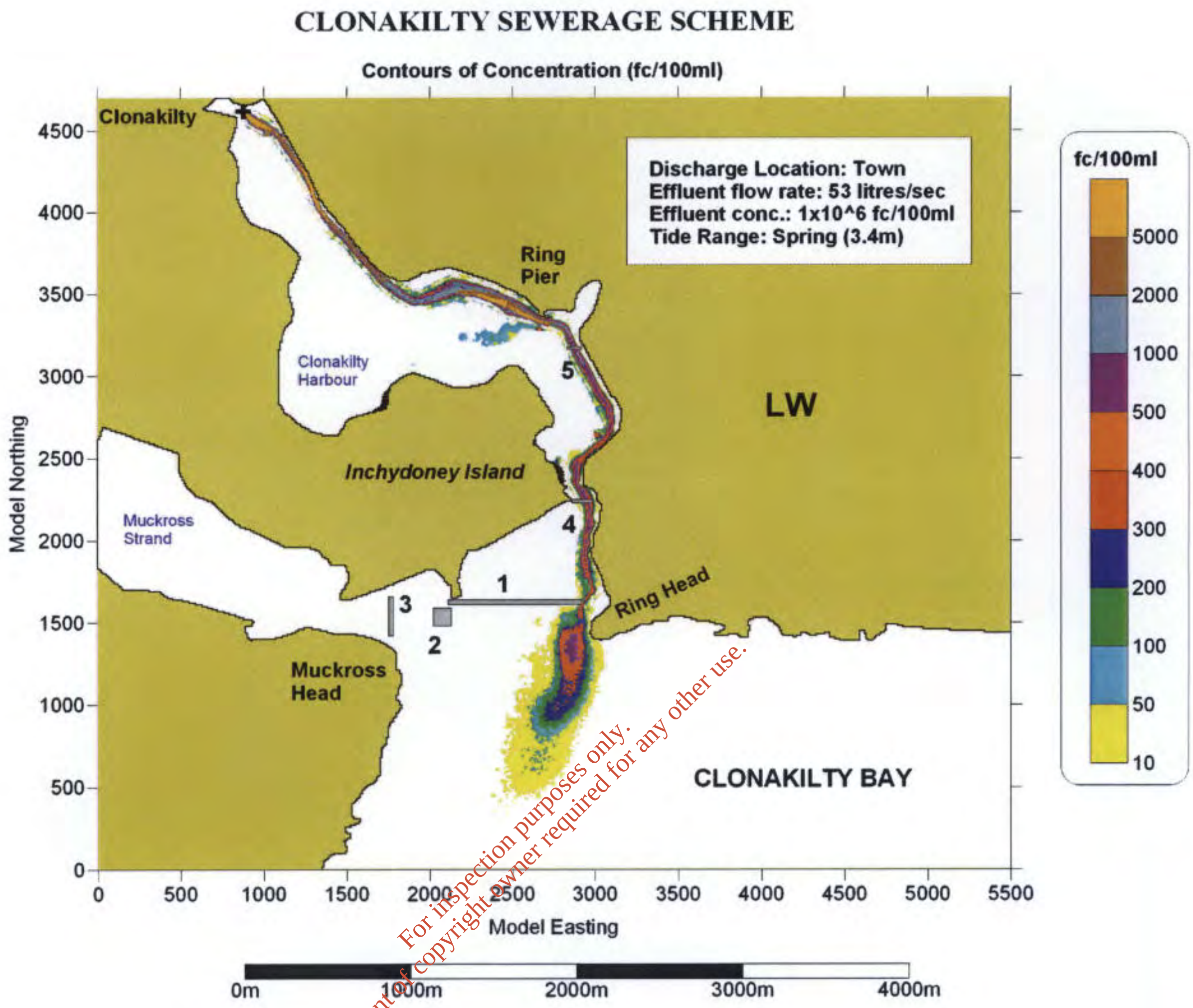
[7] ***Chart 5058, Co-tidal and Co-range Lines for the British Isles and adjacent waters.***
UK. Hydrographic Office, 2005

Appendix A

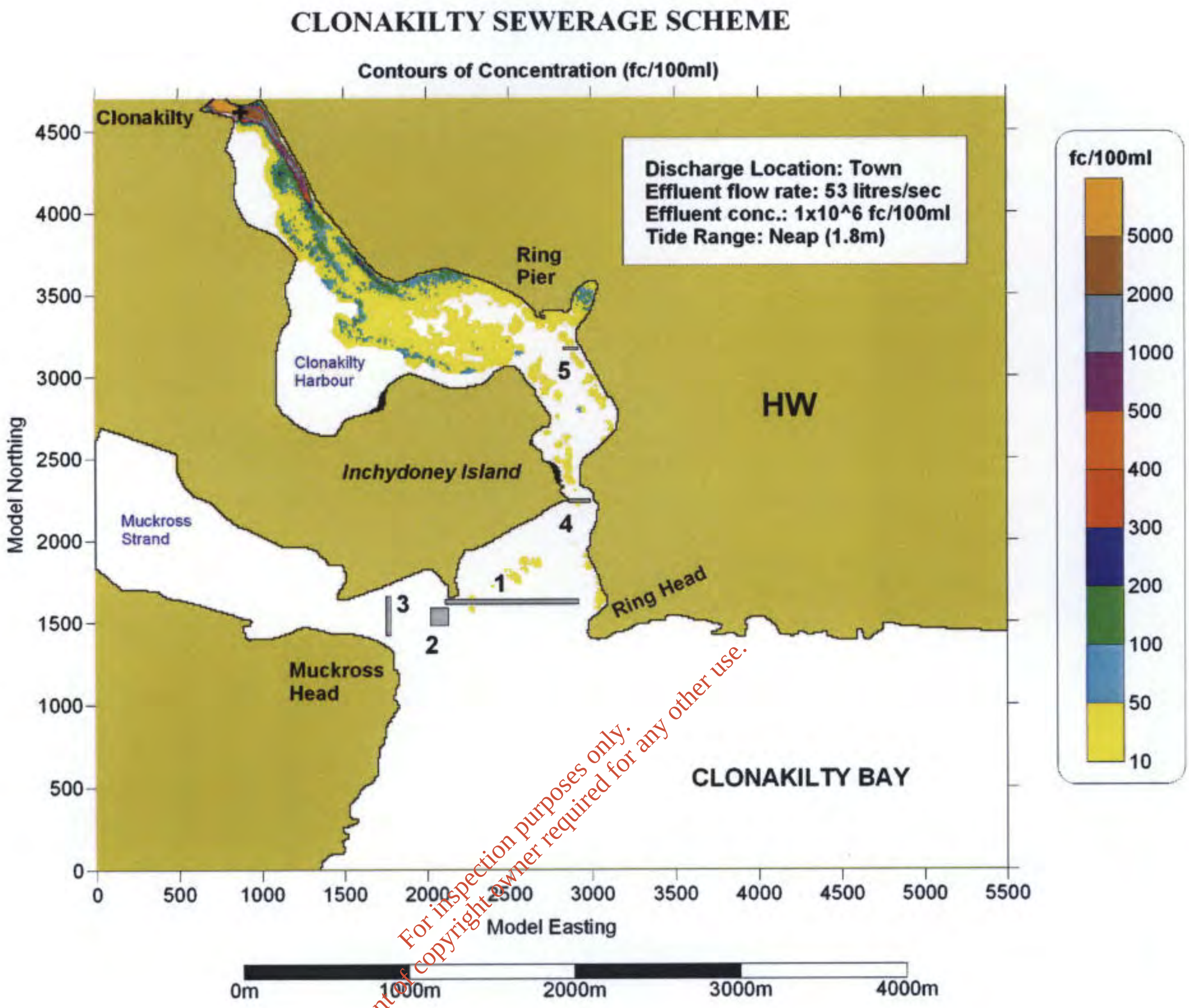
Model Output - Existing Town Outfall



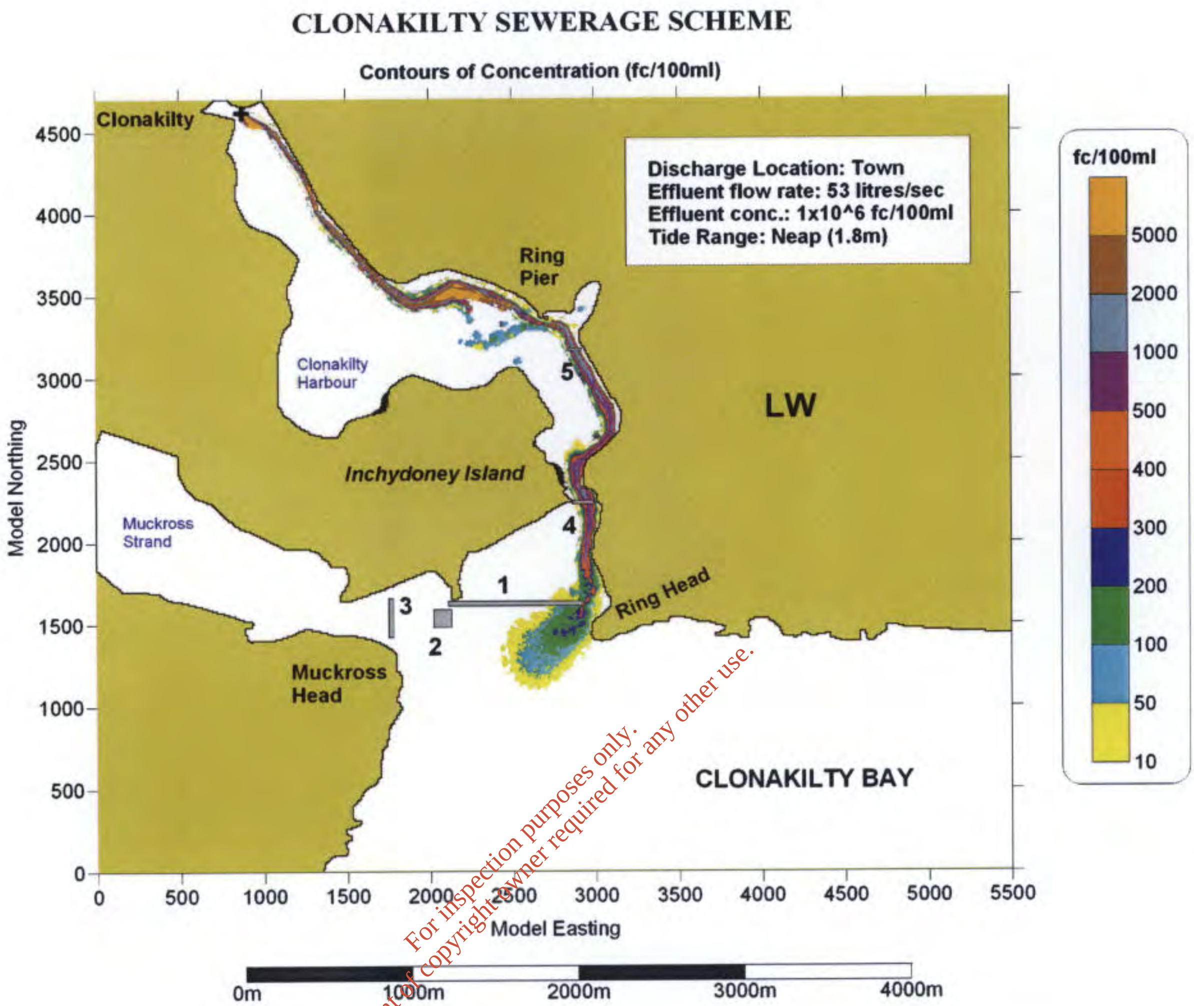
Spring Tide - HW



Spring Tide - LW


CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Town
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Ring Pier
Clonakilty Harbour
HW
Inchydoney Island
Muckross Strand
Ring Head
Muckross Head
CLONAKILTY BAY
1
2
3
4
5
Model Northing
Model Easting
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
0m
1000m
2000m
3000m
4000m
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10

Neap Tide - HW

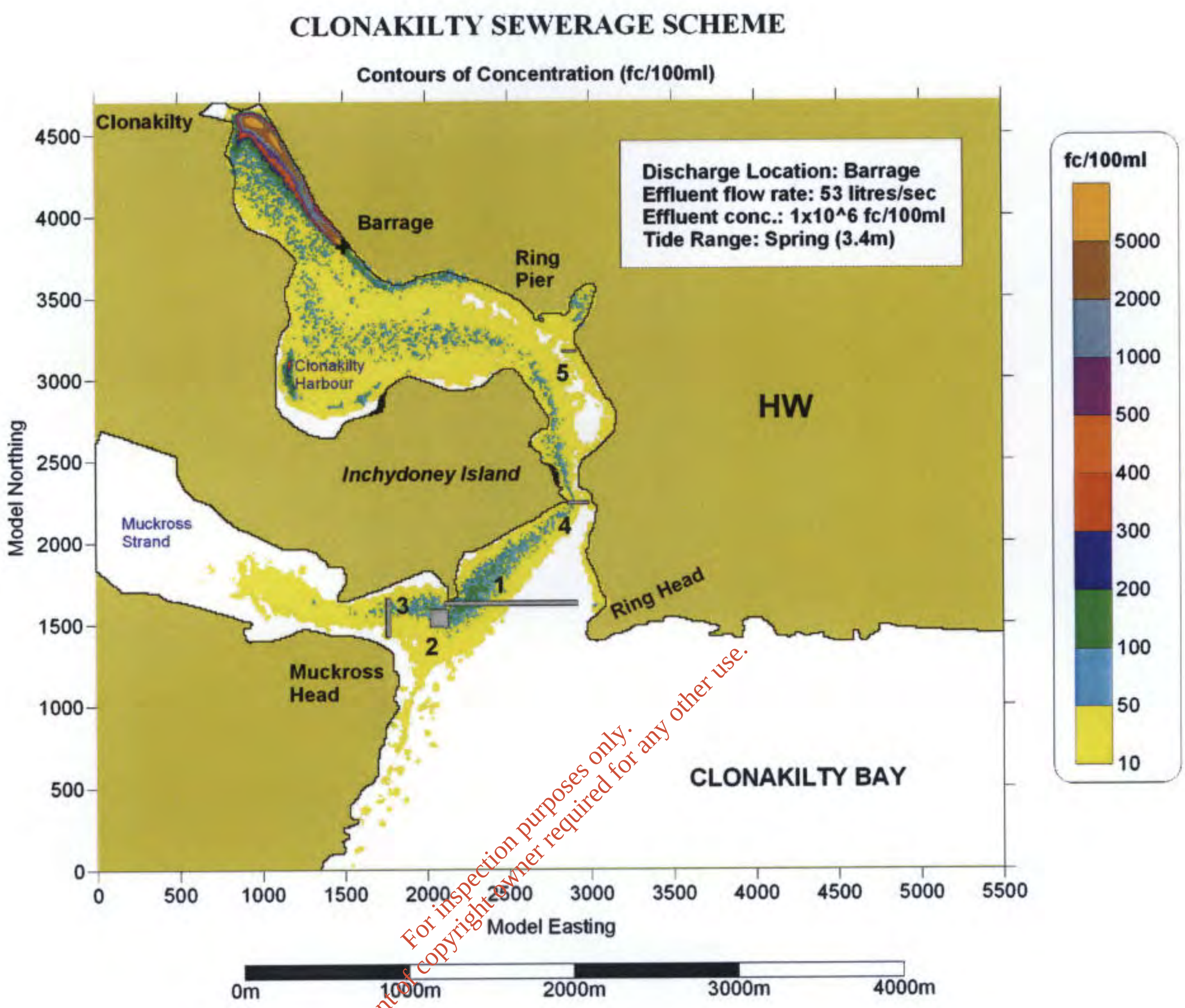


CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Town
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Ring Pier
Clonakilty Harbour
Inchydoney Island
Muckross Strand
Muckross Head
Ring Head
LW
CLONAKILTY BAY
1
2
3
4
5
Model Northing
Model Easting
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10
0m
1000m
2000m
3000m
4000m

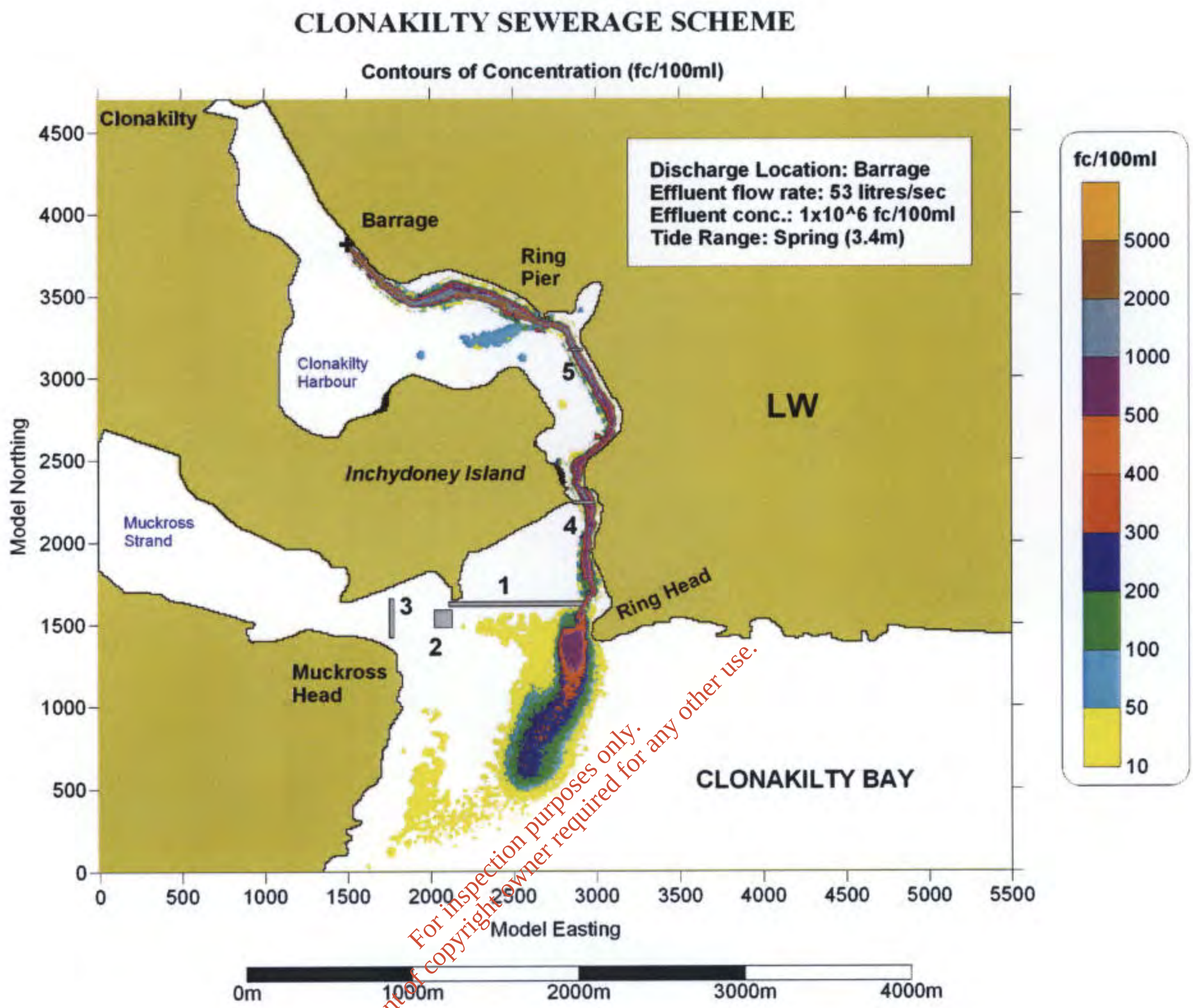
Neap Tide - LW

Appendix B

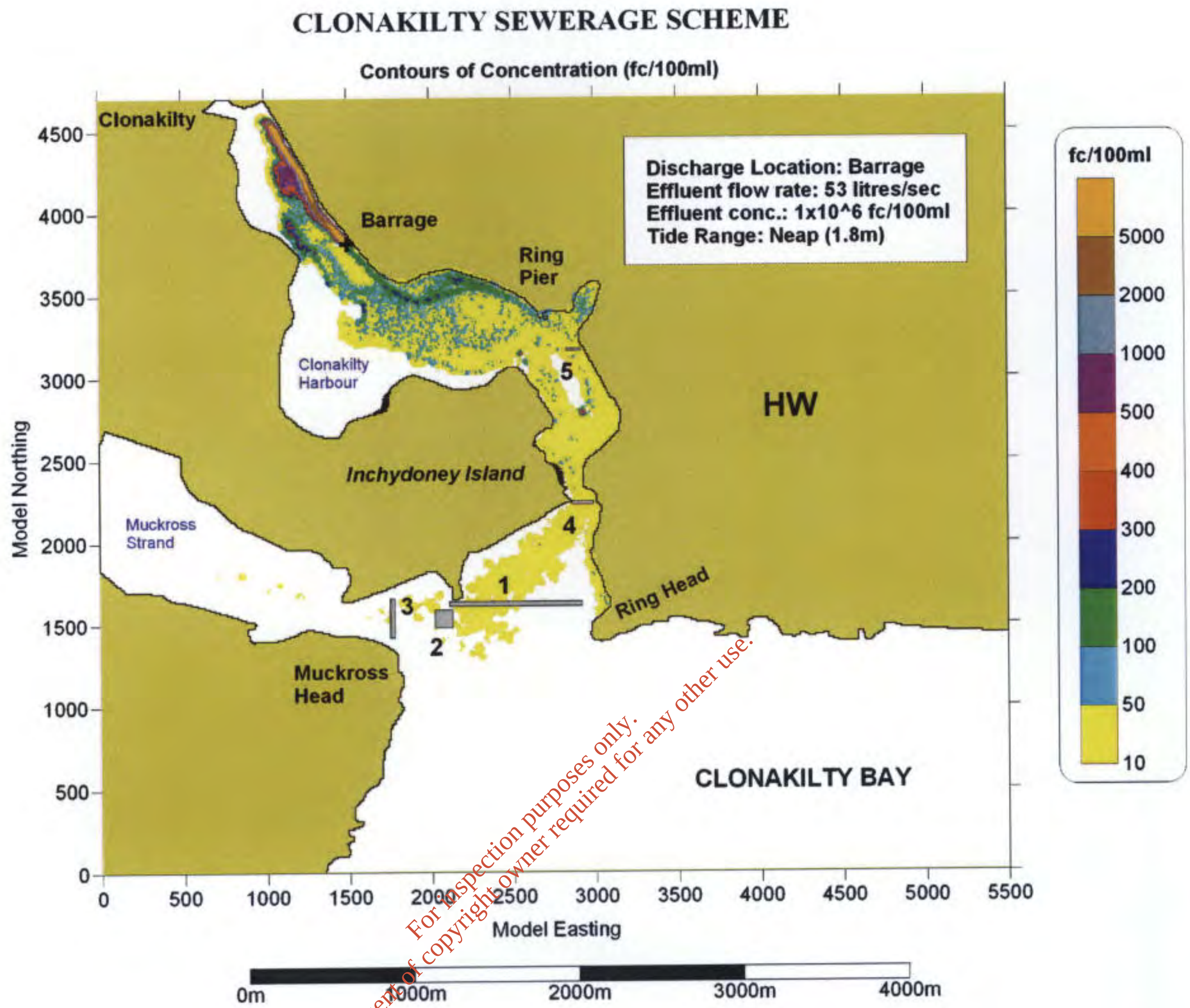
Model Output - Barrage Outfall



Spring Tide - HW

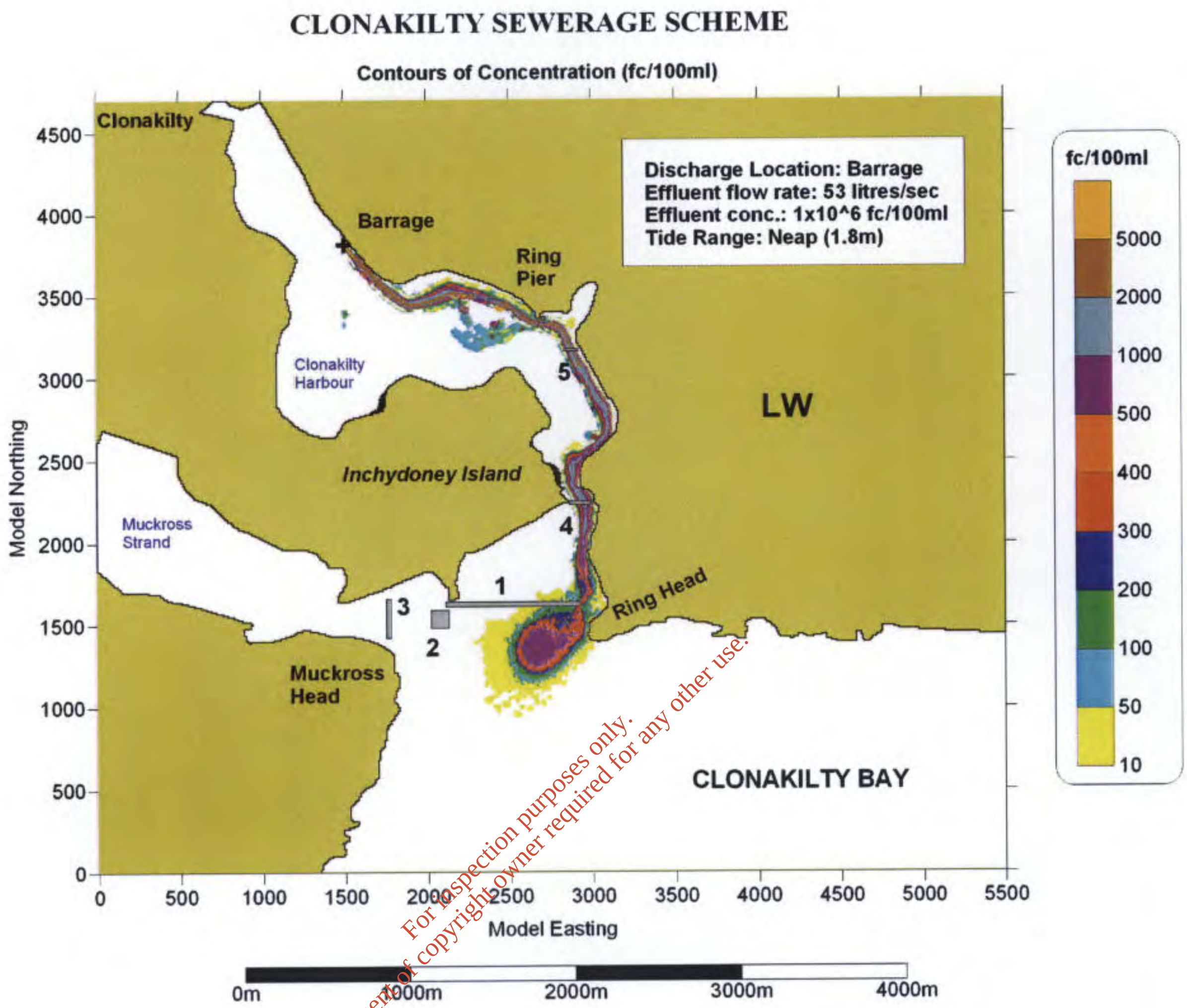


Spring Tide - LW



CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Barrage
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Barrage
Ring Pier
Clonakilty Harbour
HW
Inchydoney Island
Muckross Strand
Ring Head
Muckross Head
CLONAKILTY BAY
1
2
3
4
5
Model Northing
Model Easting
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10
0m
1000m
2000m
3000m
4000m

Neap Tide - HW

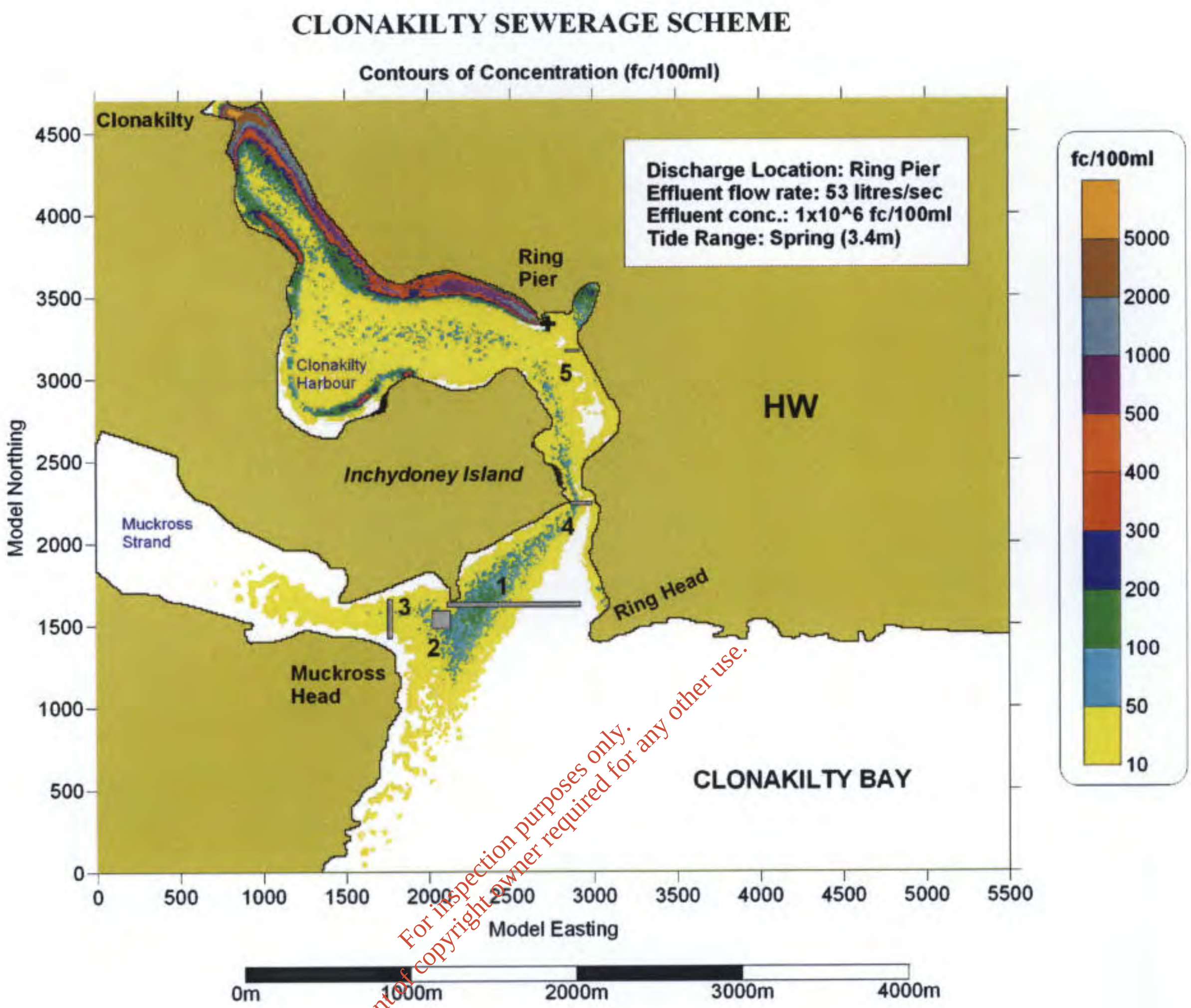


CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Barrage
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Barrage
Ring Pier
Clonakilty Harbour
LW
Inchydoney Island
Muckross Strand
Ring Head
Muckross Head
CLONAKILTY BAY
1
2
3
4
5
Model Northing
Model Easting
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10
0m
1000m
2000m
3000m
4000m

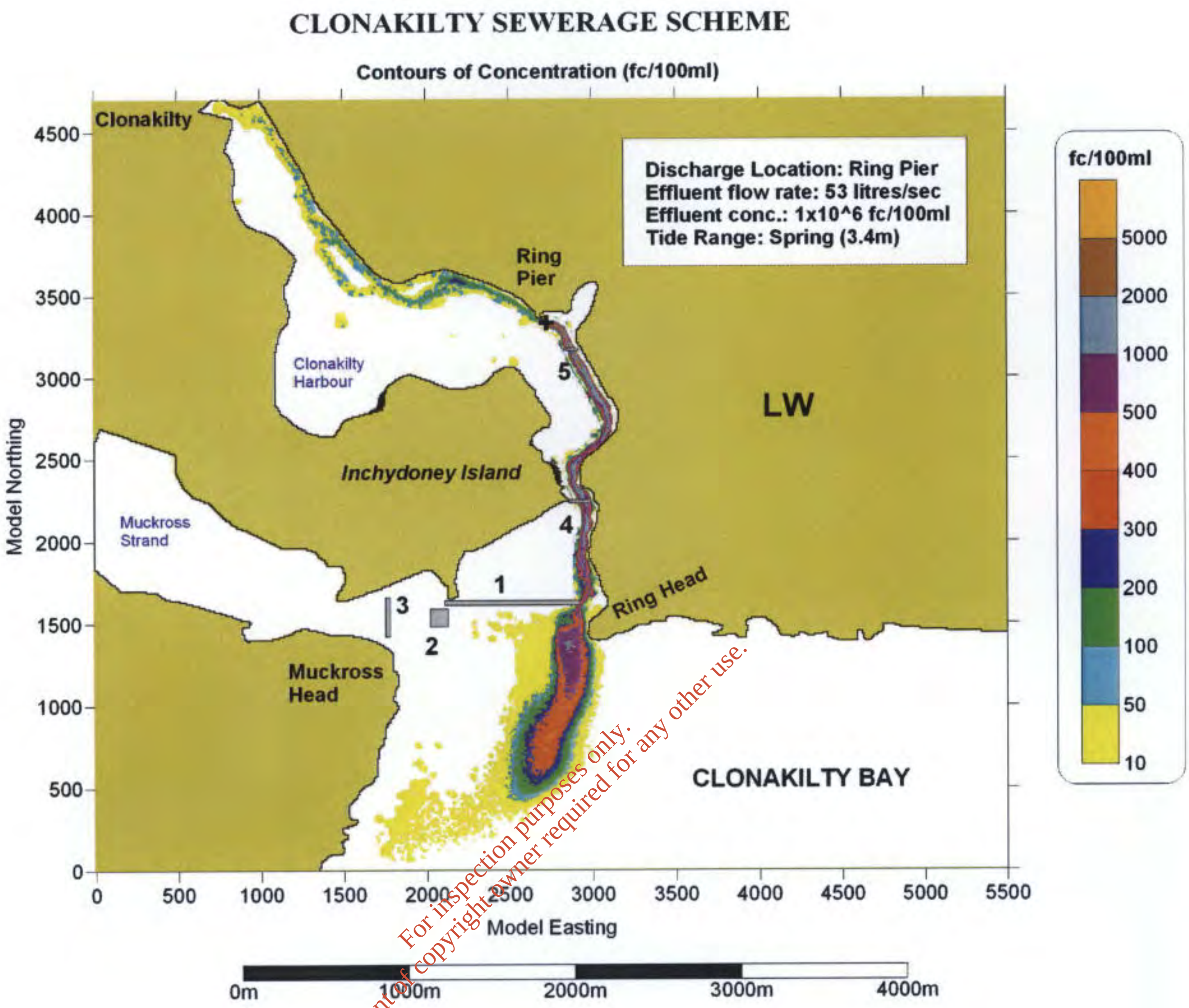
Neap Tide - LW

Appendix C

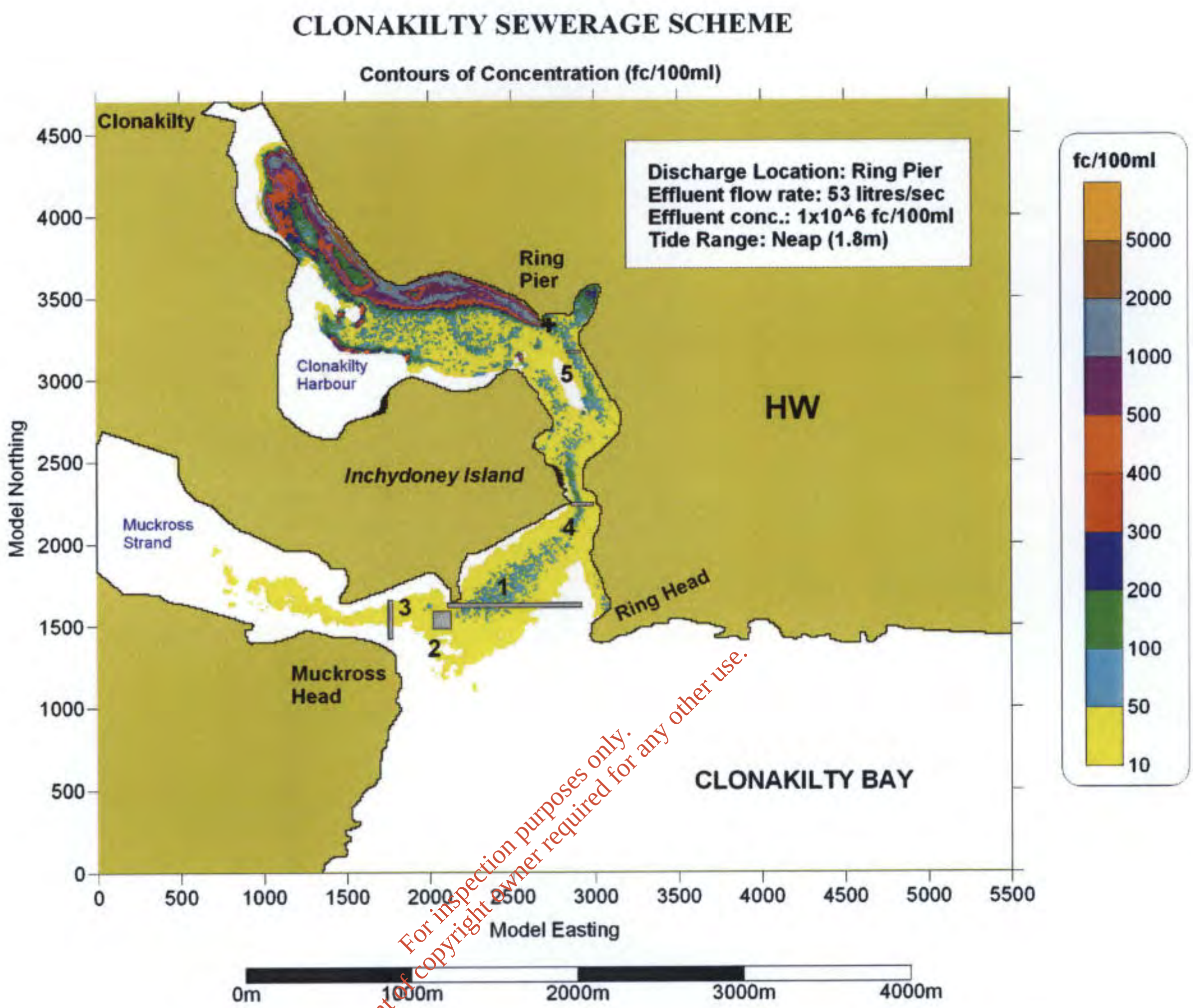
Model Output - Outfall at Ring



Spring Tide - HW

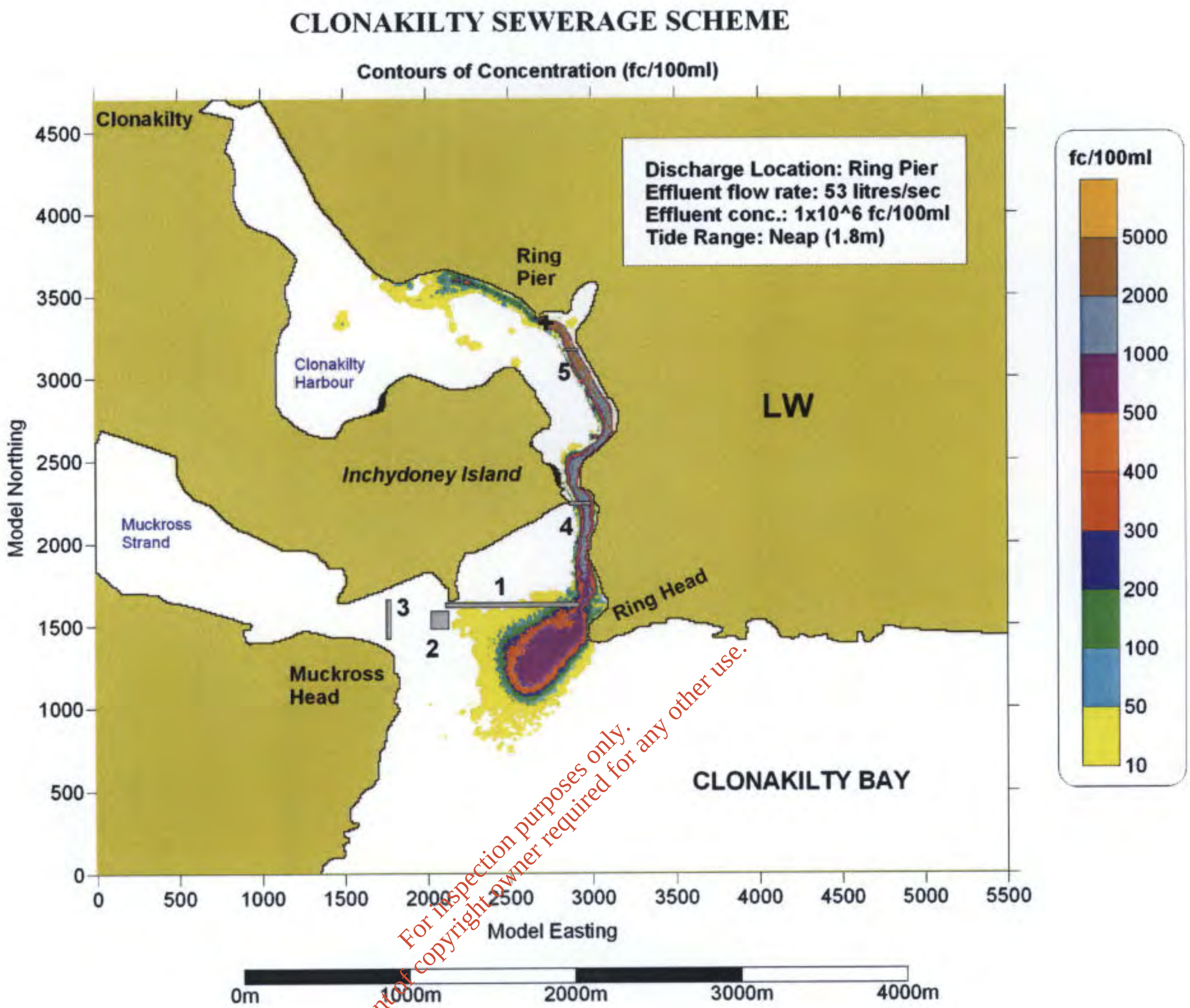


Spring Tide - LW



CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Ring Pier
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Ring Pier
Clonakilty Harbour
Inchydoney Island
HW
Muckross Strand
Ring Head
Muckross Head
CLONAKILTY BAY
Model Northing
Model Easting
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10
0m
1000m
2000m
3000m
4000m

Neap Tide - HW


CLONAKILTY SEWERAGE SCHEME
Contours of Concentration (fc/100ml)
Discharge Location: Ring Pier
Effluent flow rate: 53 litres/sec
Effluent conc.: 1x10^6 fc/100ml
Tide Range: Neap (1.8m)
Clonakilty
Ring Pier
Clonakilty Harbour
Inchydoney Island
Muckross Strand
Muckross Head
Ring Head
LW
CLONAKILTY BAY
1
2
3
4
5
Model Northing
Model Easting
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
fc/100ml
5000
2000
1000
500
400
300
200
100
50
10
0m
1000m
2000m
3000m
4000m

Neap Tide - LW

APPENDIX 6.3

TECHNICAL APPENDIX E

TIDAL & ESTUARINE STUDY

BY

IRISH HYDRODATA LTD.
RATHMACULLIG WEST
BALLYGARVAN
CO.CORK

Clonakilty Tidal Barrage Study

REPORT

Contents

7/12/00

1. Introduction

IHD were commissioned by M.C. O' Sullivan & Co. (MCOS) to undertake a study of Clonakilty Harbour in relation to the proposed construction of a tidal barrage which is intended to protect the low lying areas of Clonakilty town from flooding during extreme tides.

The study sought to address a number of queries, the principal of which were;

a) to establish estimates of extreme water levels for design purposes;
b) to assess the likely impact on the erosion/deposition patterns in the harbour area;
c) to assess the impact of short term barrage closure on water quality in the inner harbour.

Data Sources

Data was available from various sources. These included tide and topographic measurements made by M.C. O' Sullivan & Co in 1992-1993, float tracking and bathymetric measurements made by Irish Hydrodata in 1990 together with specific measurements made for this study. The latter included bathymetric and topographic measurements, tidal level measurements and current recording.

Harbour Characteristics

The harbour, from the open sea to its head at Clonakilty town covers an area of approx. 2 million square metres. The entrance between Inchydoney Island and the mainland is a narrow channel which restricts the tidal flow and alters the tidal elevation curve from the sinusoidal open sea conditions. It shortens the flood tide and lengthens the ebb tide. This effect is more noticeable on spring tides than on neaps. Occasionally a bar forms at the entrance maintaining the low tide levels in the harbour above the open sea level.

Water depths are generally very shallow with most of the inner harbour drying soon after mid-tide. At low water the entire harbour, save for a narrow channel along the eastern shoreline from Ring to the sea is dry. Tidal levels for the harbour based on data measured at Wind Rock, are presented in Table 1.1.

Tide Level	Level to OD Poolbeg
Mean High Water Spring Tide	4.1m
Mean High Water Neap Tide	3.3m
Mean Tide Level	2.4m
Mean Low Water Neap Tide	1.5m
Mean Low Water Spring Tide	0.7m

Table 1.1 - Tidal Levels at Wind Rock.

Upstream of the proposed barrage site approx. 80% of the seabed is above the 2.4m contour and therefore will be dry for half of each tidal cycle. The river channel has a bed level of approx. 2m in this area.

The harbour bed comprises extensive areas of fine to medium sand where bed forms indicate mobility and lesser areas of soft silty sediments with exposed clay/ peat formations are present. Much of this sediment has its origin in the open sea and would have been transported into the harbour on the flooding tide. The outer and middle parts of the harbour are dominated by sand while the north and western shorelines are the muddier regions.

Mobile sand ribbons are present adjacent to and along the bed of channels. These extend upstream of the proposed barrage site, almost as far as the town itself. Coarser materials including fine gravels and pebbles were occasionally observed in these channels with the sediments becoming increasingly finer away from the channels.

Local anecdotal evidence indicates that appreciable and rapid variations occur in bed levels and channel patterns in the middle harbour while nearer the town towards the head of the harbour variations are not noticeable.

Barrage Structure

The proposed barrage will be constructed at the location shown in Figure 1.1. It will extend 300m from bank to bank and contain sluice valves. These will be located to coincide with the existing river channel which runs along the eastern shoreline as shown in Figure 1.2.

The area upstream of the barrage will drain completely during each low tide as occurs at present.

2. Extreme Water Levels

Approx. 15 months of continuous tidal measurements were made at the location of the proposed barrage by MCOS during 1991-1993. The gauge was a recording float type and was attached to the eastern shoreline wall. The data is in analogue chart form and is truncated at +3.4m above Poolbeg datum. Almost all high waters, except for those coinciding with low neap conditions were recorded. The highest tidal level recorded in this data set was 4.886m above Poolbeg datum. Levels in excess of 4.5m were recorded on seven occasions during the period.

As part of this study a one month data recording was made outside the harbour, near Wind Rock, where the water depths allowed the full tidal curve to be recorded. Gauge locations are shown in Figure 2.1 while tidal data for the latter location are shown in Figure 2.2. These are reduced to Poolbeg datum. Also shown on this plot are the computed residuals; these are defined as measured level minus the predicted level and represent that part of the tidal signal that cannot be explained by astronomical variations and are of therefore of meteorological origin. The predicted tides are derived from a harmonic analysis of the measured tidal data which extracts the constants shown in Table 2.1.

Various analysis techniques were then employed to extract extreme levels. In particular a 1:50 year water level which is typically taken as a lower design limit. The resulting 1:50 year values are summarised in Table 2.2.

For Method 1 the measured spring tide is combined with results of a storm surge model as outlined in [ref:1] to give an estimate of the water level with 50 year return period.

In Method 2 the tidal constants in Table 2.1 were used to predict levels coincident with the measured values for the 1991-1993 data set. The residuals were then calculated and are illustrated in Figure 2.3. Typical values were 0.3 to 0.5m with the highest value of ≈0.8m occurring in January 1993. These residuals were plotted on probability paper, Figure 2.4, from which a 1:50 year residual was extracted. This value combined with the mean spring tide produces a level which has an overall 1:50 year probability (assuming that extreme residuals continue to follow the probability distribution). Tidal predictions were also made for a 19 year solar cycle to extract the highest astronomical tide (HAT) in this period. This predicted value is 4.56m ODP.

Reference [2] gives an equation which describes the interrelationship of events occurring with different return periods. { L_N=0.89(1+0.03 *ln* N)L_{50}, where N is the return period in years and L_{50} is the 50 year level }. Applying this to the high waters observed once per year and 12 times per year in the 1991-1993 data set allows further estimates of the 50 year levels. These are referred to as Methods 3 and 4 in Table 2.2.

From these data sources we conclude that a best estimate of the 1:50 year extreme water level is 5.3m above OD Poolbeg.

A worst case scenario would be to combine the 50 year residual with the highest astronomical tide giving an extreme level of 5.7m ODP. It is not possible to assign a probability level to this estimate.

The highest tidal level recorded in Clonakilty in recent times was 5.5m OD Poolbeg at 2000hrs on December 16th, 1989. The predicted tide level for this time was 4.0m giving a residual of 1.5m. If this residual had occurred at the time of HAT then the water level would have reached 6.06m. Again it is not possible to assign a probability level to this value.

The above values ignore global warming effects which are difficult to reliably quantify. Various sources suggest these may account for a 0.3m to 1m rise in sea levels over the next 100 years. An appropriate number should be added to the 1:50 year extreme level of 5.3m to arrive at the design level.

	Amplitude (m)	Phase (°)
MEAN	2.386	
MM	0.051	119.3
MSF	0.098	114.3
Q1	0.005	129.7
O1	0.029	15.2
M1	0.006	257.8
P1*	0.006	153.3
K1	0.018	139.7
J1	0.004	197.2
OO1	0.016	126.1
2N2*	0.028	65.9
MU2	0.043	324.0
N2	0.224	119.5
NU2*	0.050	113.3
M2	1.269	142.3
L2	0.033	162.0
T2*	0.022	192.1
S2	0.390	180.2
K2*	0.105	179.2
2SM2	0.013	72.5
MO3	0.004	47.0
M3	0.010	27.6
MK3	0.005	320.3
MN4	0.013	218.1
M4	0.053	237.4
SN4	0.012	246.7
MS4	0.032	293.8
2MN6	0.004	183.1
M6	0.008	186.7
MSN6	0.003	190.3
2MS6	0.012	234.9
2SM6	0.005	295.5

Table 2.1 - Harmonic Constants, Wind Rock, OD Poolbeg.

	Method	50 year level
1	Based on ref. 2 using spring tide amplitude and surge from continental shelf storm surge model	5.11m
2	Residual analysis of measured tidal data	5.24m
3	Extrapolated from measured data using 12 highest levels in 1 year	5.30m
4	Extrapolated from measured data using highest level in 1 year	5.50m

Table 2.2 - Estimates of extreme water levels with return period of 50 years.

3. Erosion/Deposition

Model

The impact of the proposed barrage on the tidal regime within the harbour was investigated with a 2d hydrodynamic model (ref.4). The model covered the entire harbour area from the open sea to Clonakilty town as shown in Figure 3.1. A cell size of 10m X 10 m was used. Bathymetric & topographic data obtained from surveys and other sources were used in the construction of the model. Tidal, current and float data was used in the calibration.

Flow Patterns & Tidal Elevations

The existing situation (i.e. no barrage) was first simulated for spring and neap tides. Currents and tidal elevations were estimated for each cell within the model domain. Subsequently the model was altered to include the barrage and simulations repeated for the same cases. Figures 3.2a,b and 3.3a,b show examples for the flood and ebb flow patterns with and without the barrage. Essentially in the vicinity of the barrage the formation of jets associated with the sluice valves cause localised speed increases while reduced speeds occur in the dead zones.

Model time series data (spring tide) for a cell downstream of the barrage, near Ring pier, are shown in Figure 3.4a,b. For the existing situation the model shows that the tidal amplitude curve is already distorted from the open sea sinusoid as a result of the narrow entrance channel. Introduction of the barrage, Figure 3.4b, produces no change in the tidal amplitude. The high water levels remain the same as do the duration of the flood and ebb tides. The main differences are that the rate of fall of the tide is reduced towards the latter stages of the ebb and that the speed curve is slightly altered.

The before and after tidal data for a cell just downstream of the proposed barrage are shown in Figures 3.5a,b. No change is predicted in tidal amplitude. The main differences are again a reduction in the rate of tide fall during the ebb and alteration of the speed curve.

Upstream of the barrage model data, Figure 3.6a,b, indicate that the barrage will cause a time shift in the tidal curve. This is due to the delay introduced by flow through the sluice valves. The tidal amplitudes do however remain the same while speeds are altered at this cell.

Variations in speed and directions are to be expected at cell locations and taken in isolation are not necessarily of significance.

Siltation / Erosion

Results from the models were used in a qualitative manner to assess likely changes in siltation / erosion patterns resulting from construction of the barrage. Exact simulation of this process is complex due to the difficulty in assigning reliable pick-up and deposition patterns to the sediment particles and accounting for the influences of factors such as wave action, wind induced circulations, fresh water inflow and extreme events.

It is change in the overall flow dynamics that ultimately determines the transport of fine sediments and such change is therefore indicative of likely erosion or deposition patterns. Sediment in the harbour is a fine/ medium sand and therefore will have an erosion threshold velocity of ≈0.2m/s and a deposition threshold velocity of ≈0.05m/s (ref: 5). Speeds between these limits will be sufficient to transport material around the harbour.

For a general comparison of before and after situations two factors are considered significant and are examined with the model data. These are:

a) have the speeds increased or do they remain above the erosion threshold velocities for greater periods of time on either the flood or ebb tide;

b) have the speeds reduced and do they remain below the deposition threshold velocity for longer periods of time on either the flood or ebb tide.

Comparisons were made of the flood and ebb tide speed changes for both spring and neap tide conditions. Spring and neap data have been combined together in single plots and are presented in Figures 3.7a,b and 3.8a,b.

Figure 3.7a compares changes in flood tide erosion speeds. Areas indicated as blue represent locations where the current speed will remain above the erosion threshold for increased periods of time following construction of the barrage. Therefore the potential for erosion is increased on the incoming tide in the vicinity of the sluices and just upstream of the barrage.

Figure 3.7b compares changes in the ebb tide erosion speeds and shows that increased erosion potential occurs downstream of the barrage in the main channel.

Settling out of sediment will occur when speeds fall below the necessary deposition threshold. Figure 3.8a shows areas where the flood tide speeds remain below the deposition threshold velocity for a greater period of time after construction of the barrage. Increased deposition potential is seen to occur adjacent to the western part of the barrage and just downstream of it.

Figure 3.8b compares changes in the ebb tide deposition speeds and shows that deposition potential is increased in the inner harbour area.

Figure 3.9 presents a summary plot of potential deposition/erosion areas based on an interpretation of all data plots. The variation in erosion/deposition potential as illustrated by the model refer to differing aspects of the tidal cycle and therefore are not necessarily related. (i.e. a location can have both increased erosion potential at one time in the cycle and increased deposition potential at another depending on how the speed curve has been altered). Therefore Figure 3.9 should not be interpreted as indicating net erosion / deposition.

4. Water Quality

Under normal operating conditions the barrage will remain open at all stages of the tide. This will allow free flow of waters in the harbour with a regime that is little different from that presently prevailing. During extreme events when flooding is anticipated the barrage may be closed for a period around high water. This will be a short duration event, limited to a period of less than 6 hours and will not appreciably influence the water quality within the barrage lagoon.

On occasion it may be desirable to close the barrage for an extended period of one tide to facilitate recreational activities within the lagoon. The impact of such closures have been simulated with a box type model. The assumption of this model is that waters entering the lagoon either from the rivers or the outer harbour become fully mixed, no stratification or lateral variation is allowed.

The proposed barrage and sluice arrangement are indicated in Figure 1.2. The area upstream of the barrage will drain completely on each tide. Model simulations have been prepared for the cases indicated in Table 4.1 and results are presented in Figures 4.1 to 4.10. Fresh water inflow will have significant impact on the lagoon conditions therefore simulations include both a typical (estimated) average daily flow of $0.5m^3/s$ and a 1-year flood flow of $5m^3/s$.

Downstream of the barrage a sinusoidal tide is applied. This together with the fresh water inflow from the rivers govern the water levels upstream of the barrage. As the outside water levels fall those within the barrage fall at a slower rate with the outflow lasting for approx. 8-9 hours. When the tide level rises above the cill level inflow commences, forcing a rapid rise in the water levels upstream of the barrage. The water levels will eventually reach the same level though there is a time lag as seen in Figure 4.1 for example. The size of this lag will ultimately depend on the dimensions of the sluice valves adopted in the final design which govern the rate of inflow/outflow.

Model simulations are started from time zero with assumed initial conditions. After a short time equilibrium is reached where water levels and parameter concentrations approximate to a steady state. Output is then commenced. After a period of time and on a subsequent high water the barrage is 'closed' for approximately 15 hours. The model simulates this situation and the response of conditions upstream of the barrage. When the barrage is again opened the model shows the return to steady state conditions.

	Parameter Simulated	Tide	River Input
1	Salinity	Spring Tide Neap Tide	$0.5m^3/s$& $5.0m^3/s$
2	Ammonium (NH4+)	Spring Tide Neap Tide	20 l/s @ 5mg/l
3	Coliform	Spring Tide Neap Tide	Unit Source
4	BOD	Spring Tide Neap Tide	Unit Source

Table 4.1 - Lagoon Conditions Simulated.

5. Closure

The potential impact of the proposed barrage construction on water movements in Clonakilty Harbour has been investigated.

Tidal data have been analysed and an estimate of the 1:50 year level is 5.3m above Poolbeg datum. To this should be added an appropriate long term sea level rise to allow for global warming.

Models have shown little changes in tidal patterns within the harbour, high water will reach the same level while there will be a small distortion in the tidal curve. Local velocities will change due to the formation of jets and quiescent areas adjacent to the barrage. These will result in erosion and deposition of fine materials. The locations where erosion and deposition potentials will change have been identified.

The impact of short term (1 tide) closure of the barrage on water quality within the barrage lagoon has been examined for various conditions.

References

1. Department of Energy, 1990
Offshore Installations, Guidance on design, construction & certification, HMSO, London.

2. OTH 89,299
Offshore Technology Report, Metocean Parameters (other than waves), HMSO London.

3. BS6349,1984
Code of Practice for Maritime Structures, Part 1, General Criteria.

4. Hunter 1987,
User Manual for Numerical Models of Marine Systems, CMST, Curtin University of Technology.

5. McDowell & O'Connor, 1977
The Hydraulic Behaviour of Estuaries, Macmillan Press

Surveyed 1842
Revised 1935
Levelled 1935

Record PLACE Map

Clonakilty Harbour & Proposed Barrage Location

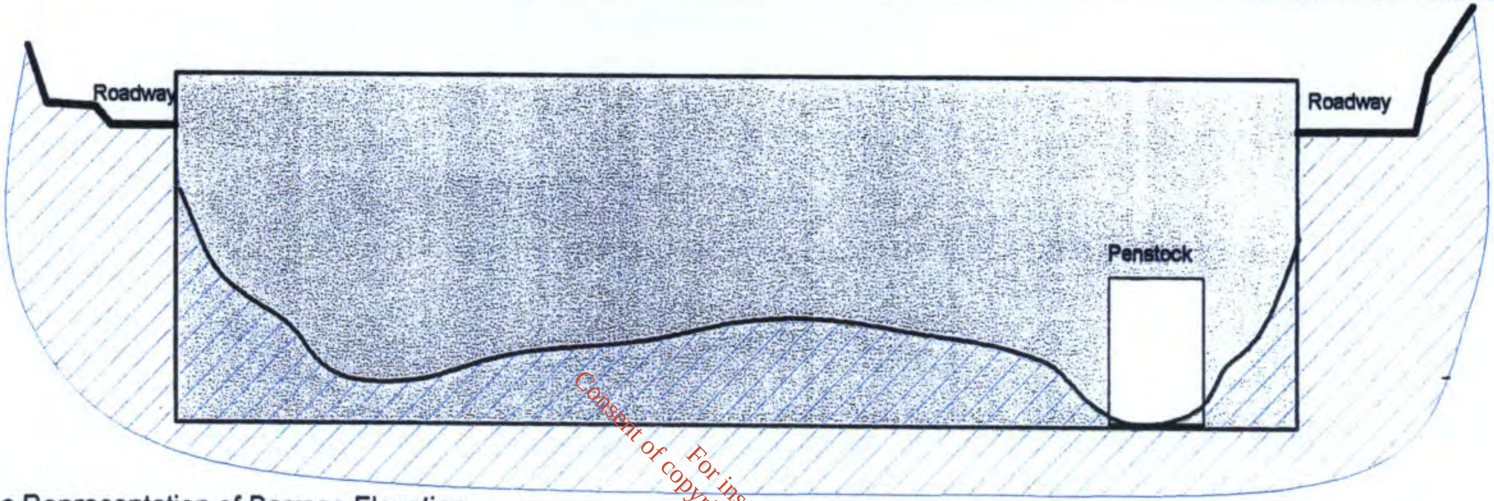
Figure 1.1

Scale:- 1:10560
Scála:- 1:10560
500 400 300 200 100 0 500 Metres
1000 500 0 1000 Feet

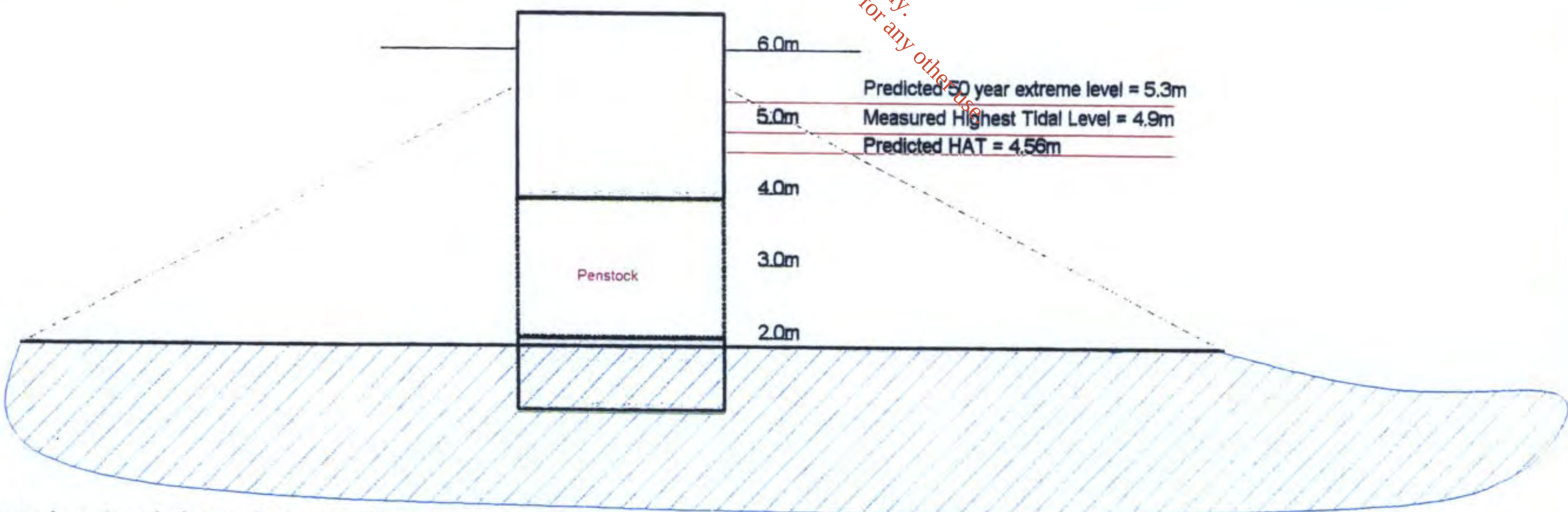
Plot Ref. No. 6220_1
Plot Date 31-OCT-2000

6inch
CK135

Clonakilty Tidal Barrage Study 2000



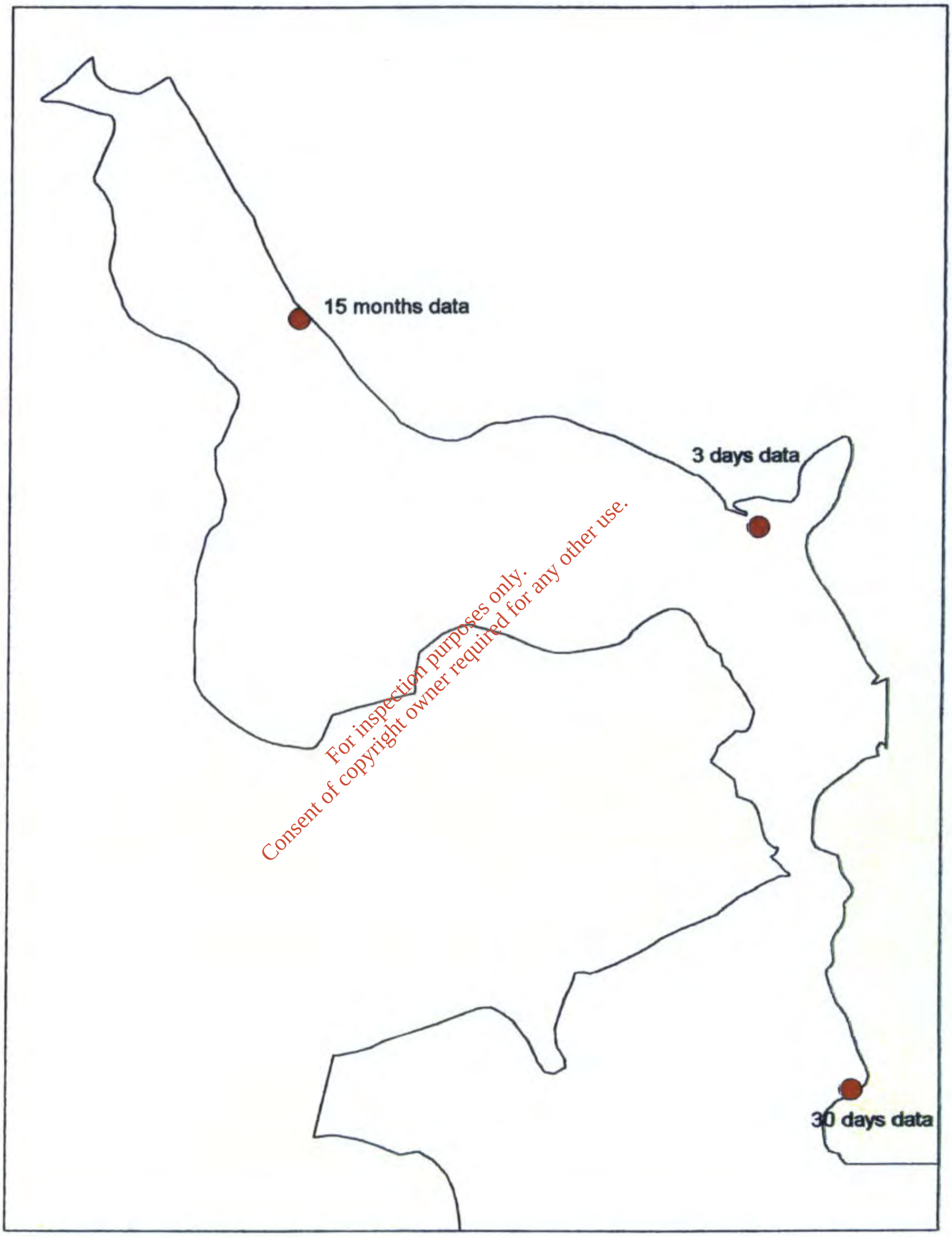
Schematic Representation of Barrage Elevation



Barrage Levels relative to Ordnance Datum

Figure No 1.2

Clonakilty Tidal Barrage Study 2000



● Recording Tide Gauge Location

Figure No 2.1

CLONAKILTY TIDAL BARRAGE STUDY

OBSERVED TIDE LEVELS AT PIER AT ENTRANCE TO HARBOUR (O.D. POOLBEG)

RESIDUAL TIDE FROM HARMONIC ANALYSIS OF OBSERVED TIDE

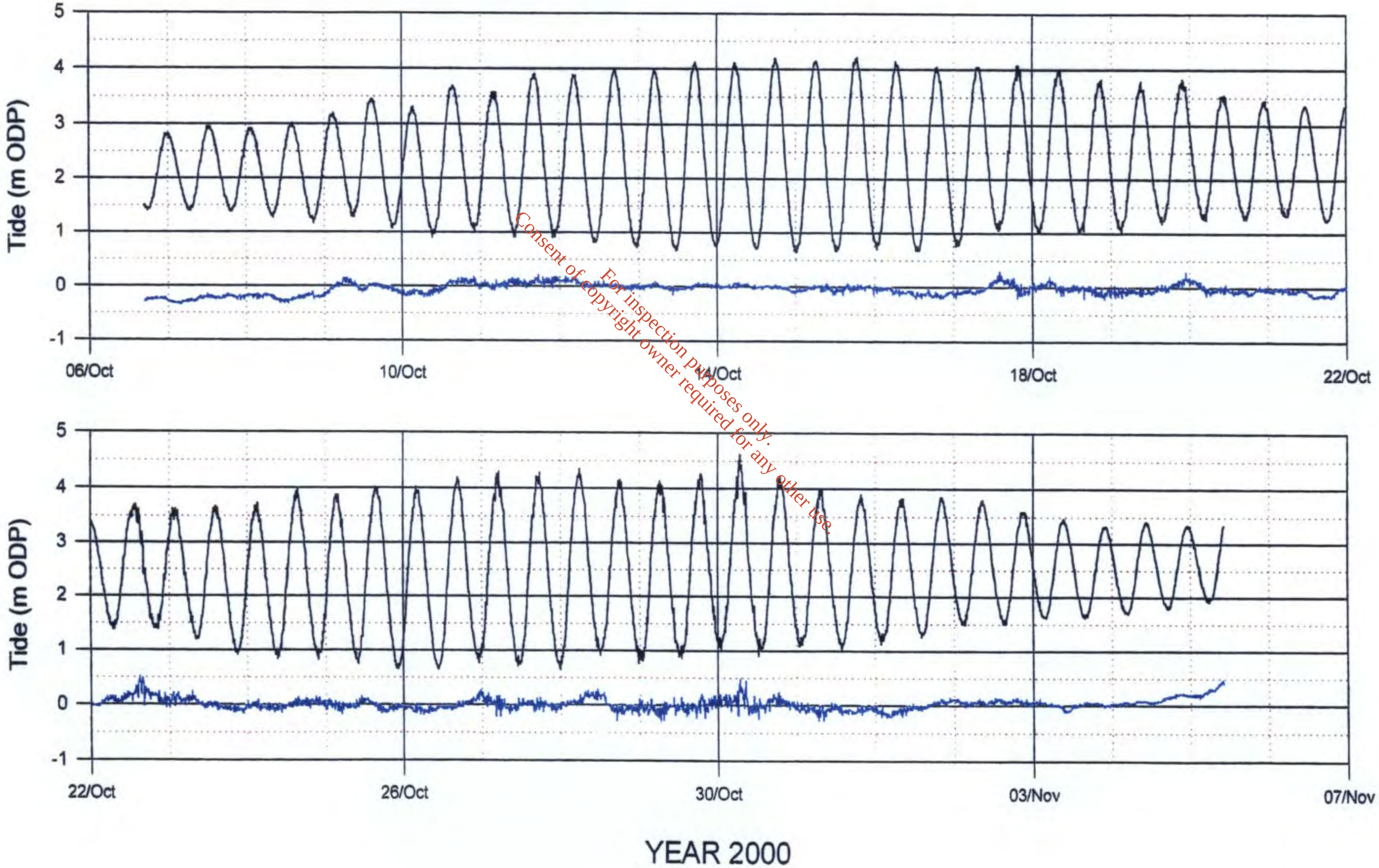


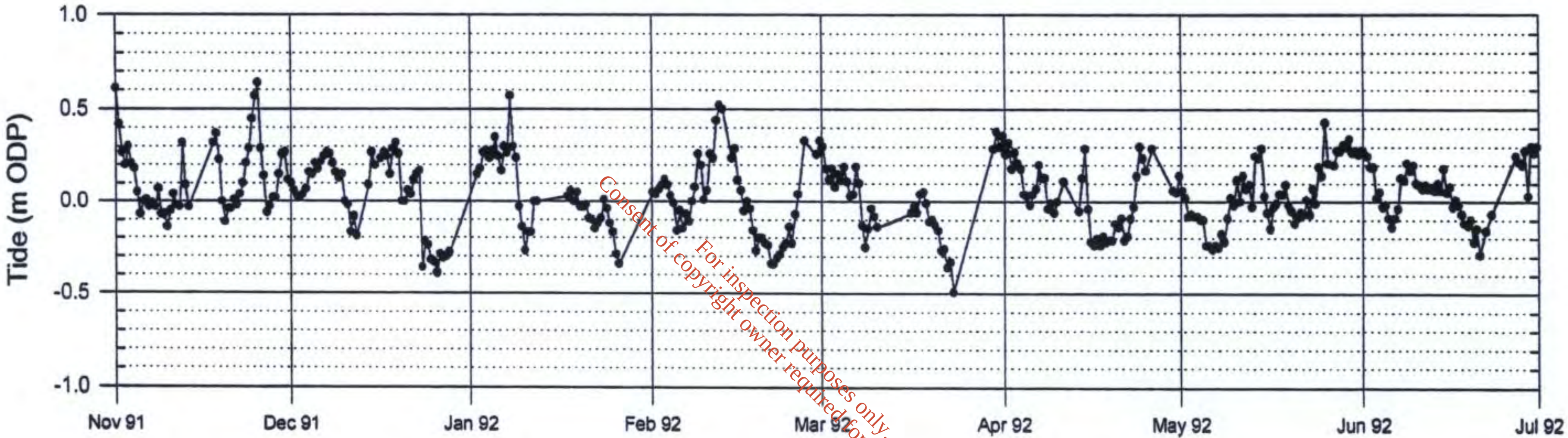
Fig. 2.2

CLONAKILTY TIDAL BARRAGE STUDY

TIME SERIES OF TIDAL RESIDUALS OCCURRING AT HW's

(Observed HW - Predicted HW)

NOVEMBER 1991 to JANUARY 1993 inclusive



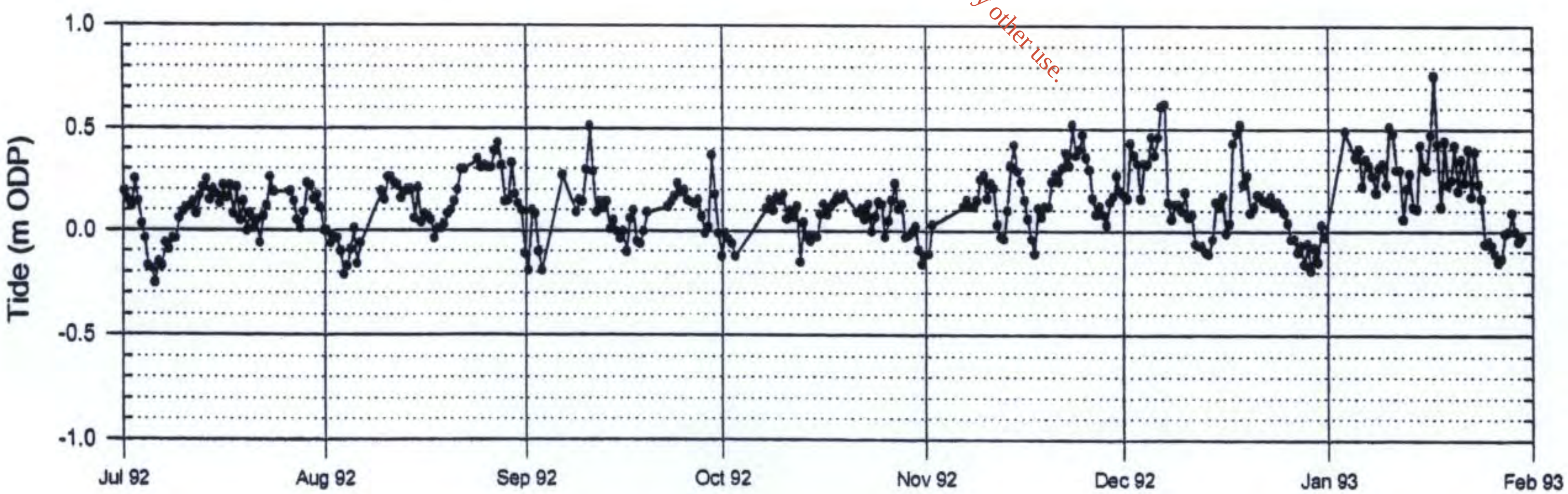


Fig. 2.3

CLONAKILTY TIDAL BARRAGE STUDY

PREDICTION OF EXTREME VALUES OF TIDAL RESIDUALS

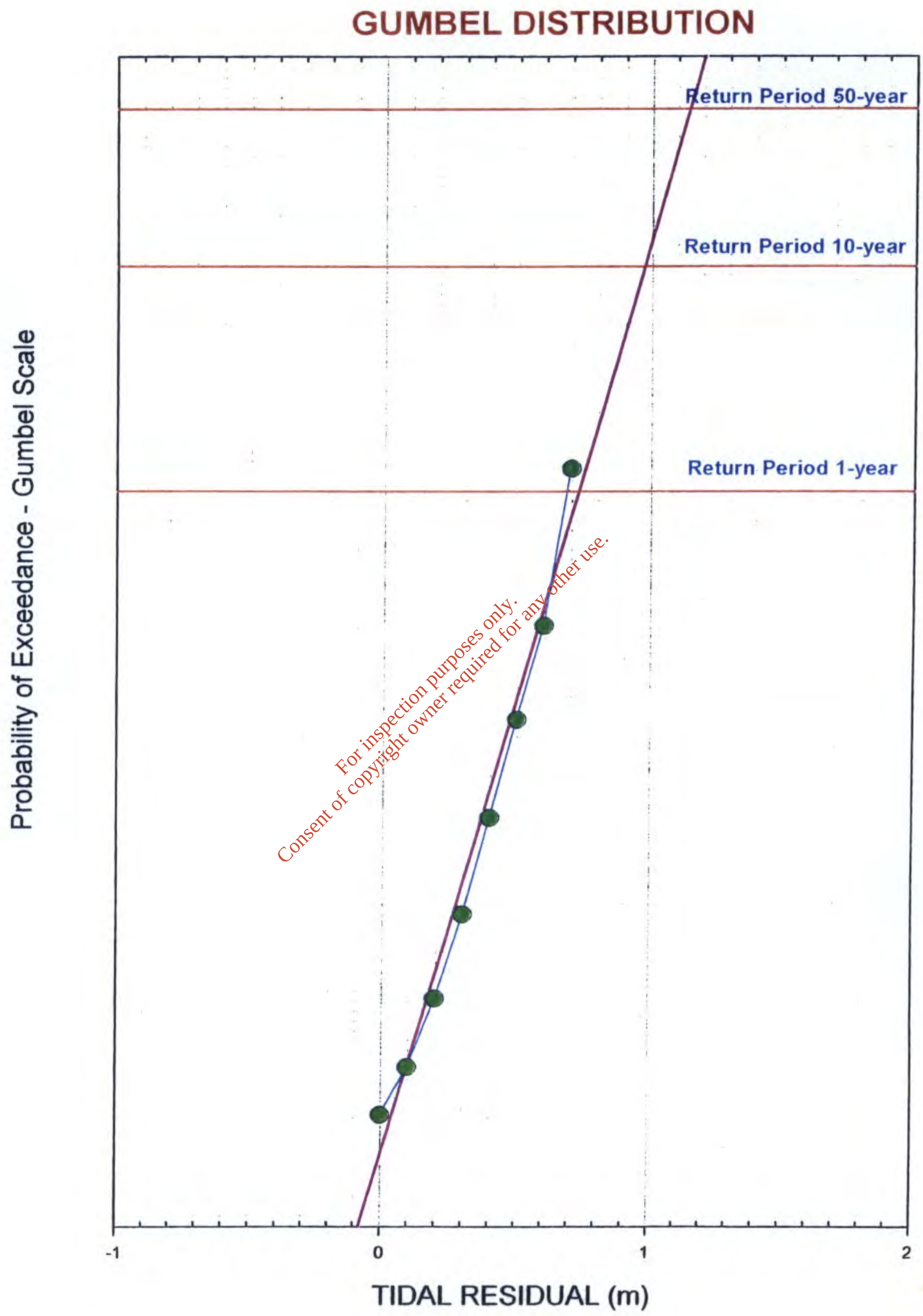
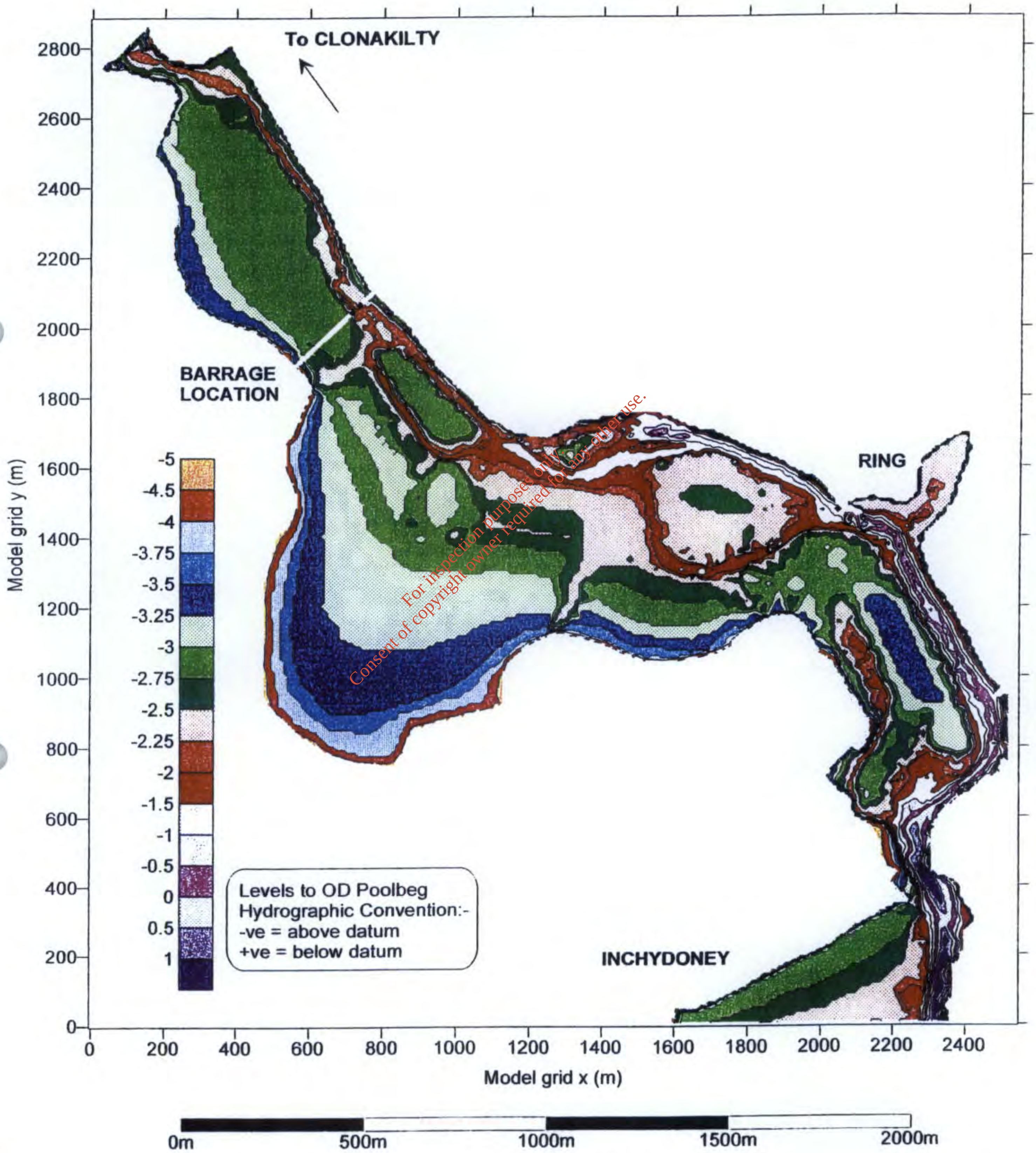


Fig. 2.4

CLONAKILTY TIDAL BARRAGE STUDY

MODEL DEPTH GRID (O.D. Poolbeg)



Scale 1 : 15,000

Fig. 3.1

CLONAKILTY TIDAL BARRAGE STUDY

FLOW FIELD FOR EXISTING SITUATION (NO BARRAGE)

SPRING TIDE

MAX FLOOD (HW-1.5hrs)

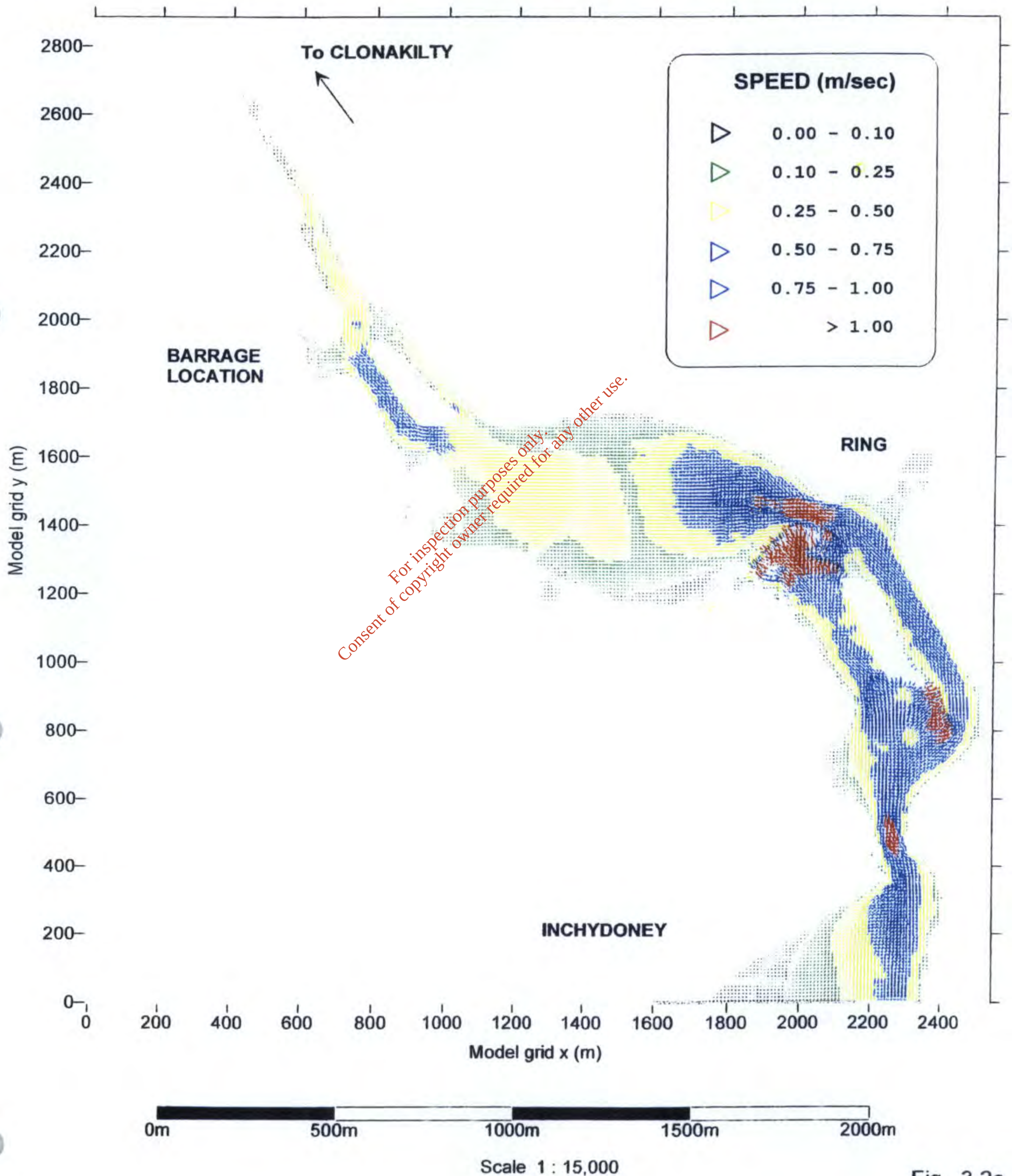


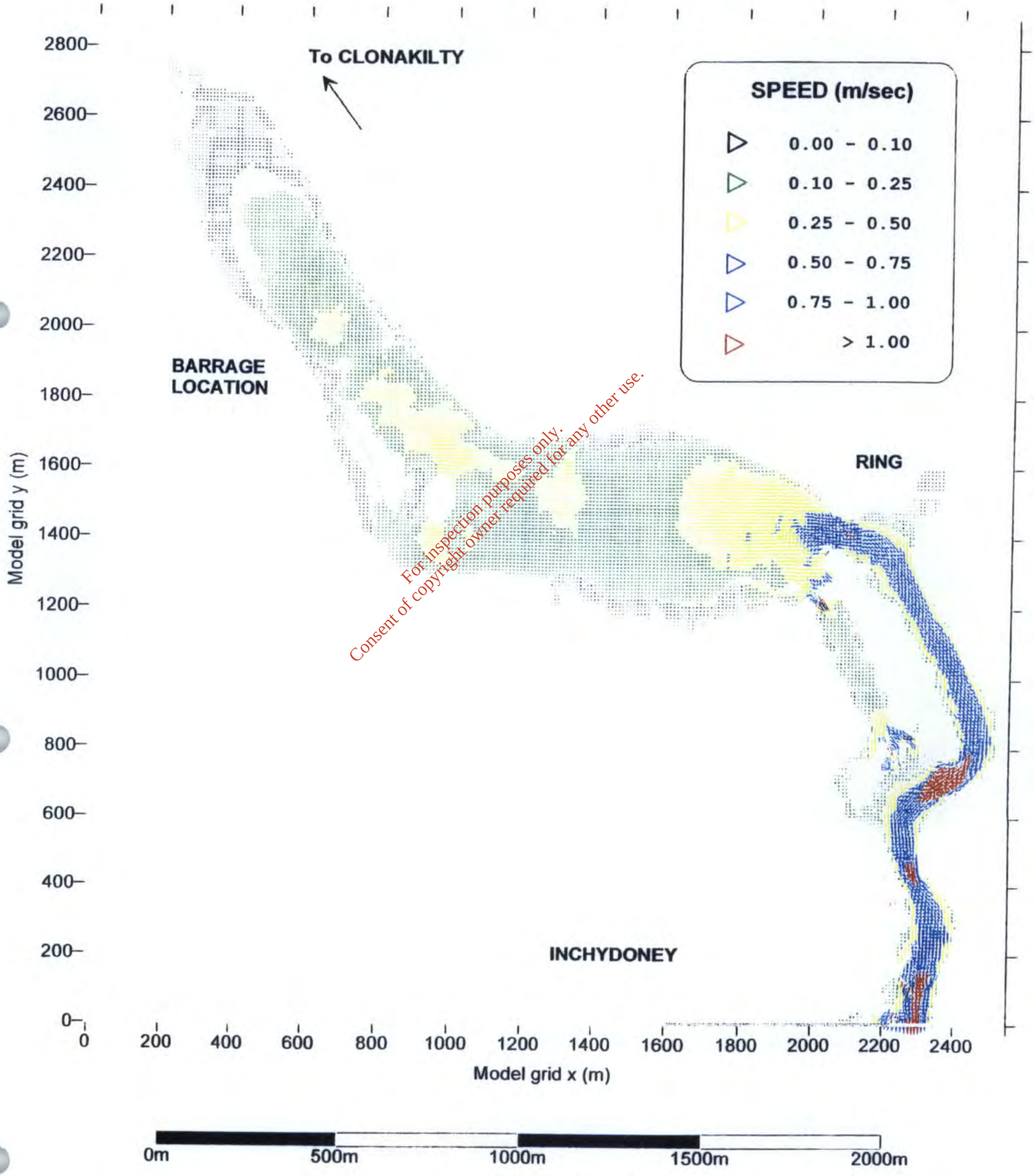
Fig. 3.2a

CLONAKILTY TIDAL BARRAGE STUDY

FLOW FIELD FOR EXISTING SITUATION (NO BARRAGE)

SPRING TIDE

MAX EBB (HW+3hrs)



Scale 1 : 15,000

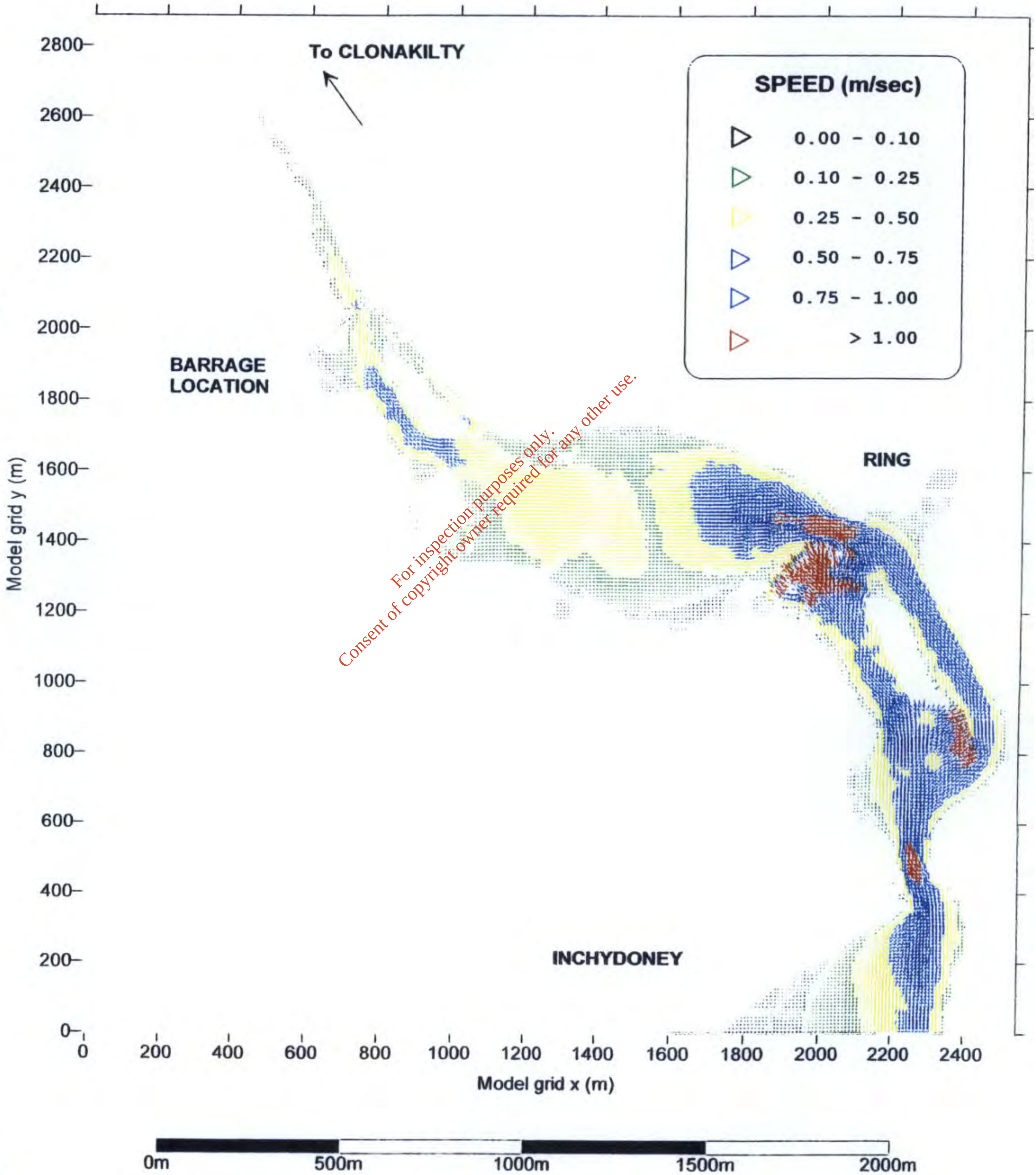
Fig. 3.2b

CLONAKILTY TIDAL BARRAGE STUDY

FLOW FIELD FOR PROPOSED SITUATION (WITH BARRAGE)

SPRING TIDE

MAX FLOOD (HW-1.5hrs)



Scale 1 : 15,000

Fig. 3.3a

CLONAKILTY TIDAL BARRAGE STUDY

FLOW FIELD FOR PROPOSED SITUATION (WITH BARRAGE)

SPRING TIDE

MAX EBB (HW+3hrs)

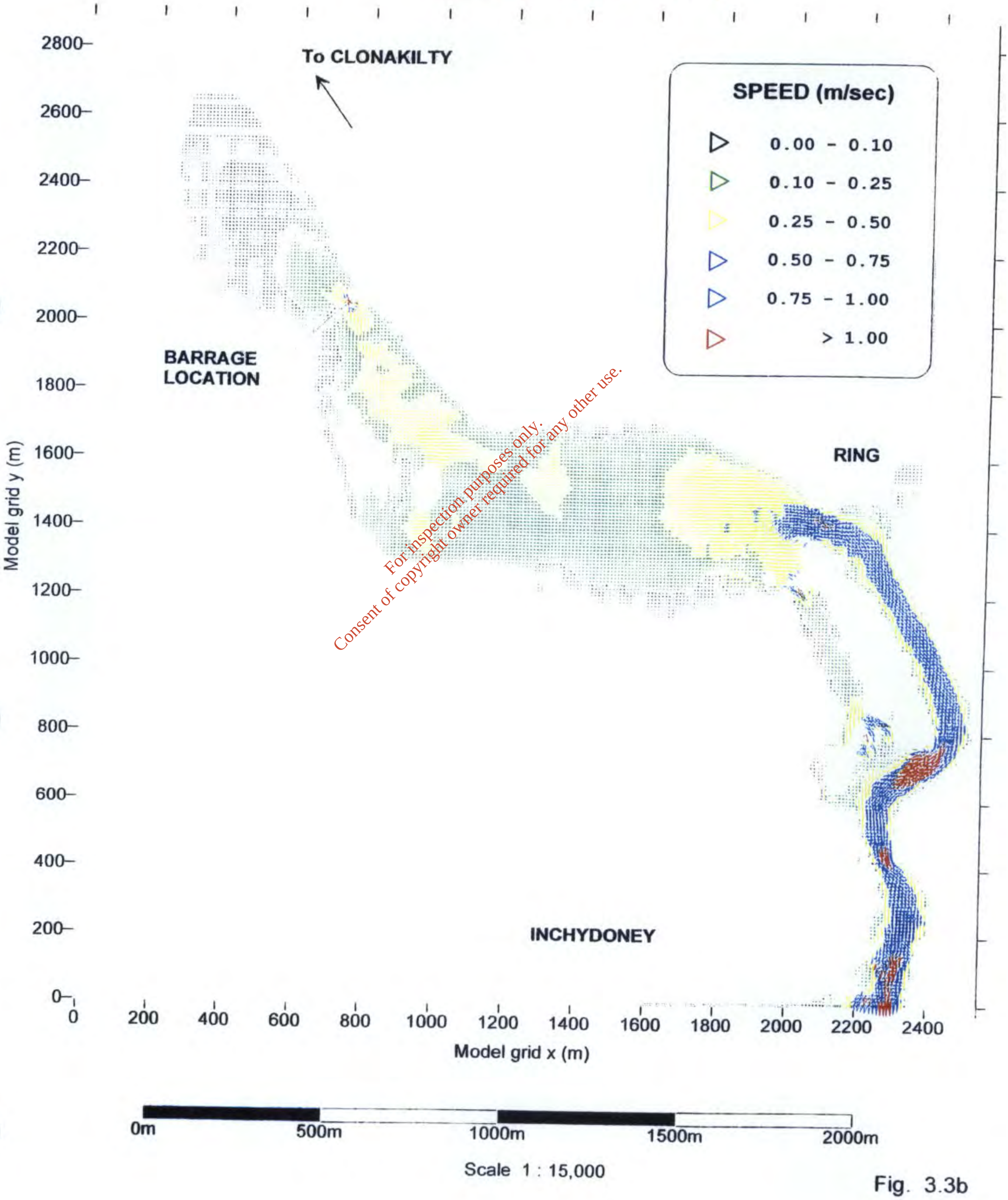


Fig. 3.3b

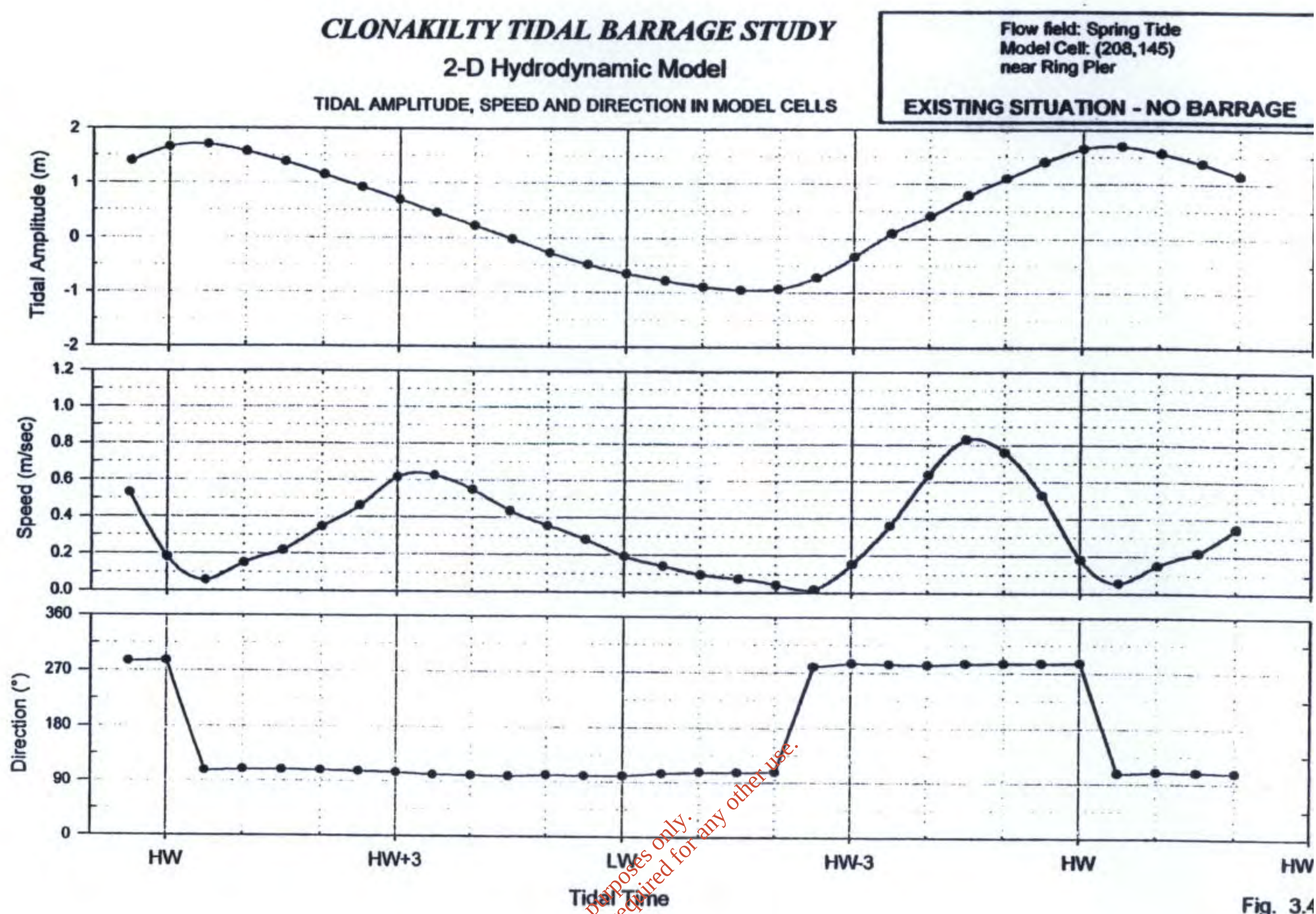


Fig. 3.4

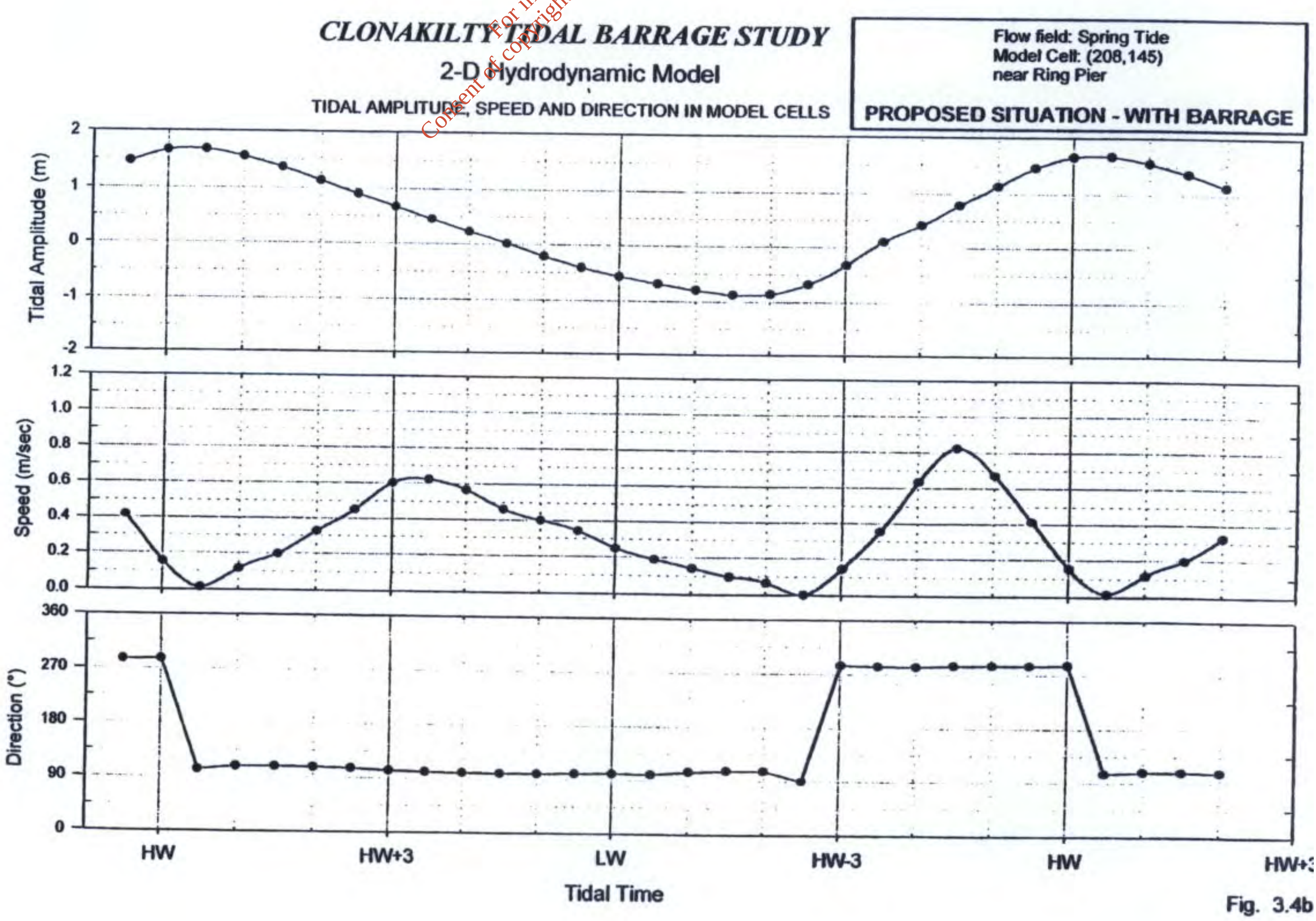


Fig. 3.4b

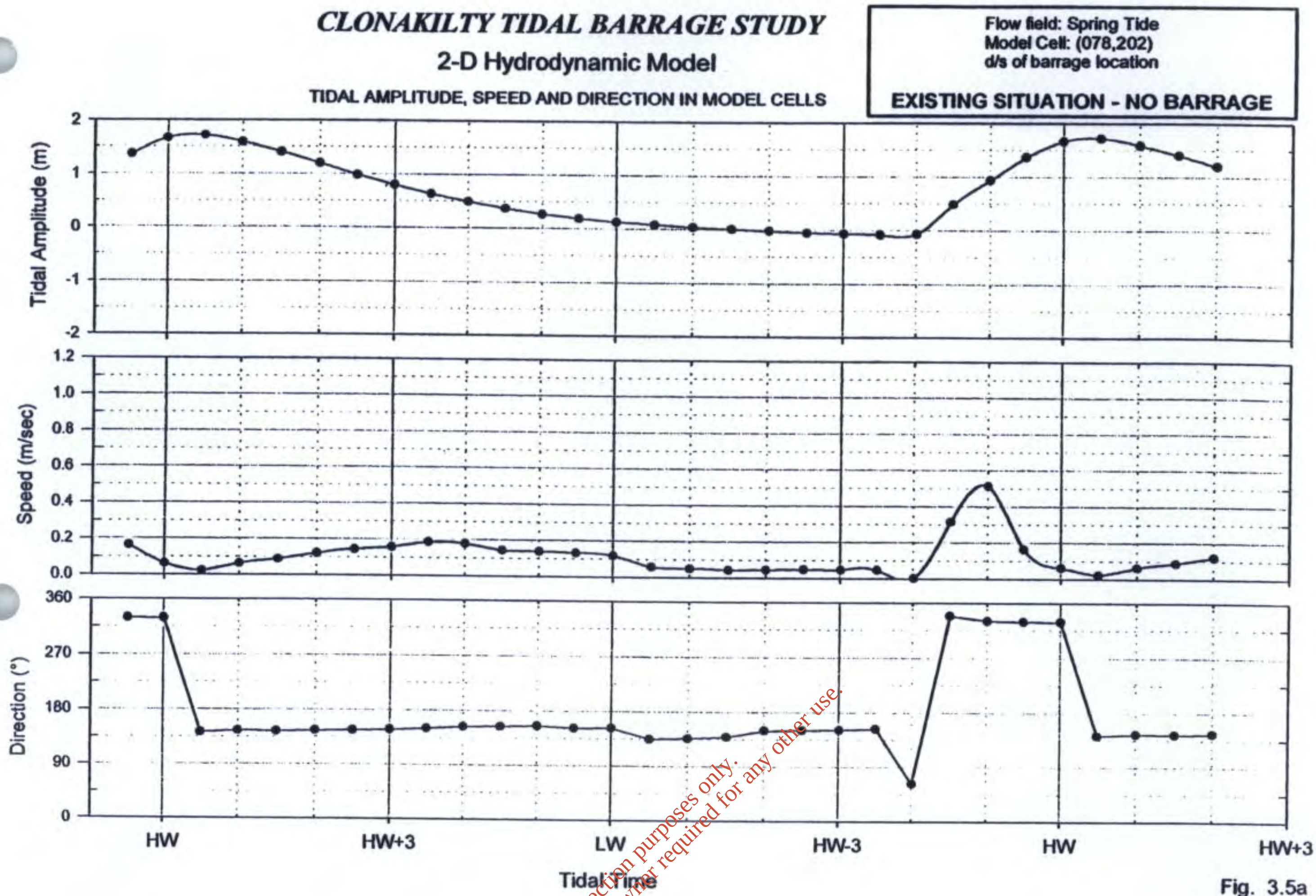


Fig. 3.5a

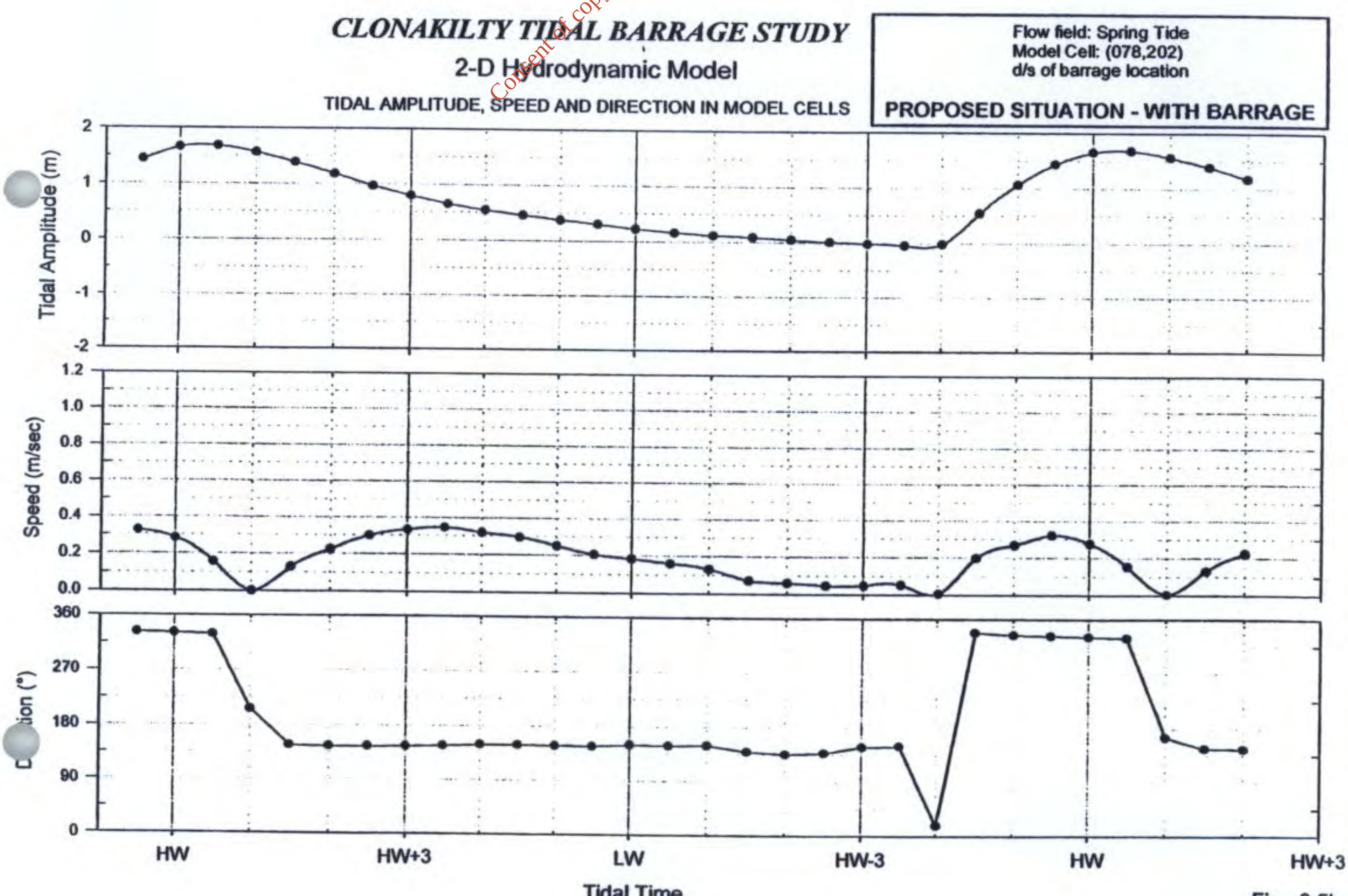


Fig. 3.5b

CLONAKILTY TIDAL BARRAGE STUDY
2-D Hydrodynamic Model

TIDAL AMPLITUDE, SPEED AND DIRECTION IN MODEL CELLS

Flow field: Spring Tide
Model Cell: (069,211)
u/s of barrage location

EXISTING SITUATION - NO BARRAGE

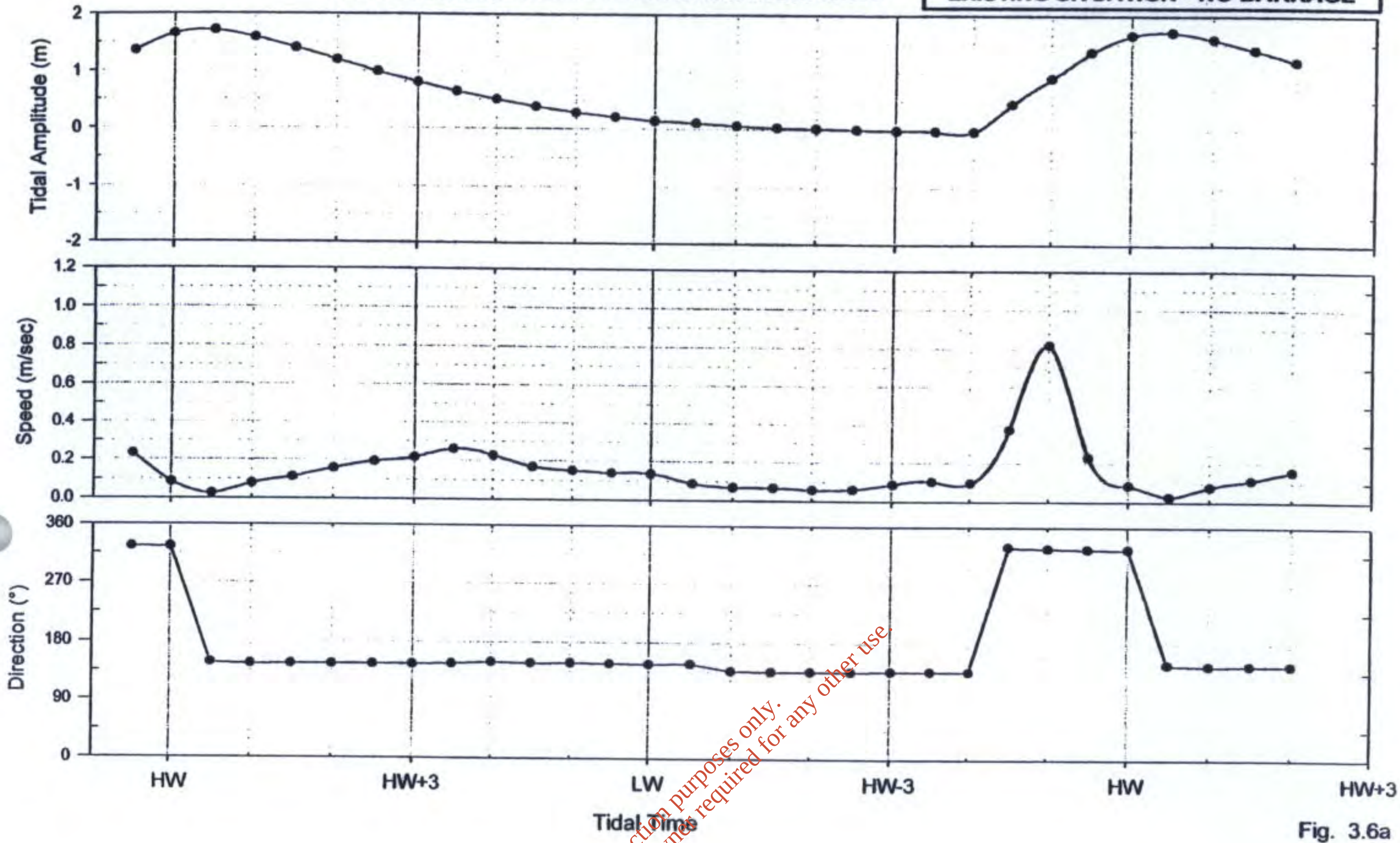


Fig. 3.6a

CLONAKILTY TIDAL BARRAGE STUDY
2-D Hydrodynamic Model

TIDAL AMPLITUDE, SPEED AND DIRECTION IN MODEL CELLS

Flow field: Spring Tide
Model Cell: (069,211)
u/s of barrage location

PROPOSED SITUATION - WITH BARRAGE

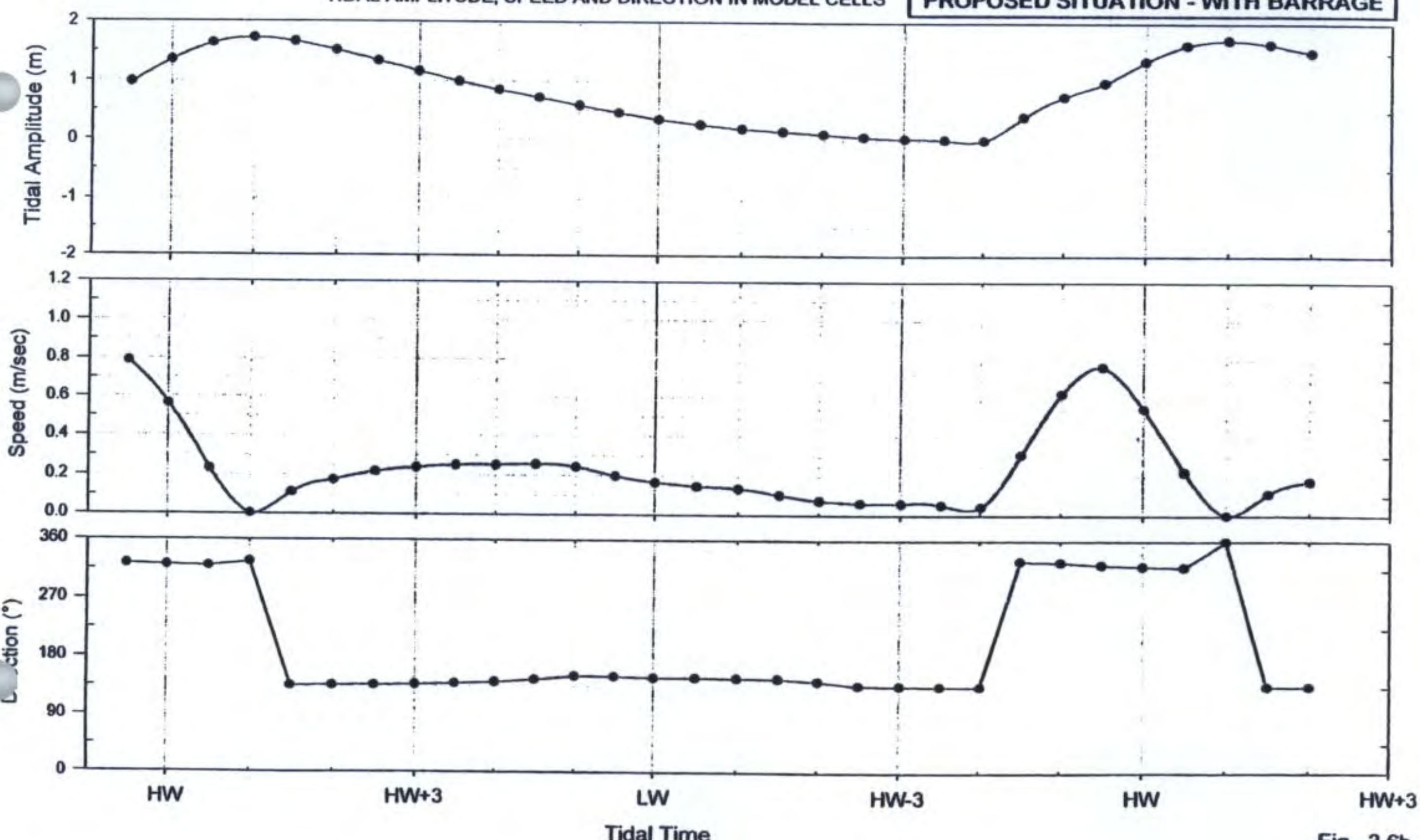


Fig. 3.6b

CLONAKILTY TIDAL BARRAGE STUDY

AREAS OF INCREASED EROSION POTENTIAL ON FLOODING TIDE

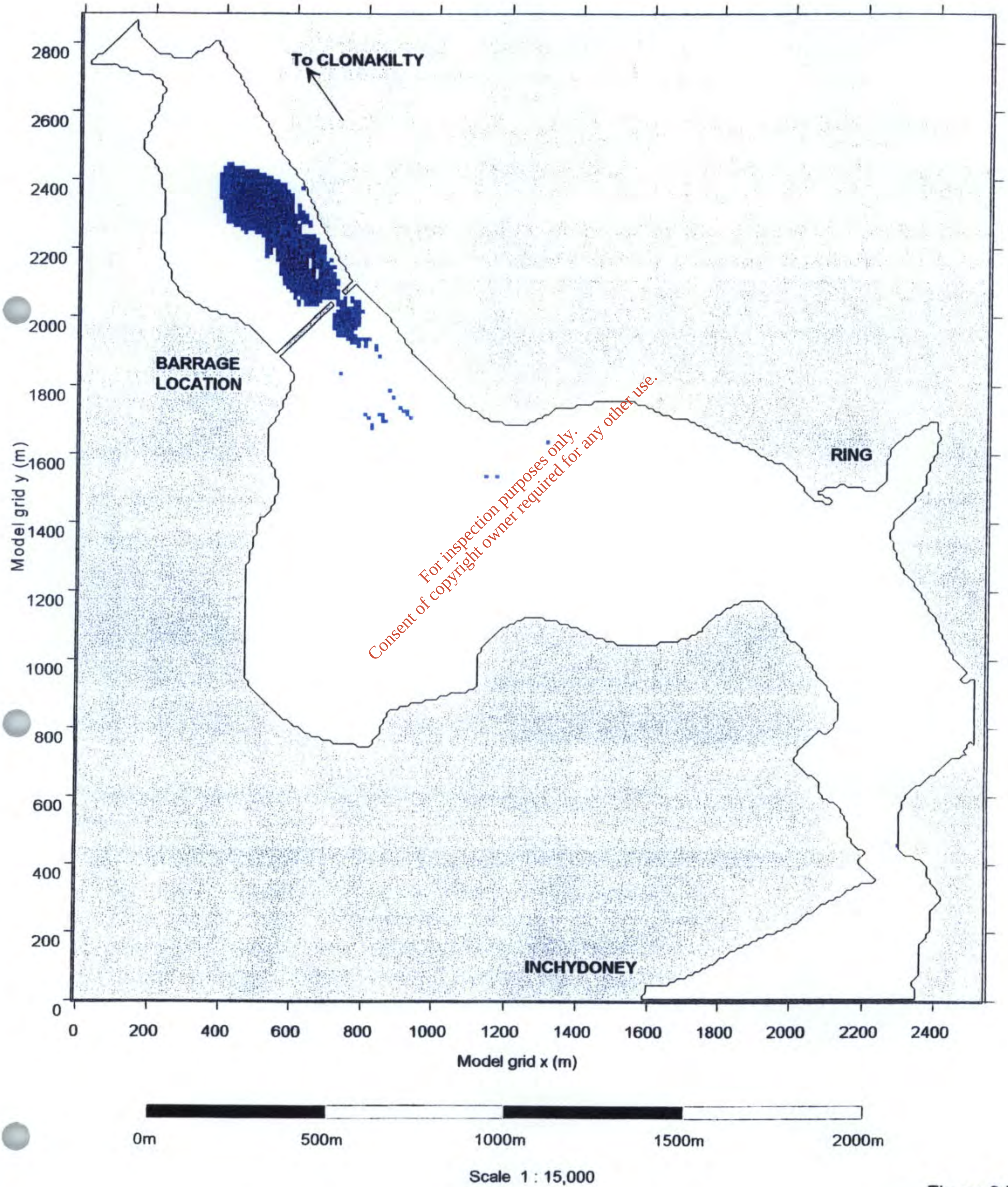
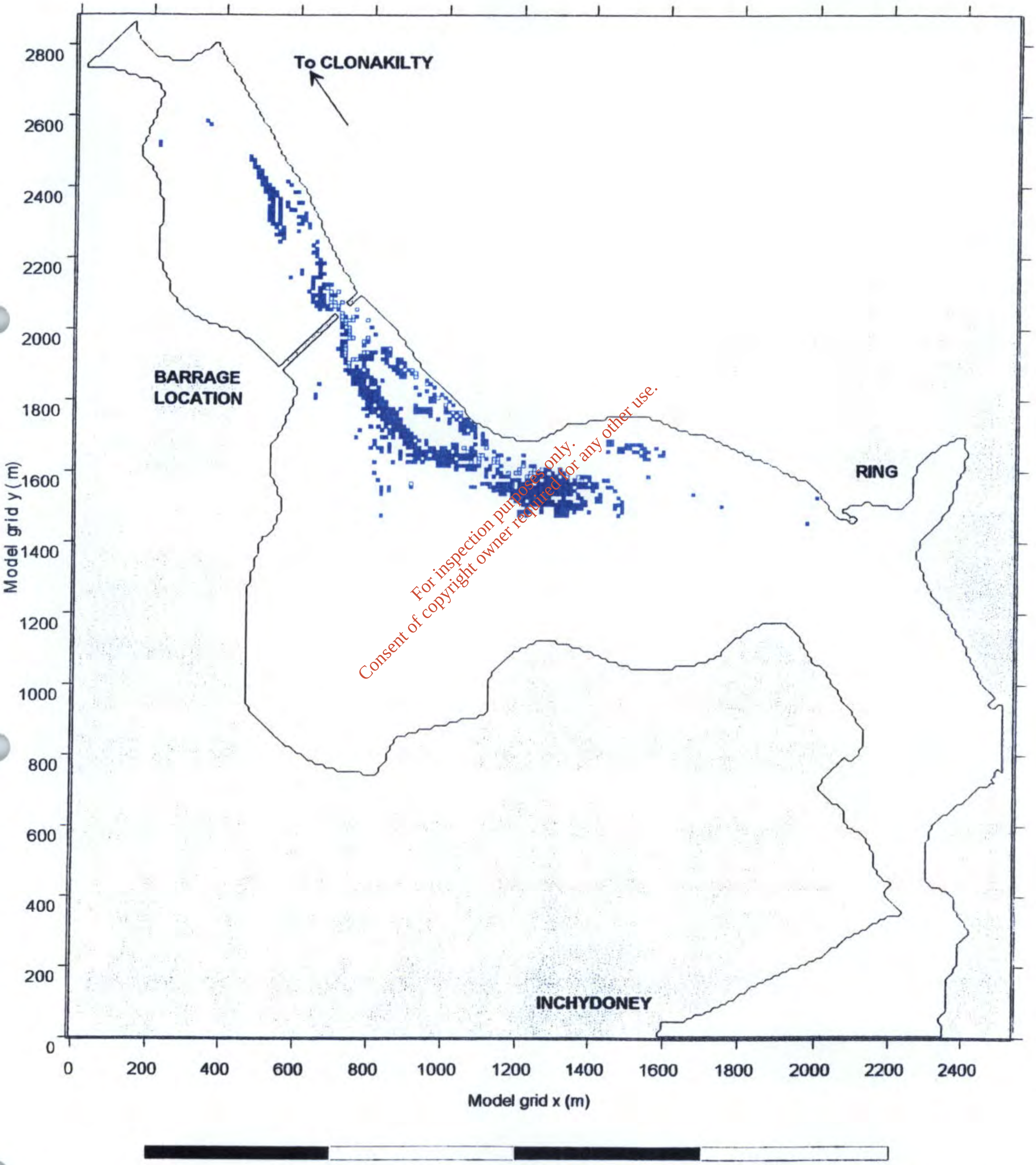


Figure 3.7a

CLONAKILTY TIDAL BARRAGE STUDY

AREAS OF INCREASED EROSION POTENTIAL ON EBBING TIDE



Scale 1 : 15,000

Figure 3.7b

CLONAKILTY TIDAL BARRAGE STUDY

AREAS OF INCREASED DEPOSITION POTENTIAL ON FLOODING TIDE

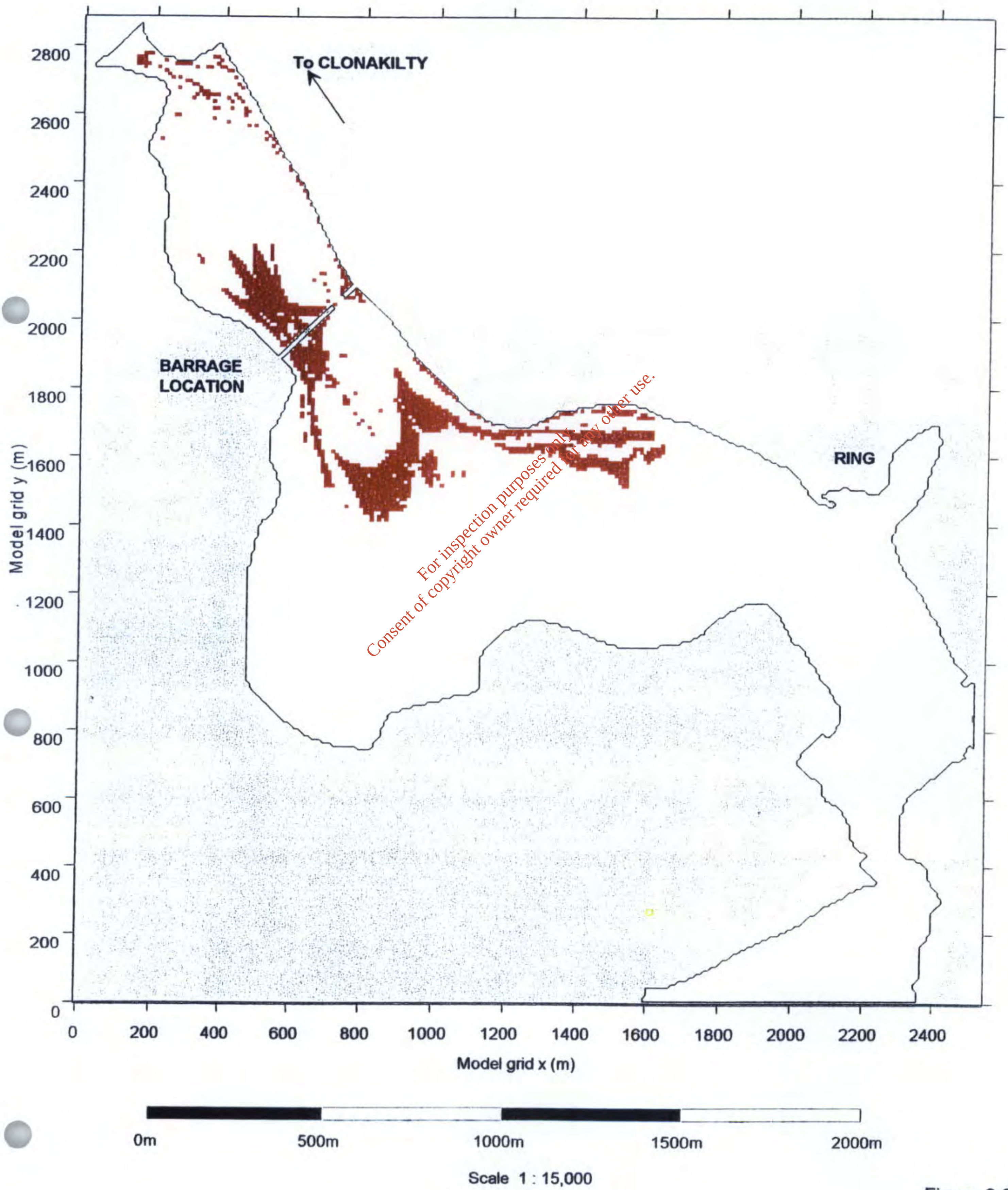


Figure 3.8a

CLONAKILTY TIDAL BARRAGE STUDY

AREAS OF INCREASED DEPOSITION POTENTIAL ON EBBING TIDE

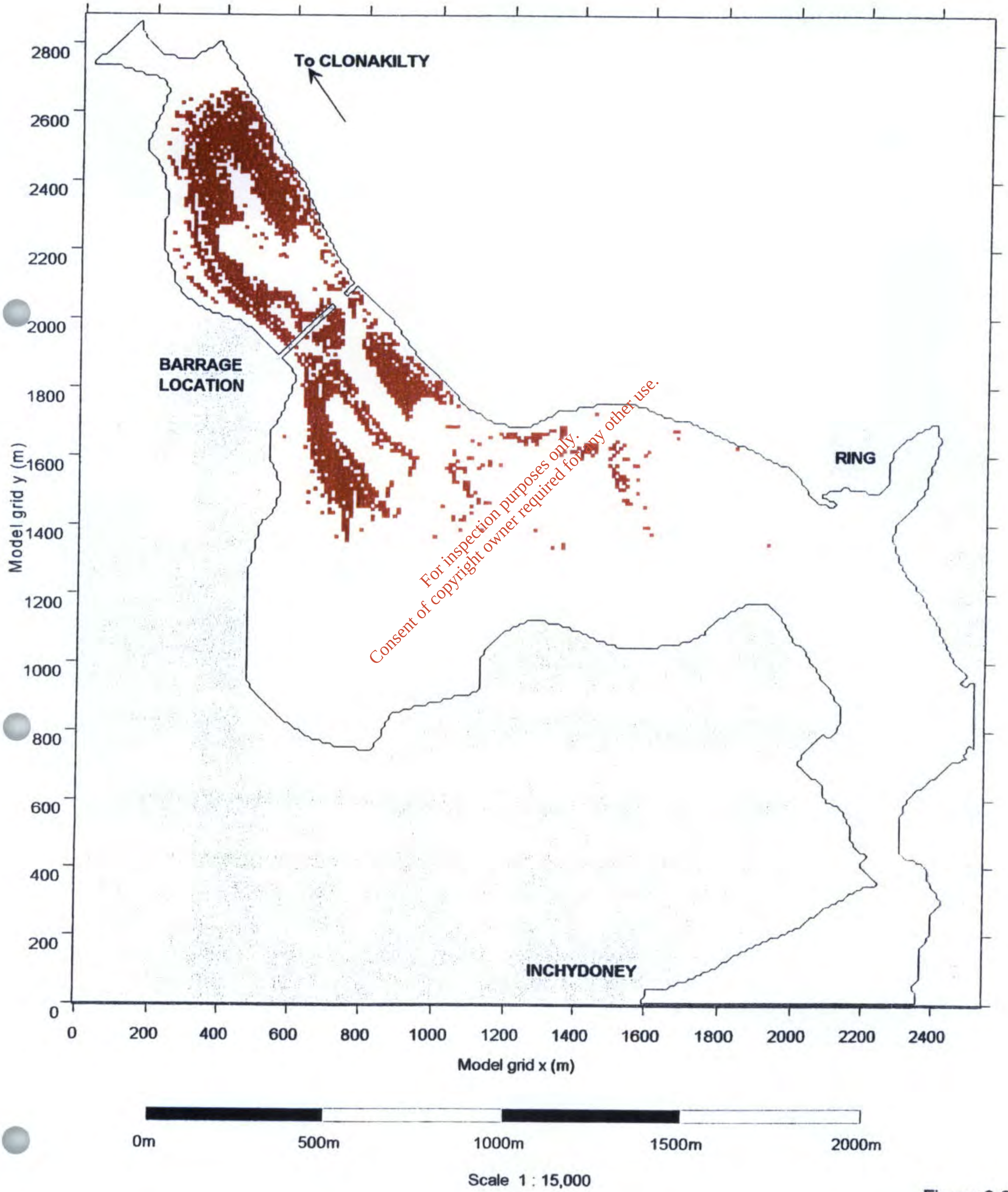


Figure 3.8b

CLONAKILTY TIDAL BARRAGE STUDY

AREAS OF POTENTIAL EROSION & DEPOSITION

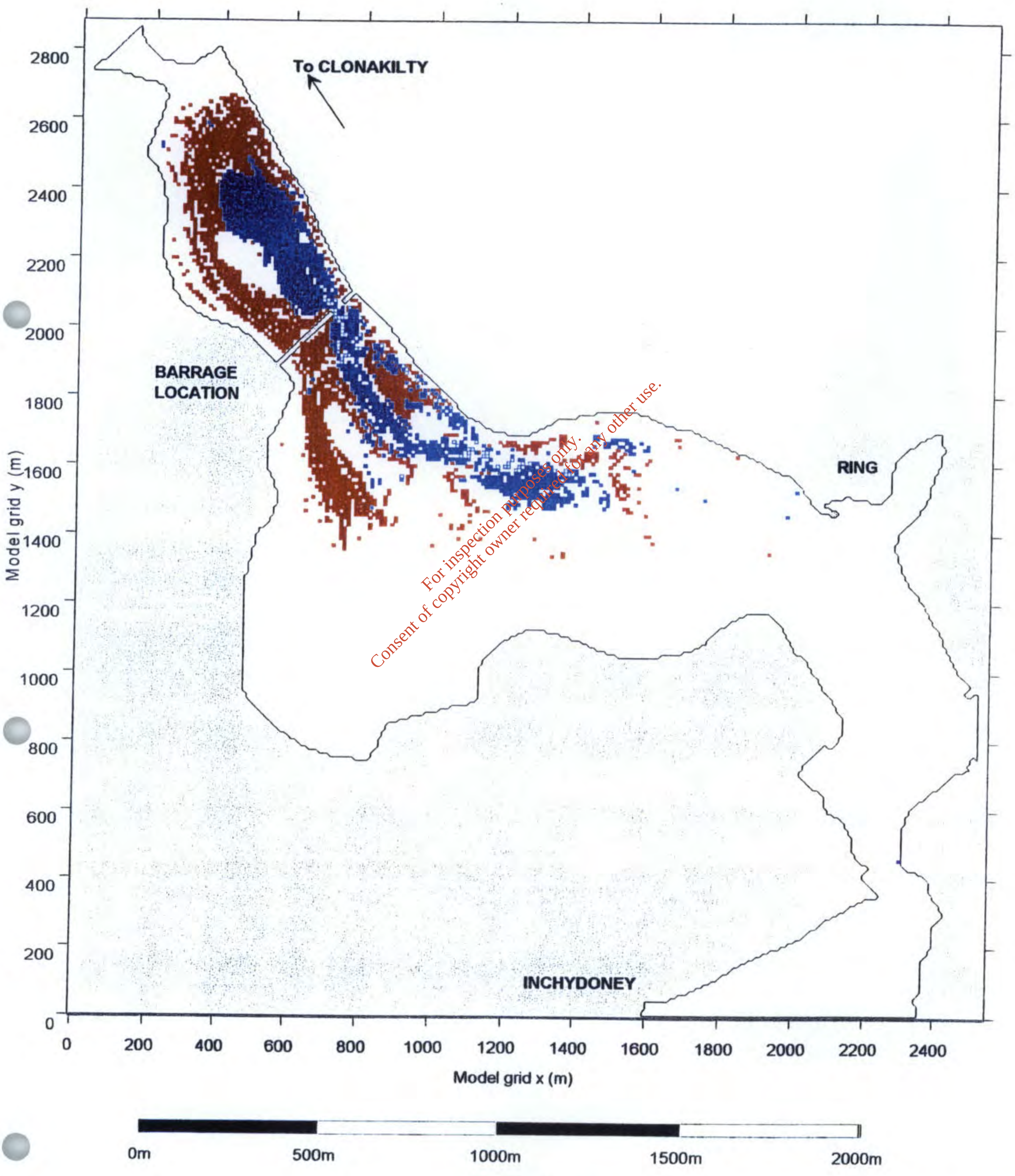


Figure 3.9

Clonakilty Tidal Barrage Study 2000

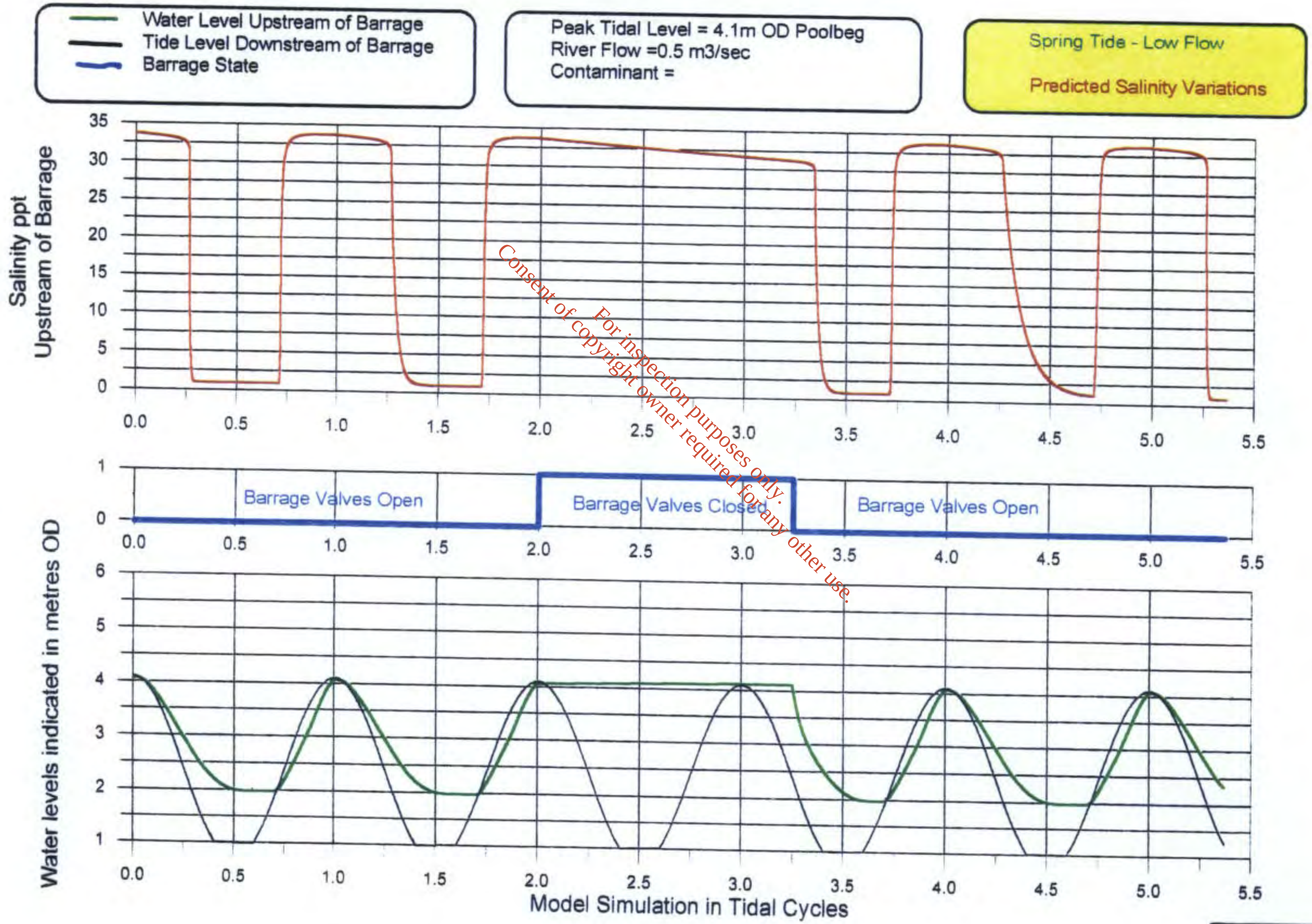
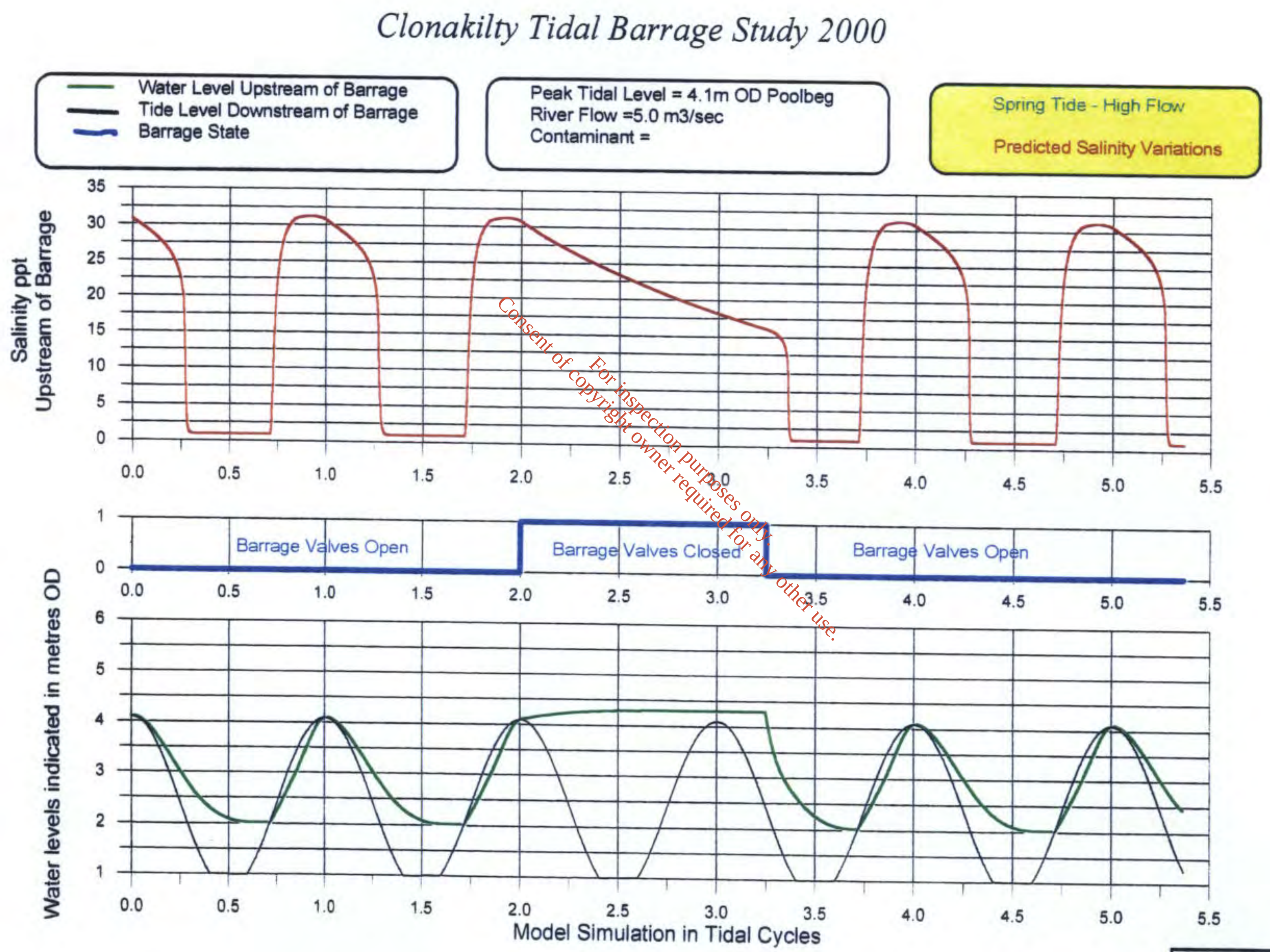


Figure 4.1



Clonakilty Tidal Barrage Study 2000
Water Level Upstream of Barrage
Tide Level Downstream of Barrage
Barrage State
Peak Tidal Level = 4.1m OD Poolbeg
River Flow =5.0 m3/sec
Contaminant =
Spring Tide - High Flow
Predicted Salinity Variations
Salinity ppt
Upstream of Barrage
Barrage Valves Open
Barrage Valves Closed
Barrage Valves Open
Water levels indicated in metres OD
Model Simulation in Tidal Cycles

Figure 4.2

Clonakilty Tidal Barrage Study 2000

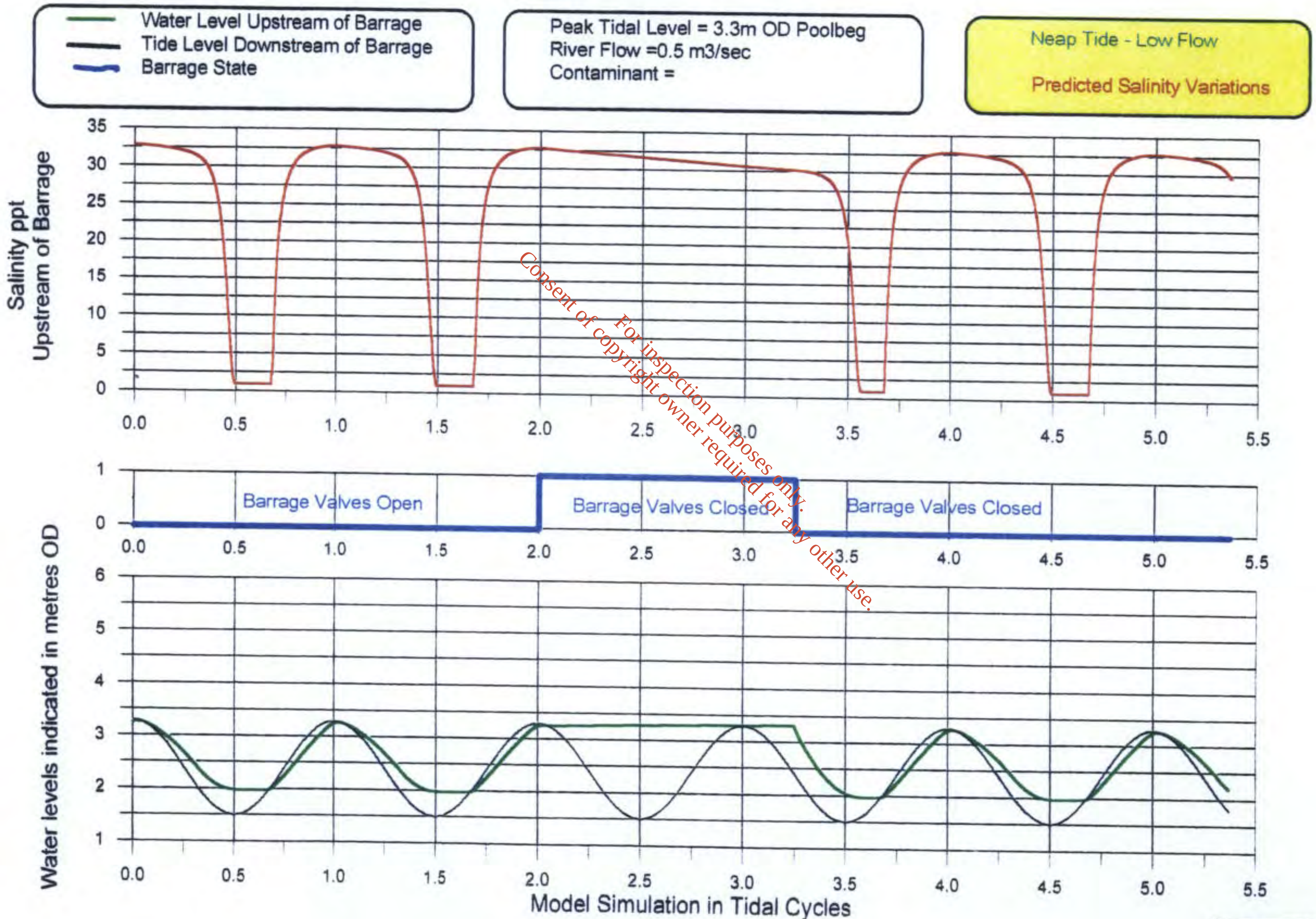
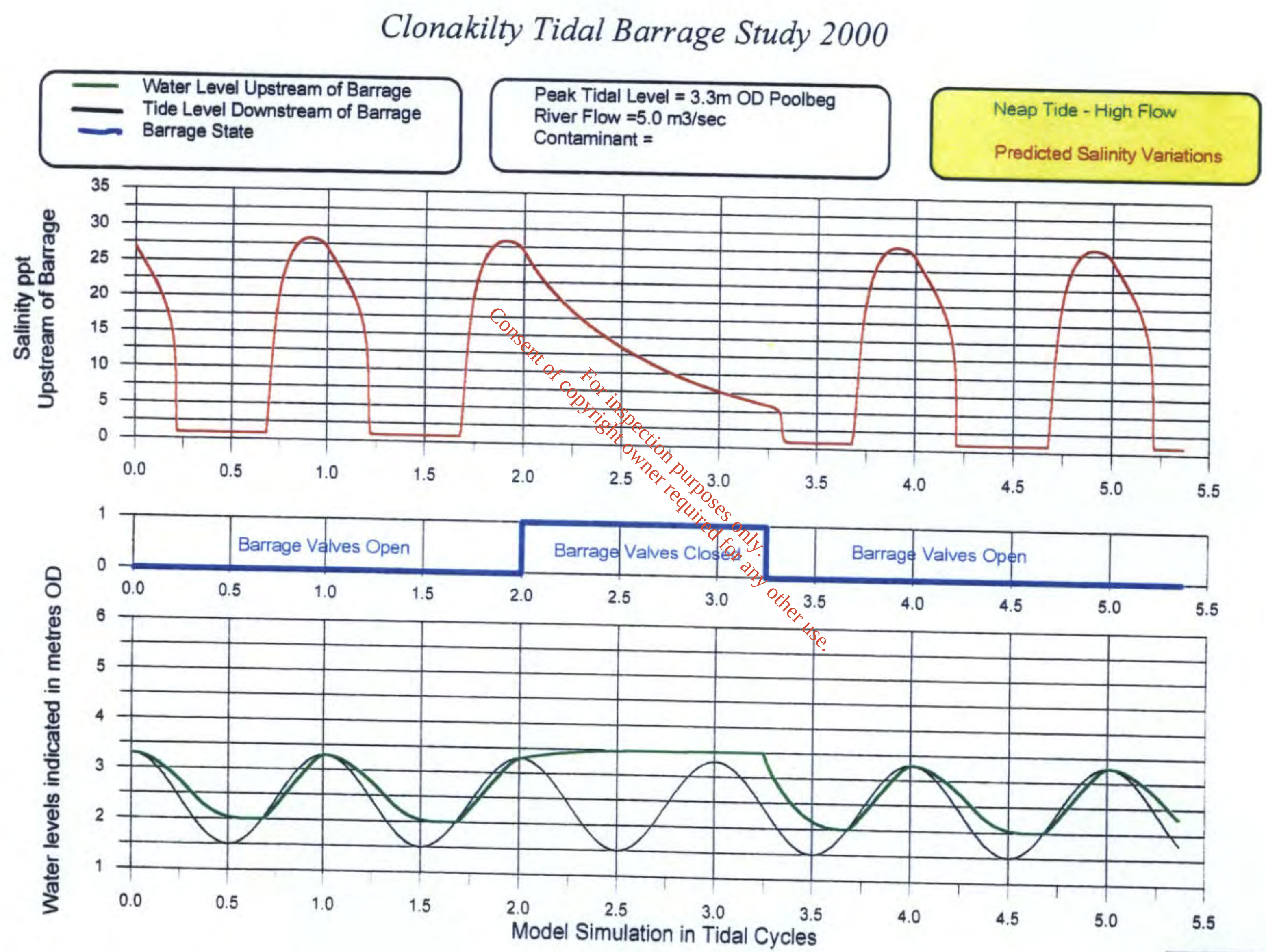


Figure 4.3



Clonakilty Tidal Barrage Study 2000
Water Level Upstream of Barrage
Tide Level Downstream of Barrage
Barrage State
Peak Tidal Level = 3.3m OD Poolbeg
River Flow =5.0 m3/sec
Contaminant =
Neap Tide - High Flow
Predicted Salinity Variations
Salinity ppt Upstream of Barrage
35
30
25
20
15
10
5
0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
1
0
Barrage Valves Open
Barrage Valves Closed
Barrage Valves Open
Water levels indicated in metres OD
6
5
4
3
2
1
Model Simulation in Tidal Cycles

Figure 4.4

Clonakilty Tidal Barrage Study 2000

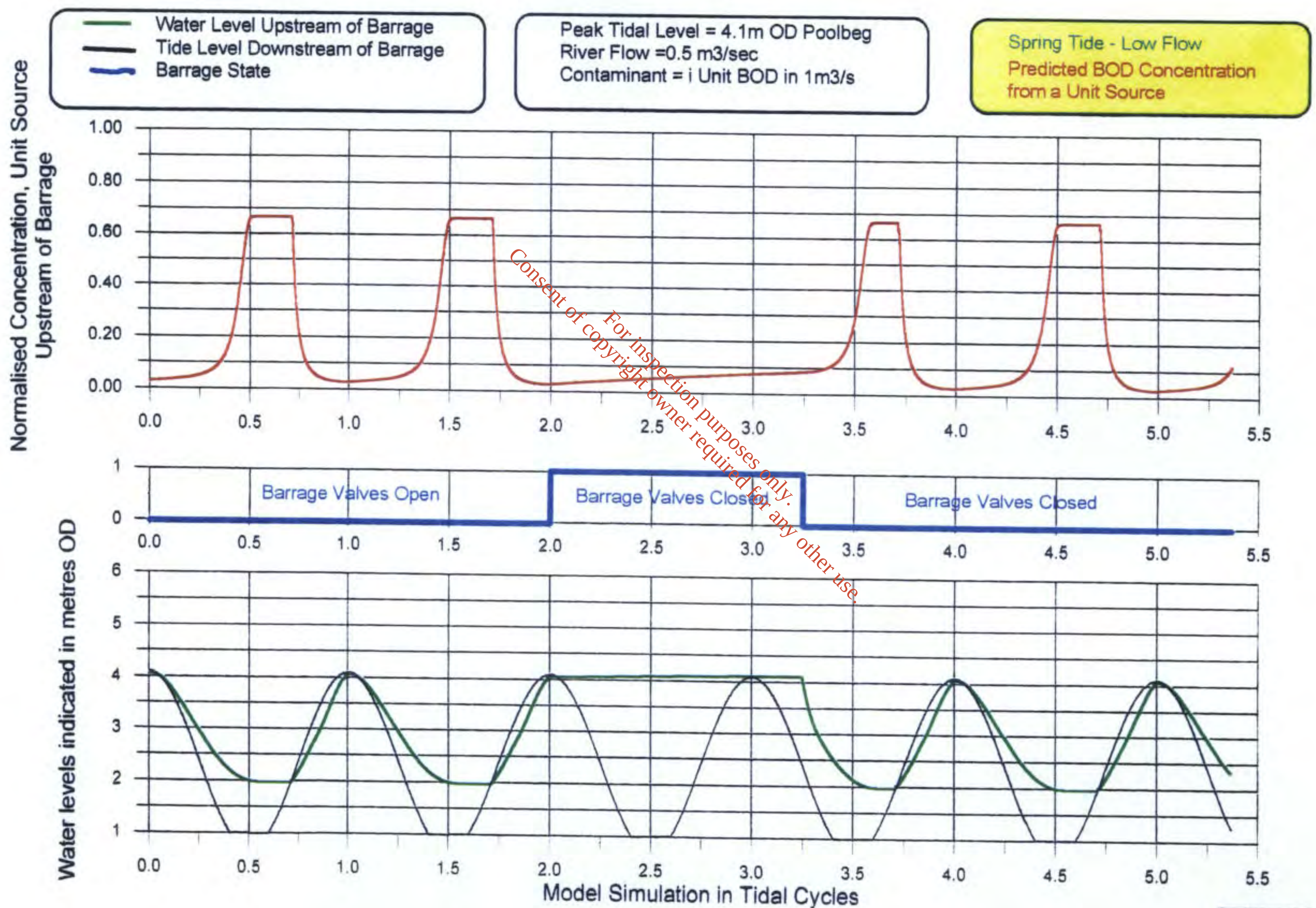


Figure 4.5

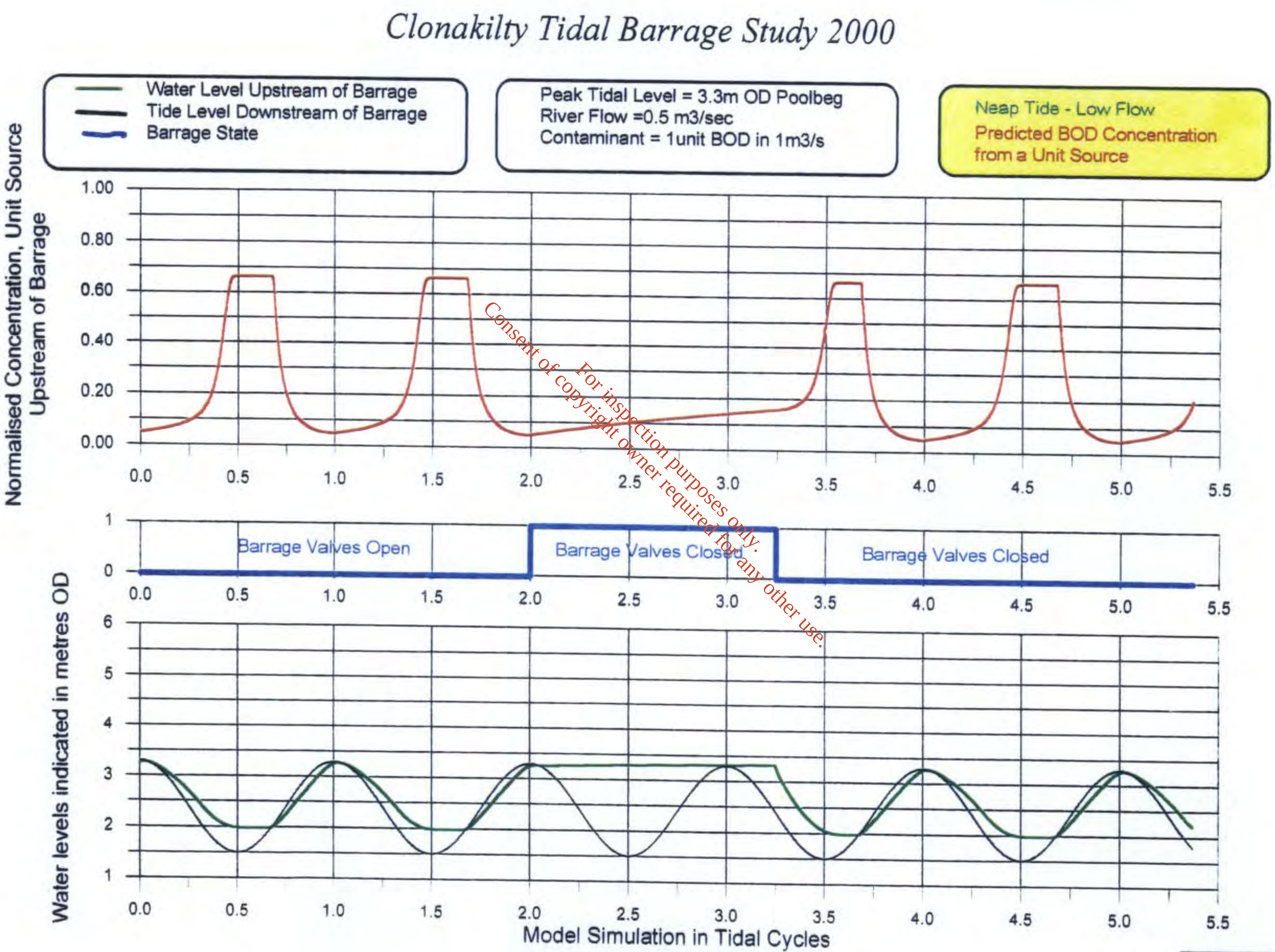

Clonakilty Tidal Barrage Study 2000
Water Level Upstream of Barrage
Tide Level Downstream of Barrage
Barrage State
Peak Tidal Level = 3.3m OD Poolbeg
River Flow =0.5 m3/sec
Contaminant = 1unit BOD in 1m3/s
Neap Tide - Low Flow
Predicted BOD Concentration from a Unit Source
Normalised Concentration, Unit Source Upstream of Barrage
Barrage Valves Open
Barrage Valves Closed
Barrage Valves Closed
Water levels indicated in metres OD
Model Simulation in Tidal Cycles

Figure 4.6

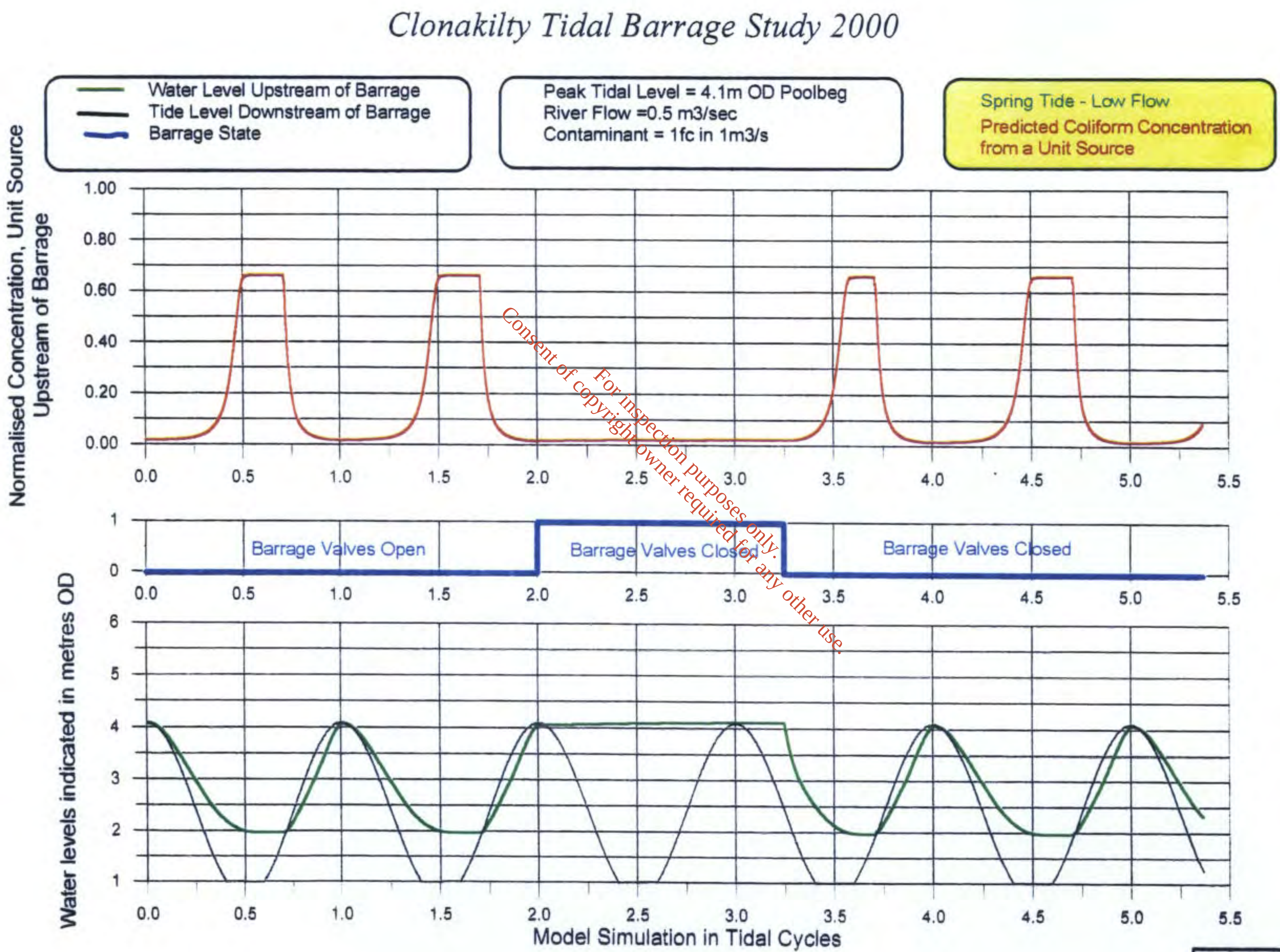

Clonakilty Tidal Barrage Study 2000
Water Level Upstream of Barrage
Tide Level Downstream of Barrage
Barrage State
Peak Tidal Level = 4.1m OD Poolbeg
River Flow =0.5 m3/sec
Contaminant = 1fc in 1m3/s
Spring Tide - Low Flow
Predicted Coliform Concentration
from a Unit Source
Normalised Concentration, Unit Source
Upstream of Barrage
1.00
0.80
0.60
0.40
0.20
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
Barrage Valves Open
Barrage Valves Closed
Barrage Valves Closed
Water levels indicated in metres OD
Model Simulation in Tidal Cycles

Figure 4.7

Clonakilty Tidal Barrage Study 2000

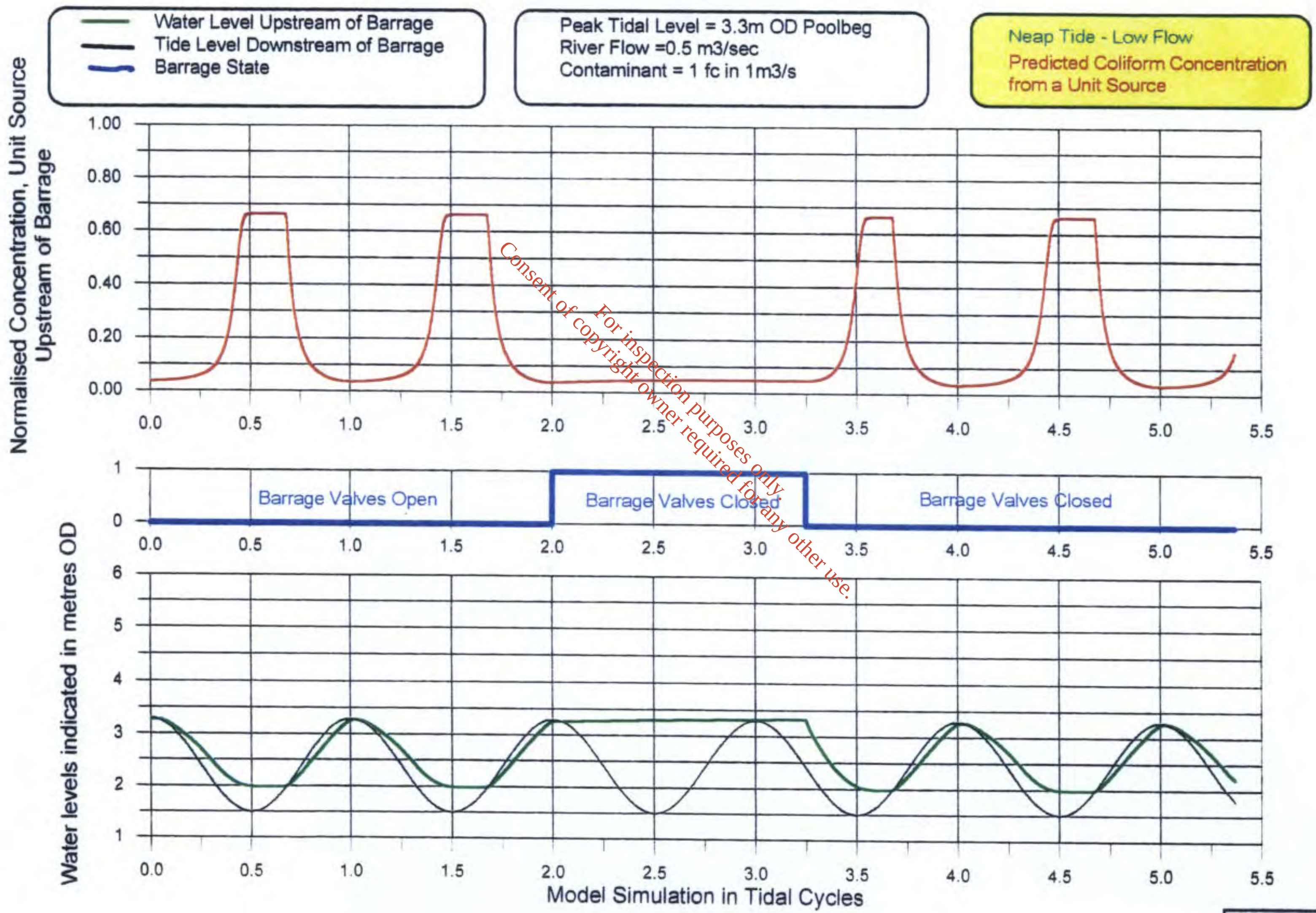


Figure 4.8

Clonakilty Tidal Barrage Study 2000

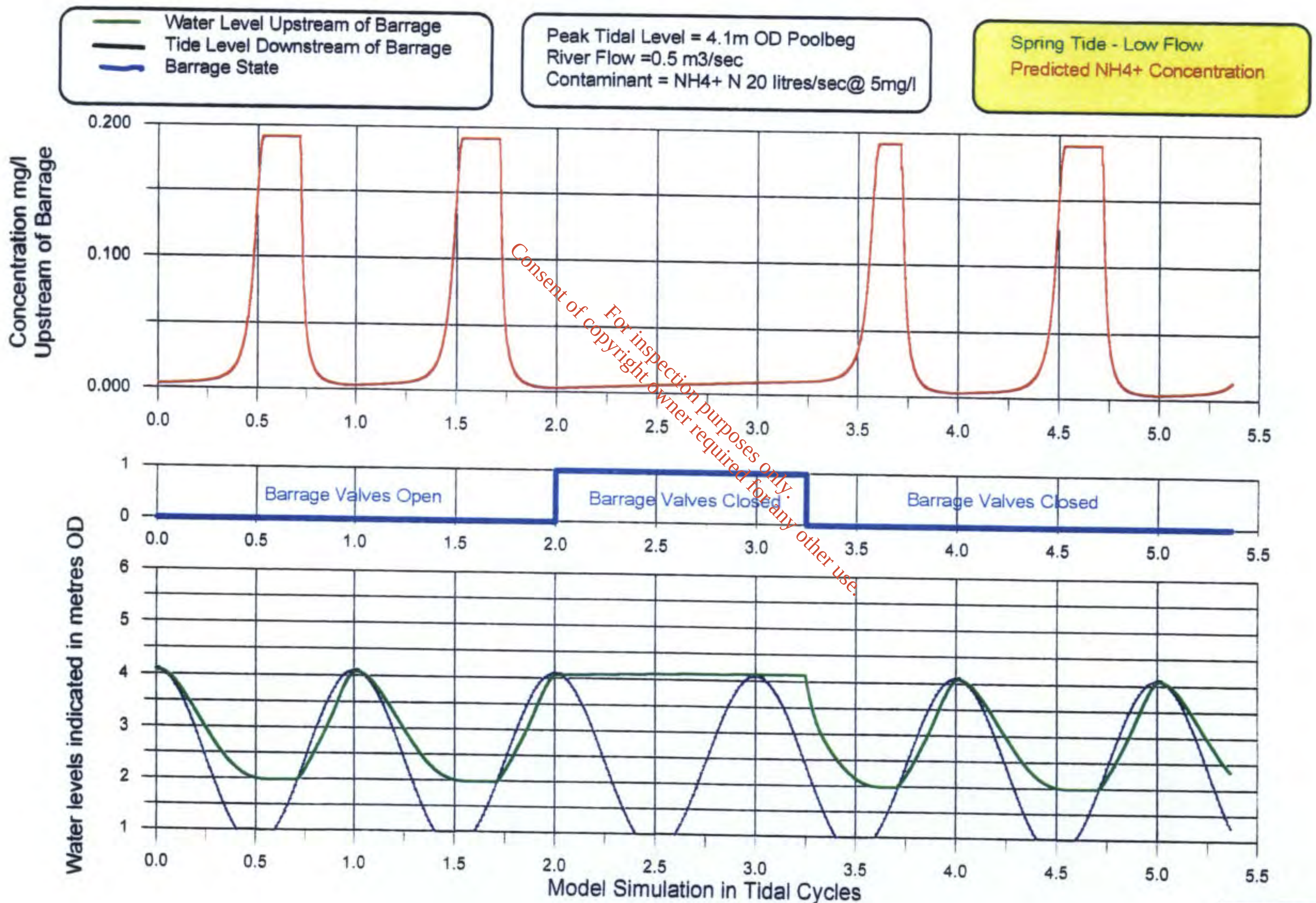
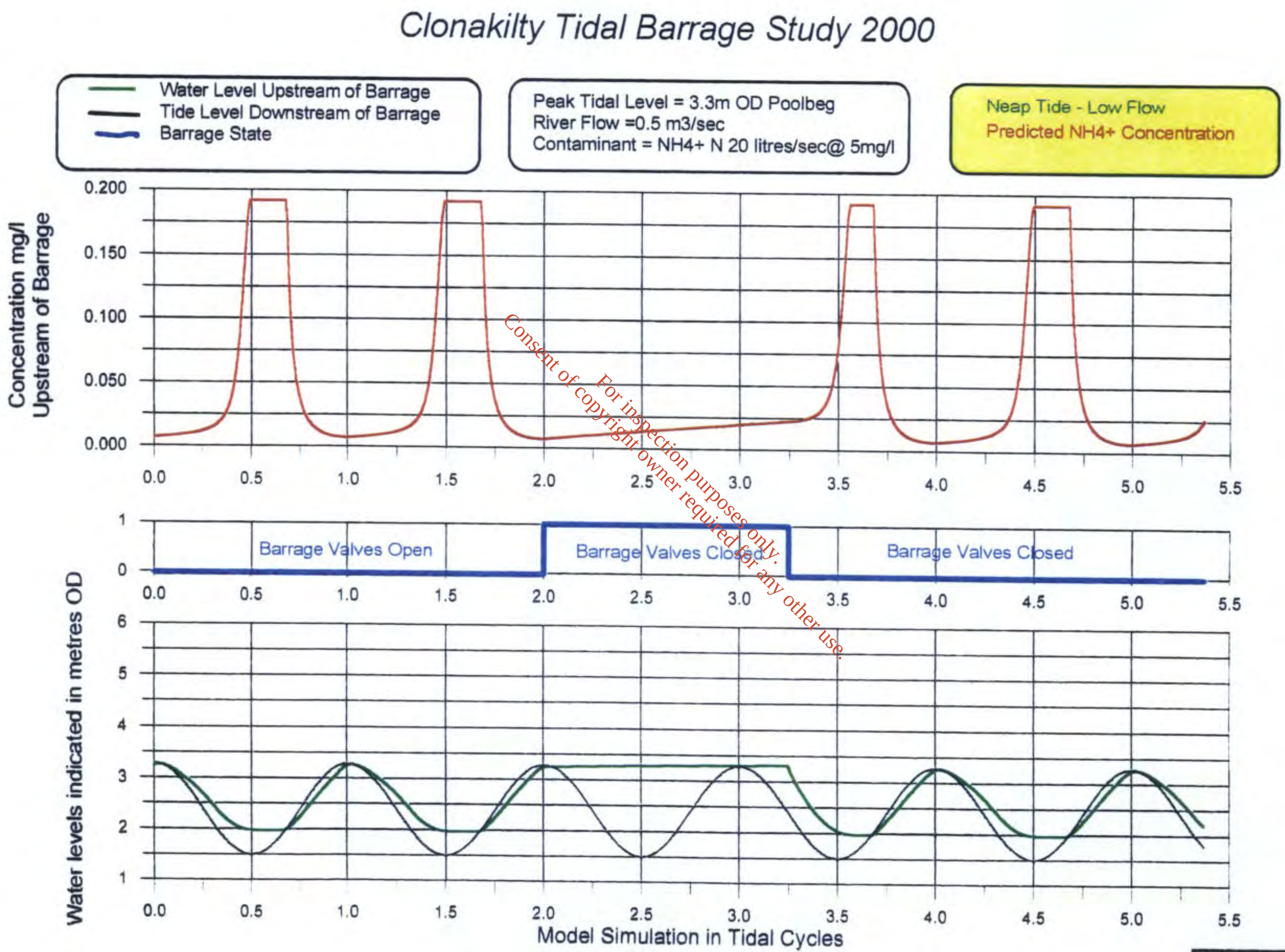


Figure 4.9



Clonakilty Tidal Barrage Study 2000
Water Level Upstream of Barrage
Tide Level Downstream of Barrage
Barrage State
Peak Tidal Level = 3.3m OD Poolbeg
River Flow =0.5 m3/sec
Contaminant = NH4+ N 20 litres/sec@ 5mg/l
Neap Tide - Low Flow
Predicted NH4+ Concentration
Concentration mg/l Upstream of Barrage
0.200
0.150
0.100
0.050
0.000
0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5
Barrage Valves Open
Barrage Valves Closed
Barrage Valves Closed
1
0
Water levels indicated in metres OD
6
5
4
3
2
1
Model Simulation in Tidal Cycles

Figure 4.10