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REPORT

NOBLE DENTON NEWCASTLE

NEW PONTOON AT GRAVESEND
WAVE ENVIRONMENT STUDY

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1 EXECUTIVE SUMMARY

The wave climate at Gravesend in the River Thames has been assessed using the Mike 21 spectral wave and hydrodynamic models, to support the design of a new jetty. Five scenarios were modelled to assess the effects on the wave climate of extreme wind speed, storm surge and also climate change. The following key results were extracted from the output of these scenarios:

INPUT CASES			OUTPUT			
Y.R.P	**Wind Sp (m/s)**	**Sea Level (m)**	**Y.R.P**	**Hs (m)**	**Hs(m)** (including climate change)	**Tp (s)**
0.5	14	9.2	0.5	0.17	0.18	1.85
1.0	15	9.4	1.0	0.23	0.24	1.85
50	20	10.6	50	0.38	0.42	2.67
100	21	11.3	100	0.40	0.44	2.60
200	21	12.8	200	0.43	0.47	2.58

Table 1-1 Summary of simulate cases and results

Y.R.P is the return period in years.

2 INTRODUCTION

2.1 BACKGROUND

This document presents the wave modelling study carried out as support for the Marine Works involved in the installation of a ferry pontoon facility on the south bank of the river Thames in Kent. The pontoon facility is to be connected to Gravesend Town Pier to provide access for the ferry that runs across the Thames to Tilbury.

The wave modelling study is required for the detailed design of the structure. The design life of the pontoon facility is 50 years, assuming suitable maintenance within that period.

Noble Denton Consultants Ltd (NDL) was contracted to carry out the wave modelling exercise required for the detailed design of the structure. In this study the wave climate near the proposed pontoon facility was examined in detail. Specific assessments included

1. The wave climate due to locally wind generated waves over the available fetch, for the 1/2, 1, 50, 100 and 200 year return period conditions.
2. The above cases to be run over the highest possible water level, which corresponds with the MHWS plus the surge corresponding with each return period.
3. All the cases to be run from the worst possible directions.
4. Bathymetry in the locality.

2.2 STUDY AREA

The ferry pontoon facility will be installed on the south bank of the river Thames in Kent, connected to Gravesend Town Pier.

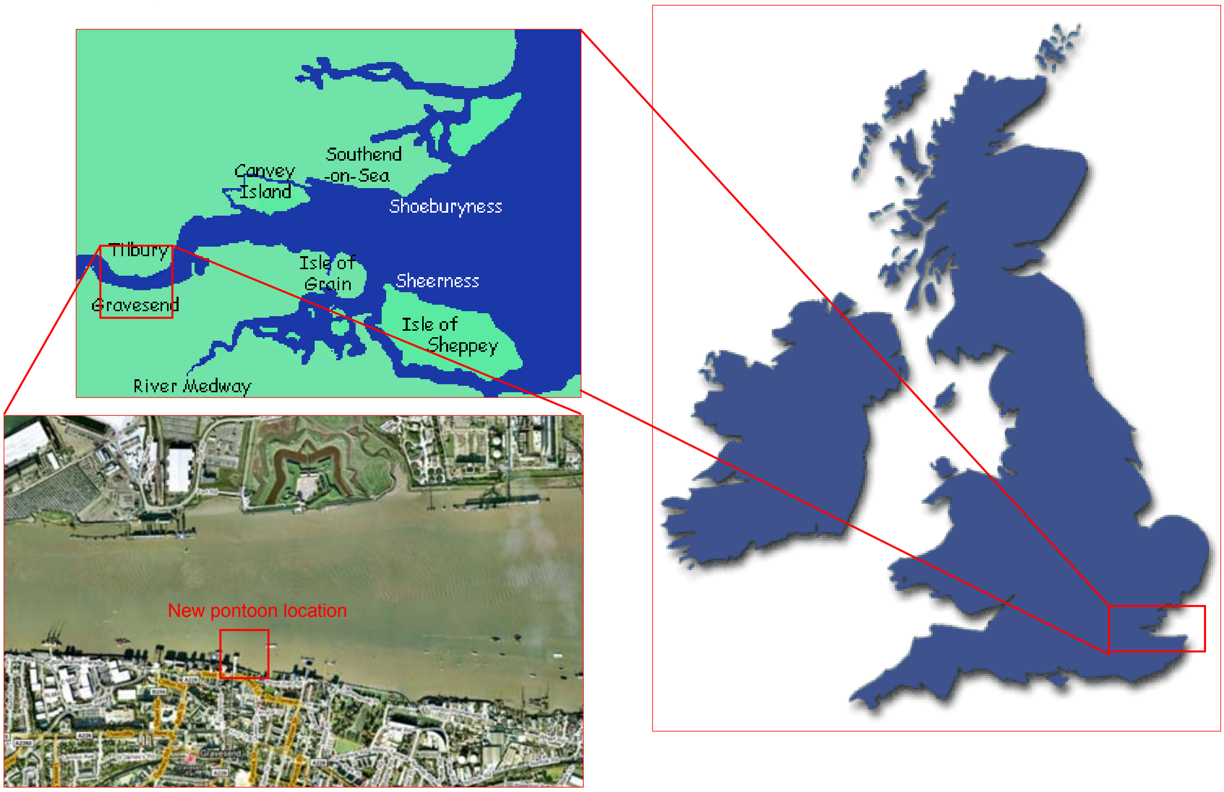

Figure 2.1: Location of the new pontoon facility

2.3 UNITS AND CONVENTIONS

The following conventions have been adopted in this study:

- Wave heights in metres; wave periods in seconds
- Wave directions are from which the waves are travelling, relative to North (0° or 360°)
- Mean wind speed at 10m above sea level speeds in m/s
- Wind directions are from which the wind is blowing relative to North (0° or 360°)
- Water levels and depths in metres above the LAT

2.4 GENERAL APPROACH

Due to the location of the study area inside the Thames Estuary, and approximately 15 miles away from its entrance (see Figure 2.1), Gravesend is protected from the offshore waves, which means that the wave climate in the area is due to the SEA or locally generated waves. These waves are generated by the local winds blowing over the River Thames, and their height will depend on the wind speed, the duration of the wind and the length over which the wind is blowing, or fetch.

In order to reproduce these conditions, a numerical modelling exercise was carried out using the Mike 21 software developed by the Danish Hydraulic Institute (DHI)

Numerical modelling provides an ideal solution, in which complex dynamic equations are solved by computer at every point on a pre-determined, high resolution grid; data can be extracted from model output databases at any number of locations and, provided the model has been carefully calibrated, will be of a higher level of precision than would be available from traditional methods. In outline, the general approach for this study involved:

- The setting up of a 2-dimensional numerical spectral wave transformation model, underpinned by a grid (mesh) across the area that is of higher resolution in the areas of specific interest.
- Calibration of the models against (as available) measured, water level and wave data in the area.
- Generation and transformation of the waves over the model area to the location of interest.

3 DATA SOURCES

3.1 WINDS

Getting a time series of measured data in the area long enough to perform a reliable extreme analysis was not possible, so using the 50 year return period value given by the British Standard [1] was considered the best option.

In order to calculate the directional coefficients of the wind at location, the Strongblow software [2] was used, and the 0.5, 1, 100 and 200 year extreme values derived by using the formulae included in The Designer's Guide to Wind Loading of Building Structures [3]:

$$ST = \left(\frac{5 + Ln(T) - Ln(-Ln(0.37))}{5 + Ln(50)} \right)^{1/2}$$; where T is the return period.

The following table shows the wind values obtained from these calculations:

YRP	DIRECTION							
	N	NE	E	SE	S	SW	W	NW
0.5	13	13	14	8	9	10	10	15
1	14	14	15	9	10	11	10	17
50	19	19	20	12	13	15	14	22
100	20	20	21	13	14	15	14	23
200	20	21	21	13	14	16	15	24

Table 3-1 Wind speed extreme values

3.2 TIDE

The east coast of England has a semi- diurnal tidal regime; the tidal range changes considerably in the North Sea due to the presence of three amphidromic points; in general, shallow coasts and the tunnel effect of narrow straits increase the tidal range.

The following table shows the tidal levels at Tilbury, just opposite to Gravesend, on the north bank of the Thames (levels are given with respect to the LAT)

	Level (m)
HAT	7.0
MHWS	6.4
MHWN	5.4
MSL	3.3
MLWN	1.4
MLWS	0.5

Table 3-2 Tidal Levels at Tilbury

3.3 SURGE

The storm surge is an offshore rise of water associated with a low pressure system. The storm surge is caused primarily by high winds pushing on the ocean's surface and causing the water to pile up higher than the ordinary sea level. The bathymetry has also an effect on the surge level, with higher surges in shallower waters. Storm surges in the North Sea are caused primarily by storms tracking to the north of the UK and have two components: external surges, which are changes in the sea surface height caused by winds associated with the storm driving the water down the east of the UK; internal surges, which are changes in the sea surface height generated by local winds (either associated with the storm or otherwise).

Due to the impossibility of finding surge measurements in the study area, data at Sheerness have been purchased from the BODC (British Oceanographic Data Centre).

These data consist of a time series with the monthly extreme surges measured at Sheerness between January 1990 and December 2008. An extreme analysis has been done with these data deriving the following extreme values:

0.5 yrp	1yrp	50yrp	100yrp	200yrp
2.0	2.2	2.9	3.1	3.3

Table 3-3 Extreme surges at Sheeness (m)

In order to calculate the surge value at location, a model of the Thames Estuary has been prepared and the extreme surges propagated through it from the open (West) boundary up to the end of the estuary.

The following figure shows the model bathymetry used to carry out the simulations, and the table below the values obtained at Gravesend.

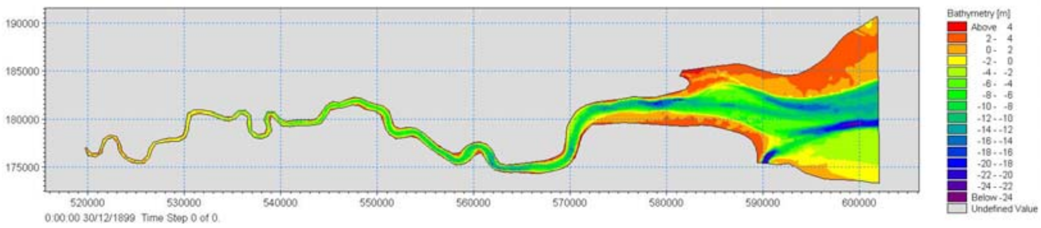

Figure 3.1: Bathymetry of the Thames Estuary

0.5 yrp	1yrp	50yrp	100yrp	200yrp
2.7	2.9	3.7	3.9	4.1

Table 3-4 Extreme surges at Gravesend (units)

As can be seen from the table above, the bathymetric effect on the surge elevation can be important, increasing the water level in Gravesend by up to 35% with respect to the level outside of the estuary.

3.4 CLIMATE CHANGE

As requested by the client, the total sea level rise due to increased storminess, estuarine flow and changes in mean sea level has been taken into account according to the methodology described in the "Planning Policy Statement 25: Development and Flood Risk". This document provides the following two tables to calculate the level rise:

Administrative Region	Net Sea Level Rise (mm/yr) Relative to 1990			
	1990 to 2025	2025 to 2055	2055 to 2085	2085 to 2115
East of England, East Midlands, London, SE England (south of Flamborough Head)	4.0	8.5	12.0	15.0
South West	3.5	8.0	11.5	14.5
NW England, NE England (north of Flamborough Head)	2.5	7.0	10.0	13.0

Table 3-5 Recommended contingency allowances for net sea level rise

Parameter	1990 to 2025	2025 to 2055	2055 to 2085	2085 to 2115
Peak rainfall intensity	+5%	+10%	+20%	+30%
Peak river flow	+10%	+20%		
Offshore wind speed	+5%		+10%	
Extreme wave height	+5%		+10%	

Table 3-6 Recommended national precautionary sensitivity ranges for peak rainfall intensities, peak river flows, offshore wind speeds and wave heights.

According to these recommendations, and considering that the new pontoon facility will be built in 2010, the following sea level rises will be considered:

0.5 yrp	1yrp	50yrp	100yrp	200yrp
0.08m	0.08m	0.46m	1.01m	2.26m

Table 3-7 Sea level rises due to Climate Change

Regarding the waves, the percentages recommended will be added to the significant wave heights obtained at location with the numerical model. The total values are shown in 5 Results .

4 NUMERICAL MODELLING

4.1 MIKE 21 BY DHI

The Mike 21 modelling suite produced by the Danish Hydraulics Institute is a state-of-the-art software package for simulating flows and waves in estuaries, coastal areas and seas. Noble Denton operates the software in-house and for this project developed a bespoke model solution to provide the required metocean information.

Included in the MIKE21 package are models to simulate the waves propagation (SW, PMS, Boussinesq, EMS and NSW), as well as models to simulate the change of level and currents induced by the tides (FM, HD), the advection- dispersion of substances in the water (AD), the sediment transport (ST), and the coastline evolution (LITLINE) among others. For this study two models have been used: the Spectral Wave model (SW) to simulate the generation, growth and propagation of the waves inside of the study area, and the Flow Model (FM) to calculate the surge elevation at the location.

MIKE21 SW [4] is a new generation of spectral wind-wave model based on unstructured meshes. The model simulates the growth, decay and transformation of wind generated waves and swell, in offshore and coastal areas. It includes physical phenomena such as wave growth by action of wind, non-linear wave-wave interaction, dissipation due to white-capping, dissipation due to bottom friction, dissipation due to depth- induced wave breaking, wave-current interaction, refraction and shoaling due to depth variations, diffraction and reflection.

The MIKE21 FM [5] modelling system has been developed for applications within oceanographic, coastal and estuarine environments. It is based on the numerical solution of the two-dimensional shallow water equations (the depth-integrated incompressible Reynolds averaged Navier-Stokes equations). Thus, the model consists of continuity, momentum, temperature, salinity and density equations.

In both models, the spatial discretization is performed using a cell-centred finite volume method.

4.2 MODEL SET UP

4.2.1 Bathymetry

The bathymetry for this study was extracted from two sources:

1. The C-Map Global Electronic Chart database CM-93/3, produced by C-Map Norway (www.c-map.no), using tools provided with the Mike 21 modelling software.

   C-Map specialises in the production of electronic charts for marine navigation and the CM-93/3 database is designed for use in shipborne Electronic Chart Display Information Systems (ECDIS). It therefore complies with content, format and quality Performance Standards stipulated by the International Maritime Organisation. The database includes charts produced by a number of international Hydrographic Offices and the marine application requires strict quality control of the data.

2. Hard copy chart supplied by Gifford, showing spot depths close to the interest location.

The model bathymetry was prepared as follows:

- Extract from the C-MAP the coastline and the bathymetry covering the model domain – the native resolution is approximately 5m.
- Add the pontoon facility, as determined from drawings supplied by the client.
- Digitalize hard copy of local bathymetry
- Create an appropriate flexible mesh, and interpolate the bathymetry on to it.

The resulting interpolated bathymetry is shown in Figure 4.1 and the new pontoon facility in Figure 4.2.

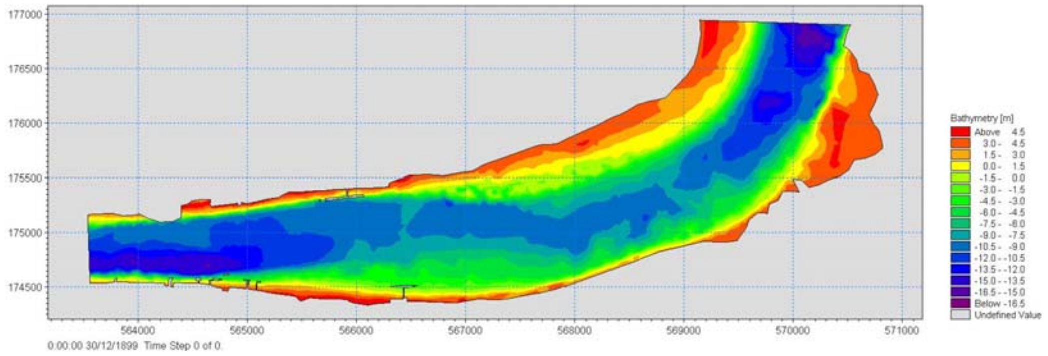

Figure 4.1 Bathymetry representing the study area

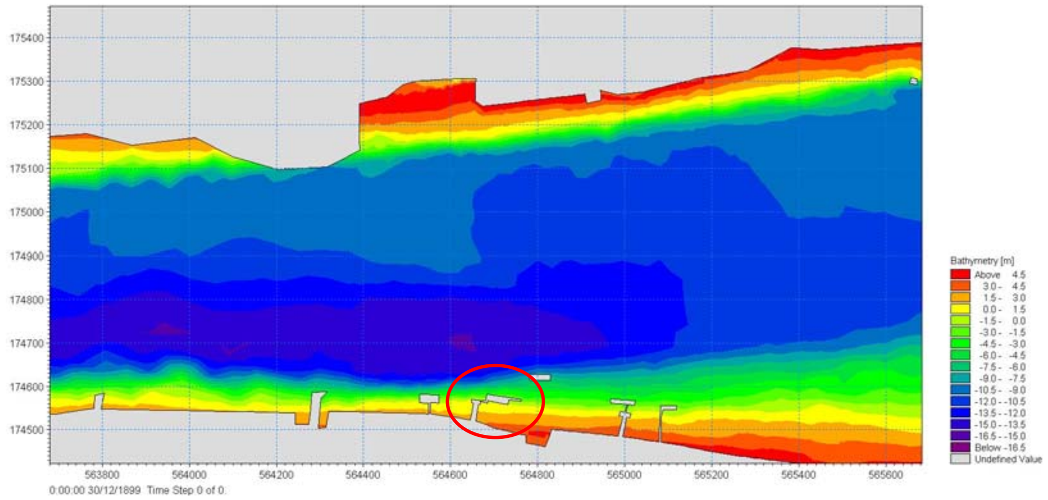

Figure 4.2 New Pontoon facility

4.2.2 Model flexible mesh

A key feature of the flexible mesh system is the facility to vary the mesh resolution according to depth or the need to simulate conditions more precisely in some areas than others. Defining the model computational mesh required consideration of the following:

- Adequate resolution of the bathymetry, flow, wind and wave fields. This is very important in the coastal areas, where the more complex bathymetry undoubtedly affects the physical conditions.
- The need for computational efficiency to achieve timely delivery of the study results.

- Model stability. The propagation speed across the model domain of information about water levels and wave groups, and hence the mesh resolution, is critical to model stability and to avoid “blow ups”.

Mesh development involved:

1. Fitting a coarse mesh over the entire model domain.
2. Definition of areas requiring finer resolution coverage.
3. Careful examination of the mesh configuration at the model open and land boundaries to ensure instabilities were avoided.
4. Checking model run times were acceptable.

The selected mesh configuration is shown in Figure 4.3. Note:

- The variable grid resolution over the domain provided by the flexible mesh system.
- The finest mesh adjacent to the pontoon is at a resolution of around 20m. The coarsest mesh has a resolution of around 70m.

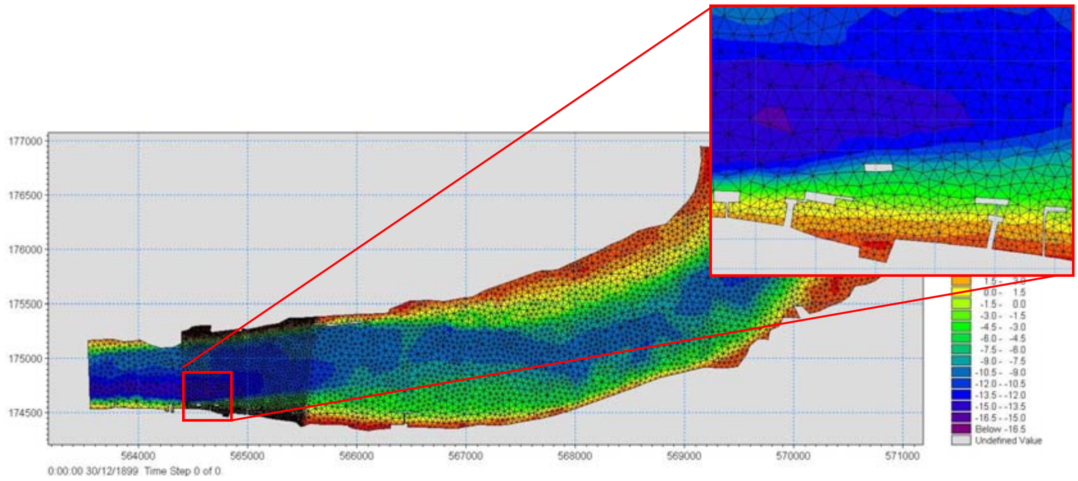

Figure 4.3 Flexible mesh

4.2.3 Boundary Conditions

4.2.3.1 Winds

In this case, as only the sea waves were needed, both boundaries were left as closed, and the wind input in the model as a constant wind blowing throughout the model area during the simulation period.

In order to find out which are the worst possible conditions at the pontoon location, three cases were tried with the numerical model:

- The longest fetch direction: 71°N; 3.3 nm approximately.
- The strongest wind direction: 290°N; 1.8 nm approximately.
- The north direction (which generates waves arriving from a perpendicular direction to the structure): 0°N; 0.4 nm approximately.

The waves obtained for each one of these situations, all with the 50 year return period wind, are shown bellow:

	Hs(m)
Longest fetch	0.38
Strongest wind	0.10
North	no discernible waves

Table 4-1 Significant wave heights at Gravesend due to different wind directions.

As shown in the table above, the biggest waves are generated when the wind blows from the longest possible fetch, which corresponds with an approximate direction of 71°N, so this is the direction chosen to simulate all the extremes.

The air-sea interaction has been set up as "uncoupled", which means that the momentum transfer from the wind to the waves solely depends on the wind speed, and consists on a sea state independent roughness description. The white capping (reduction of wave energy due to the wind and controlled primarily by the steepness of the waves) is also included.

4.2.3.2 Reflection

The SW model allows the definition of reflective boundaries inside the model area. Since the wall behind the pontoon structure is a vertical wall, the waves reflected over it could affect the structure, and to include this effect in the simulations, the coastal line behind the pontoon has been defined as reflective with a reflection coefficient of 0.9.

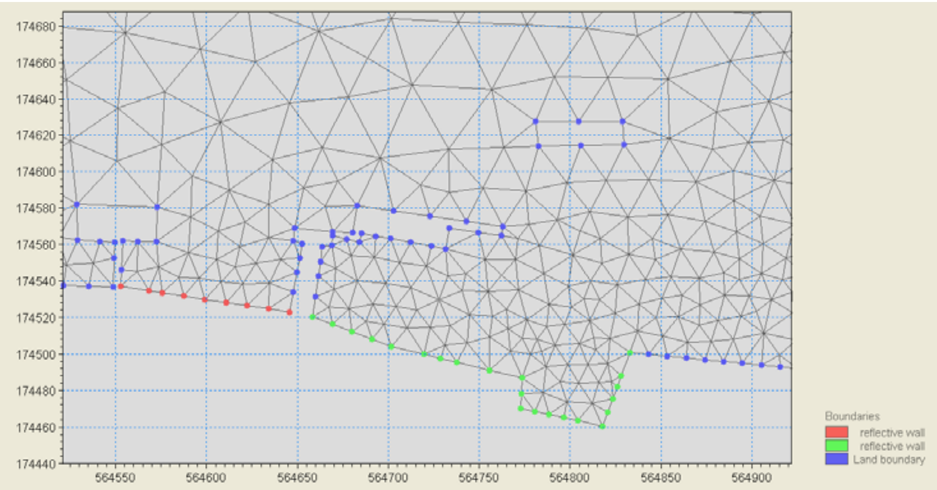

Figure 4.4 Reflective boundaries behind the pontoon

4.3 MODEL CALIBRATION

The Mike 21 model is extensively used and calibrated all over the world, but it has to be calibrated in order to assure that the set up used for each case is the most appropriate.

The Port Authority of London, the BODC (British Oceanographic Data Centre), the MetOffice, the CEFAS (Centre for Environment, Fisheries & Aquaculture Science) and the POL (Proudman Oceanographic Laboratory) were contacted regarding measured wave and surge data with which to calibrate the models, but lack of success meant that the models were run with the default settings recommended by DHI.

4.4 SIMULATED CASES

The following table summarises the cases simulated in this study:

Y.R.P	Wind Speed (m/s)	Sea Level (m) (over LAT)			
		MHWS	Surge	Climate change	Total
0.5	14	6.4	2.7	0.1	9.2
1.0	15	6.4	2.9	0.1	9.4
50	20	6.4	3.7	0.5	10.6
100	21	6.4	3.9	1.0	11.3
200	21	6.4	4.1	2.3	12.8

Table 4-2 Summary of simulated cases

5 RESULTS

The following figure shows the selected locations to extract the wave parameters, and the table below the results obtained from the simulations carried out.

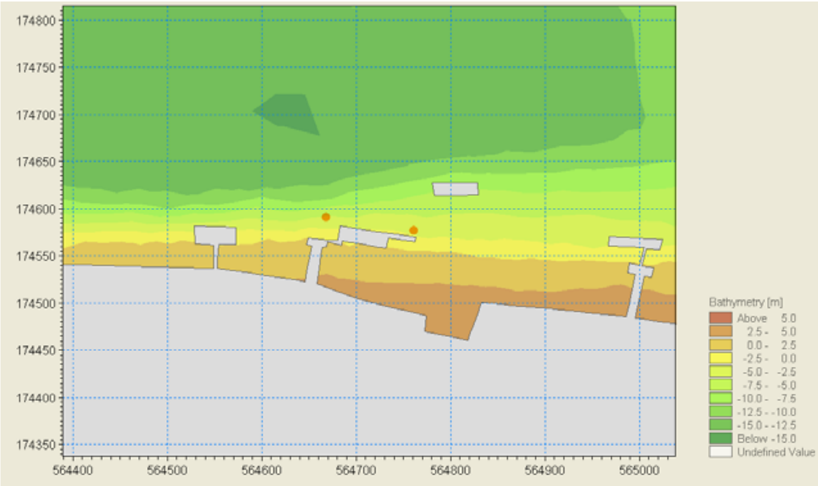

Figure 5.1 Location of the extracted results

	0.5 yrp	1yrp	50yrp	100yrp	200yrp
Hs (m) from modelling	0.17	0.23	0.38	0.40	0.43
Hs (m) including Climate Change	0.18	0.24	0.42	0.44	0.47

Table 5-1 Significant wave height in meters at Gravesend (m)

5.1 WAVE ENERGY SPECTRA

5.1.1 JONSWAP spectrum

The JONSWAP spectrum is normally acceptable for sheltered areas where seas are not fully developed and swell represents an insignificant part of the total wave energy. It is given by the expression:

$$S_{\eta\eta}(f) = \left(\frac{\beta g^2}{f^5}\right) \cdot \exp\left(-1.25\left(\frac{f_p}{f}\right)^4\right) \cdot \gamma^a$$ where

$$a = \exp\left(\frac{-(f/f_p - 1)^2}{2\sigma^2}\right)$$ and

β s related to fetch but in the absence of such information β = 0.0081

σ = 0.07 $f<f_p$ or σ = 0.09 for $f=>f_p$.

The variable γ has been shown to have a mean value of 3.3 with a standard deviation of 0.79.

5.1.2 Determination of peak enhancement factor

The following expressions have been provided by DnV for the calculation of peak enhancement factor:

$$\gamma = 5 \quad for \quad T_p \leq \sqrt{13 \cdot H_s}$$

$$\gamma = \exp\left(5.75 - 1.15 \times {T_p}/{\sqrt{H_s}}\right) \quad for \quad \sqrt{13 \cdot H_s} < T_p < \sqrt{25 \cdot H_s}$$

$$\gamma = 1 \quad for \quad T_p > \sqrt{25 \cdot H_s}$$

The peak enhancement factor has been calculated at Gravesend by using the results obtained from the numerical modelling. The results are shown in the table below:

Hs (m)	Tp (s)	γ
0.17	1.85	1.8
0.23	1.85	3.7
0.38	2.67	2.2
0.40	2.60	2.8
0.43	2.58	3.4

Table 5-2 Peak enhancement factor values

As expected the enhancement factor at site is around 3, which corresponds with areas where there is no swell and fetch limited seas.

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Dated: London, 10/09/09

REFERENCES

[1] British Standards Institution (BSI), 2002
Loading for Buildings – Part 2: Code of practice for Wind Loads.
BS 6399-2.

[2] Cook, N.J., 1985
BRE Program STRONGBLOW: users' manual.
Supplement 2 to: The Designers Guide to the Wind Loading of Building Structures.
UK Building Research Establishment. Butterworths.

[3] Cook, N.J., 1985
The Designers Guide to the Wind Loading of Building Structures.
UK Building Research Establishment. Butterworths

[4] DHI, 2005.
MIKE 21/3 Spectral Waves Model FM Module - Scientific Documentation.
DHI Group, Hørsholm.

[5] DHI, 2005.
MIKE 21/3 Flow Model FM Hydrodynamic and Transport Module - Scientific Documentation.
DHI Group, Hørsholm.