POTENTIAL EFFECTS OF OFFSHORE WIND DEVELOPMENTS ON COASTAL PROCESSES

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Contractor

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EXECUTIVE SUMMARY

Overall Aim

The overall aim of the present study has been to identify, review and assess the potential effects on coastal processes related to the development of offshore wind farms around the UK coast. Within the context of this study coastal processes are deemed to include the diffraction and focusing effects on waves and currents and their effect on long shore drift and erosion. From this work, the project outputs aim to provide generic guidance on these issues to stakeholders for use in the planning and consent stage prior to any development.

Background

The Government has made a commitment that by 2010 at least 10% of electricity demand in the UK will be generated from renewable sources. The fulfilment of this commitment will require greatly increased development of several renewable technologies, including offshore wind.

The present study is one in a series of special research topics, funded by the Department of Trade and Industry (DTI) within the New and Renewable Energy Programme, examining generic issues related to offshore wind developments.

The requirement for the project originates from the recommendations of an earlier study which provided a generic assessment of the environmental effects of offshore wind farms and served to establish the key environmental issues and potential environmental effects of large-scale offshore wind farms (ETSU, 2000). This earlier study identified areas where additional research was required, which included, amongst others, the need for a greater understanding on the potential effects on coastal processes. ETSU commissioned ABP Marine Environmental Research Ltd. (ABPmer) and Metoc Plc. to undertake this research and examine the potential effect on coastal processes under 2 scenarios; a 'reasonable worst case' and a more 'typical' installation.

Summary of work carried out

The scope of work carried for the study has included:

- Definition of main wind farm parameters for 'reasonable worst case' and 'typical' scenarios (based on the sites made available by Crown Estate under the first round of offshore wind developments for the UK), and involving:
  - early stakeholder discussions to identify the possible wind farm sites that are, or may be, under consideration
  - identification of coastal processes that may be effected

- review of existing and accessible scheme development studies and consideration of any relevant studies carried out elsewhere in Europe

- Assessment of the applicability of existing computer models, including:
  - review of applicable methods and models used in similar coastal impact studies
  - selection of appropriate model(s) to describe the coastal processes parameters from the 2 wind farm scenarios
  - assessment of the ability of the selected model approach to correctly reproduce wave scattering by the turbine base (ref. Appendix G3)

- Evaluation of the significance of the effects on coastal processes across the development site and over adjacent areas, with
  - use of an agreed modelling methodology to provide a description of coastal processes
  - definition of a baseline condition for each scenario (ie pre-development condition) and comparison to a prediction of coastal processes including the schemes (ie post-development)
  - assessment of the results to determine potential effects

- Production of an authoritative and independent report suitable for both a public and technical audience.

Summary of main results

The study has carefully considered the issues of coastal processes related to UK Interest Areas and compiled the emerging detail related to the first round of proposed offshore wind farm developments to define 'reasonable worst case' and 'typical' scenarios.

The study recognises that, for a successful wind farm development, there needs to be a good understanding of the local coastal environment in terms of:

- Designing a scheme that can cope with the environmental conditions (eg seabed, wind, wave and tidal regimes, etc) by establishing appropriate design parameters.
- Designing a scheme that the local environment can accommodate with minimum impact, by undertaking an Environmental Impact Assessment.
- Noting that the principal effects are likely to be associated with localised scour in the immediate vicinity of the structures and that scour around the cables could happen if care is not taken to secure adequate protection during and after laying.

Coastal processes play an important role in both of these issues, and for the allocated sites in the first round of UK offshore wind farm developments, it is possible to make the following generalisations:

- All sites are relatively close to the coast (with an average distance of around 7km, and all within 10km)
- They are all in comparatively shallow water (on average depths of 10m at low tide, and all <25m) - in several cases on sandbanks, and
- They are all purposely located with exposure to a good offshore wind resource and within proximity to a suitable grid connection point onshore

At these coastal locations, tides and tidal currents, waves, wave-driven currents, and wave-current interaction are the main coastal process issues and, in turn, will be the driving conditions for sediment movement. These generic properties have been used to configure a representation of a 'reasonable worst case' and 'typical' wind farm scenario. By establishing appropriate computer modelling methods, an evaluation of the significance of such offshore developments on coastal processes - both inside the wind farm and further afield - has been completed.

The results from generic tidal, wave and sediment modelling scenarios suggest that, at a regional level, there is unlikely to be a significant effect on coastal processes due to an offshore wind farm development. The impact of the 'reasonable worst case' is slightly more pronounced than the 'typical' case, but neither is at a level that should lead to any major concern. In addition, the changes predicted in each coastal process parameter are considered to be at the limit of accuracy of conventional monitoring equipment. As far as waves are concerned, the detection of such small changes by instrumental monitoring would, mainly for statistical reasons, present a very difficult challenge.

Notwithstanding these results, it is important to note that the present studies are based on an assessment of short-term impacts related to a set of idealised cases and supported only by theoretical considerations. As such, the work is only able to provide an indication of potential effects of offshore wind developments on coastal processes. Therefore, one of the key recommendations is, that once individual schemes proceed, results from associated monitoring programmes are examined to either further validate the conclusion of this report or to modify them.

Whilst the current research indicates that the impacts on the environment are likely to be negligible, the environmental impacts on the scheme, in terms of wave and current loading, are likely to be far more severe. In addition, the problems associated with locally generated scour (ie around the turbine structures and cables) will need to be dealt with adequately.

Conclusions

Europe is taking the lead in developing the technology to exploit the offshore wind resource as a viable form of renewable energy. Presently, there are 9 installed sites across Denmark, Sweden, Holland and the UK with a total generating capacity of around 90 MW. Early sites came into operation in 1991 and were based on small-scale demonstration projects to prove that the mature onshore technology could be successfully installed off the coast. Since then, schemes have grown in size and capacity and are now targeting more hostile coastal and offshore locations. Presently, the largest planned offshore wind farm project under construction is located off the west coast of Denmark at Horns Rev. This wind farm will be the first Danish project to be erected in the North Sea, with all previous schemes located in the more sheltered Baltic Sea.

The UK experience in offshore wind is still developing and needs to acknowledge that there are several important differences to the schemes in the Baltic. For instance, the design of the foundations at the Danish Middelgrunden site included provision for ice-breaking and ice-loading but was required to deal with only small wave and tidal influences. In comparison, UK coastal waters present more exposed wave conditions, relatively large tidal influences, and areas of high sediment mobility – significant challenges for wind farm design and operation.

The present study has carefully considered the potential effects on coastal processes caused by the installation of an offshore wind farm development and associated infrastructure, including cabling to shore. It has done this in the context of first-round sites around the UK coast announced by the Crown Estate on April 2001 and the likely wind farm dimensions associated with these developments. However, the general findings of the present study are considered to apply more widely.

The work has involved a brief review of existing wave, current and sediment models available for use in coastal impact studies, such as for marine aggregate extraction and engineering design. It is concluded that selected, established models can be adapted and applied to assess the interaction between an offshore wind farm and coastal processes. However, as yet these same models do not provide a direct means to predict scour development due to various simplifications in their numerical schemes, eg turbulence closure assumptions. Although, indirectly, these models can provide appropriate input parameters into a well-chosen empirical relationship to predict scour.

It is concluded from the range of tests performed that the changes in current, wave and sediment conditions brought about by the presence of the wind farm are unlikely to be significant in the far-field, with only very small influences determined in the near-field.

However, whilst the research has shown that the potential effects on coastal processes are likely to be negligible, the physical environmental impacts on the scheme, in terms of current and wave loading, are likely to be more severe.

In addition, problems associated with locally generated scour around the foundations of turbines and cables will need to be dealt with, probably involving appropriate protection measures.

Recommendations

As yet there are no large-scale offshore wind farms in similar coastal environments anywhere in the world, although some are planned for installation in 2002 and beyond. This means that there is, as yet, no project experience to draw on. The present study has therefore been based on hypothetical scenarios for a 'reasonable worst case' and a 'typical' offshore wind farm and used theoretical considerations to assess the potential effects these scenarios may have on coastal processes, primarily through the application of existing computer models. It is recommended that, when schemes are built and monitoring commences the resulting information is reviewed in the context of the present study to validate the conclusions of these investigations.

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ABBREVIATIONS

ABP	Associated British Ports
BGS	British Geological Survey
BWEA	British Wind Energy Association
CADDET	Centre for the Analysis and Dissemination of Demonstrated Energy Technologies
CCW	Countryside Council for Wales
CD	Chart Datum
CEFAS	The Centre for Environment, Fisheries and Aquaculture Science
CIS	Coastal Impact Study
CPA	Coast Protection Act, Section 34, 1949
DEFRA	Department for Environment, Food and Rural Affairs
DHI	Danish Hydraulic Institute
DTLR	Department for Transport, Local Government and the Regions
DTI	Department of Trade & Industry
EIA	Environmental Impact Assessment
ES	Environmental Statement
ETSU	Energy Technology Support Unit
FEPA	Food and Environment Protection Act 1985
MSL	mean sea level
MW	megawatt
NAW	National Assembly for Wales
NESS	North European Storm Study
NFFO	Non-Fossil Fuel Obligation
ODN	Ordnance Datum Newlyn

ORCU	Offshore Renewables Consents Unit
OWEN	Offshore Wind Energy Network
SMP	Shoreline Management Plan
TMA	Texel-Marsden-Arsloe
UK	United Kingdom

1 INTRODUCTION

1.1 Background

The UK government is working towards a target of renewable energy providing 10% of its electricity supplies as soon as possible, with the hope of achieving this target by 2010 (DTI, 1999). The fulfilment of this commitment will require greatly increased development of several renewable technologies, including offshore wind. The UK is well positioned to capitalise on the benefits of wind power having the best offshore wind resource in Europe, with an estimate of 24% of the practically exploitable offshore resource (CADDET, 2000).

As a contribution to this renewable energy target it is anticipated that a number of offshore wind farms will be developed at several areas around the UK coast. The Department of Trade and Industry (DTI) is the lead government department for ensuring that an efficient consenting process is in place to facilitate the progress of suitable offshore projects in England and Wales. A Guidance Note (DTI, 2002) describes the full consenting process, the roles and responsibilities of the consenting authorities, and the requirements of an Environmental Impact Assessment, including the Scoping Report and Environmental Statement (ES). It will be the purpose of the ES to provide a suitable review of the whole development across a range of environmental issues, including coastal processes. The latter issue will be of primary interest to DEFRA in their capacity as the licensing authority for the Food and Environment Protection Act 1985 Part II (FEPA) to ensure that fisheries and the marine environment are adequately safeguarded.

An earlier study has provided a generic assessment of the environmental effects of offshore wind farms (ETSU, 2000). It served to establish the key environmental issues and potential environmental effects of large-scale offshore wind farms. The study also identified areas where additional research was required, which included, amongst others, the need for a greater understanding of the potential effects on coastal processes.

In December 2000, ETSU commissioned ABP Marine Environmental Research Ltd and Metoc to undertake this research and examine the potential effects of offshore wind developments on coastal processes.

1.2 Aims and Objectives

The overall aim of the present study is to identify and review the potential effects on coastal processes, both positive and negative, caused by offshore wind farm developments.

In the context of the present study an offshore development is taken to be the combination of devices deployed in the coastal environment that comprise the wind farm, including the array of turbines and their associated infrastructure both within the farm and to shore. The study aim is to provide an assessment of the potential impact such a development may make on the offshore and coastal environment. In addressing these issues it is important to recognise that the potential effects may be both positive and negative and may occur both locally within the wind development area and remotely as a consequence of the development. The study context and scope are shown schematically in Figure 1.1.

The study hypothesis is based on the premise that if a ‘reasonable worst case’ scenario can be proved to have no detrimental impact on the coastal environment, then by inference any lesser ‘typical’ scenario will also be acceptable. This has required the definition of the ‘reasonable worst case’ and ‘typical’ scenarios by consideration of the types of coastal environments associated with potential offshore wind farm sites around the UK. However, each development must be considered and assessed separately, in order to identify and gauge the significance of driving physical processes in each case.

The study has focused on the various locations and typical wind farm parameters that are expected in the first round of proposed developments that were announced by Crown Estate in spring 2001. However, the general findings of the project are considered to apply more widely.

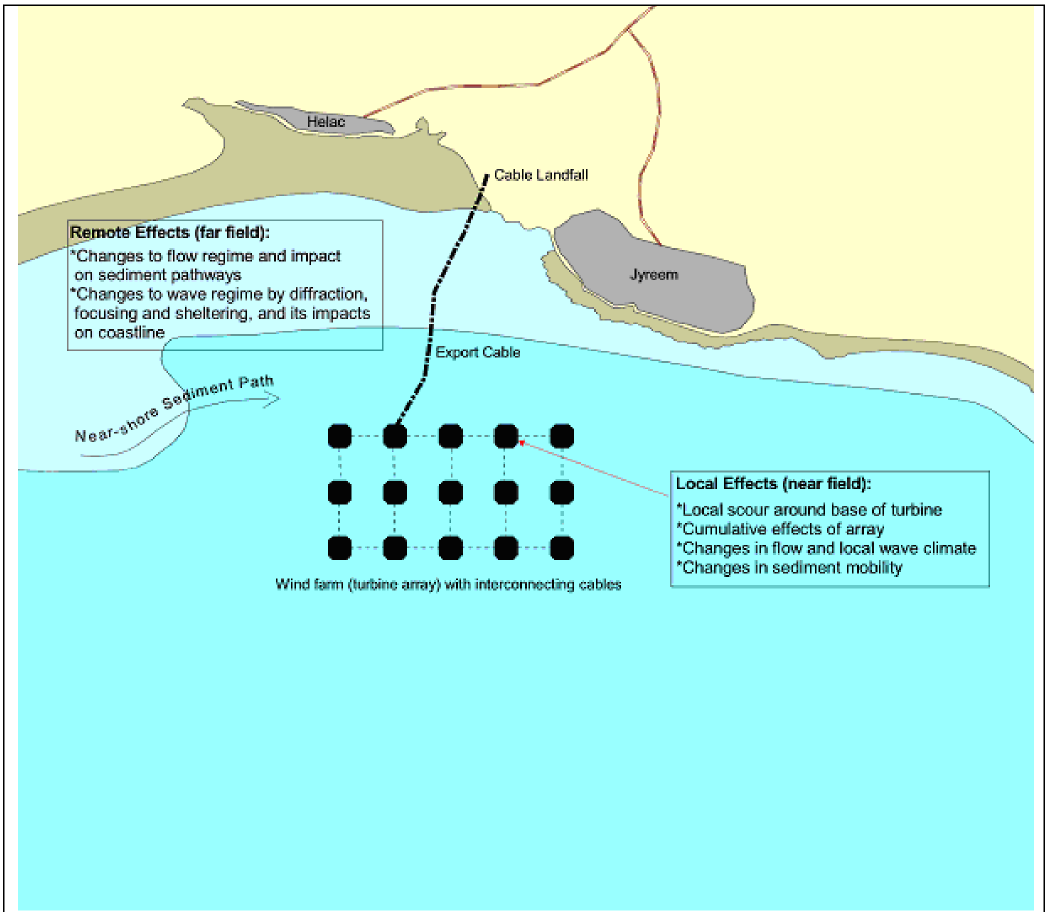
Individual objectives specified for present study included:

- Definition of wind farm parameters for ‘typical’ and ‘reasonable worst case’ scenarios.

- An assessment of the applicability of existing computer models to address the specific issues relating to offshore wind developments.

- Interpretation of output from suitable model(s) to determine the significance of the effects on coastal processes.

- Provision of an objective report of the methods employed and the results obtained, suitable for both a public and technical audience.



Project Aim: to evaluate potential effects of offshore wind developments on coastal processes

Approach:
1. Definition of wind farm parameters for 'typical' and 'reasonable worst case' scenarios.
2. Assess applicability of existing computer models
3. Interpret model output and determine significance of effects
4. Produce an objective report suitable for stakeholders

Figure 1.1 Schematic of study context and scope

2 DEFINITION OF STUDY PARAMETERS

2.1 Potential Offshore Wind Development Areas

The project has required identification of the potential offshore wind development areas for the UK, together with details of associated wind farm infrastructure. The phasing of the project has benefited from emerging details associated with the first round of proposed offshore wind farms announced by Crown Estate in spring 2001. In addition, the project team has carried out preliminary consultation with selected stakeholders. Details of the organisations contacted are provided in Appendix A.

2.1.a First UK Offshore Wind Farm

The first offshore wind farm to be developed in UK coastal waters is located around 1km off Blyth Harbour, North East England. This site is an extension to the existing waterfront development, which already includes nine turbines positioned along East Pier. The new development is generally referred to as Blyth Offshore and consists of two turbines built on the rocky shoal, North Spit, using drilled and grouted mono-pile foundations. The site is the first offshore wind farm to be built in the North Sea. The development was permitted under NFFO4 (Round Four of the Non-Fossil Fuel Obligation) and served as a demonstration project for the UK (ie for the technology, environmental issues, safety, navigation and consent process, etc), eventually coming into operation in December 2000.

Subsequently, a streamlined consenting process is being set up, with the intention for applications to be directed to a single government department, the ORCU at DTI (DTI, 2002). This unit will manage the overall consents process and act as a central co-ordinator with other government departments (eg DEFRA and DTLR) and relevant agencies (eg English Nature and CCW).

2.1.b Early Investigations

Gunfleet Sands, in the Thames Estuary, and Scroby Sands, off Great Yarmouth, have long been associated with an interest to develop offshore wind farms. Gunfleet Sands was approved under NFFO4, but Scroby Sands failed under NFFO5. Offshore anemometry has been undertaken at both sites, and in the case of Scroby Sands since around 1995. A previous study has made some initial considerations on wave climate and scour issues for the Scroby Sands wind farm (Halcrow, 1996), this report is briefly reviewed in Appendix B.

In January 1999, Crown Estate issued agreement for the installation of other offshore anemometry equipment at five additional sites around the UK coast for the purpose of monitoring local winds. The agreements conferred no rights for future development. The following sites were permitted:

- Robin Rigg, Solway Firth

- North Hoyle, North Wales
- Scarweather Sands, South Wales
- Kentish Flats, Thames Estuary
- Off Ingoldmells Point, Lincolnshire

2.1.c First Round of UK Applications

In December 2000, Crown Estate announced that sites for the development of offshore wind farms on the UK sea bed would be made available under lease arrangements for the first time in spring 2001. The criteria for sites in the first round permit a development of up to 30 turbines with a total minimum output capacity of 20MW, and within a maximum area of $10km^2$. In the first instance the period of the lease is 22 years, including construction, operation and decommissioning phases. In addition, Crown Estate stipulated a minimum distance of 10km between adjacent offshore wind farms. Furthermore, these sites all fall within the 12 nautical mile limit of UK territorial waters where Crown Estate act as the landowner.

For the first round applications, the primary UK Interest Areas can be generalised into the following areas:

- North East England
- East Coast (Humber Estuary to mid East Anglia)
- Thames Estuary
- Bristol Channel
- Liverpool Bay
- Solway Firth

Figure 2.1 provides a representation of these generalised areas which are based on a preliminary assessment of a range of factors including a suitable wind resource, environmental sensitivities and constraints, sea bed conditions, along with an accessible connection to the National Grid.

2.1.d Pre-qualification Results from First Round

On 5 April 2001, Crown Estate announced the names of 18 developers who each successfully pre-qualified to obtain a lease for an area of the sea bed for development of an offshore wind farm. Under the terms of the agreement for lease, successful companies must gain all necessary statutory consents within a period of three years from 31 July 2001.

Table 2.1 provides a summary of these sites and the developers who have successfully pre-qualified. Figure 2.1 indicates the approximate location for each of these sites. (*Note*: that the exact location and distance to shore are subject to revision, depending on sea bed conditions, environmental constraints, etc.).

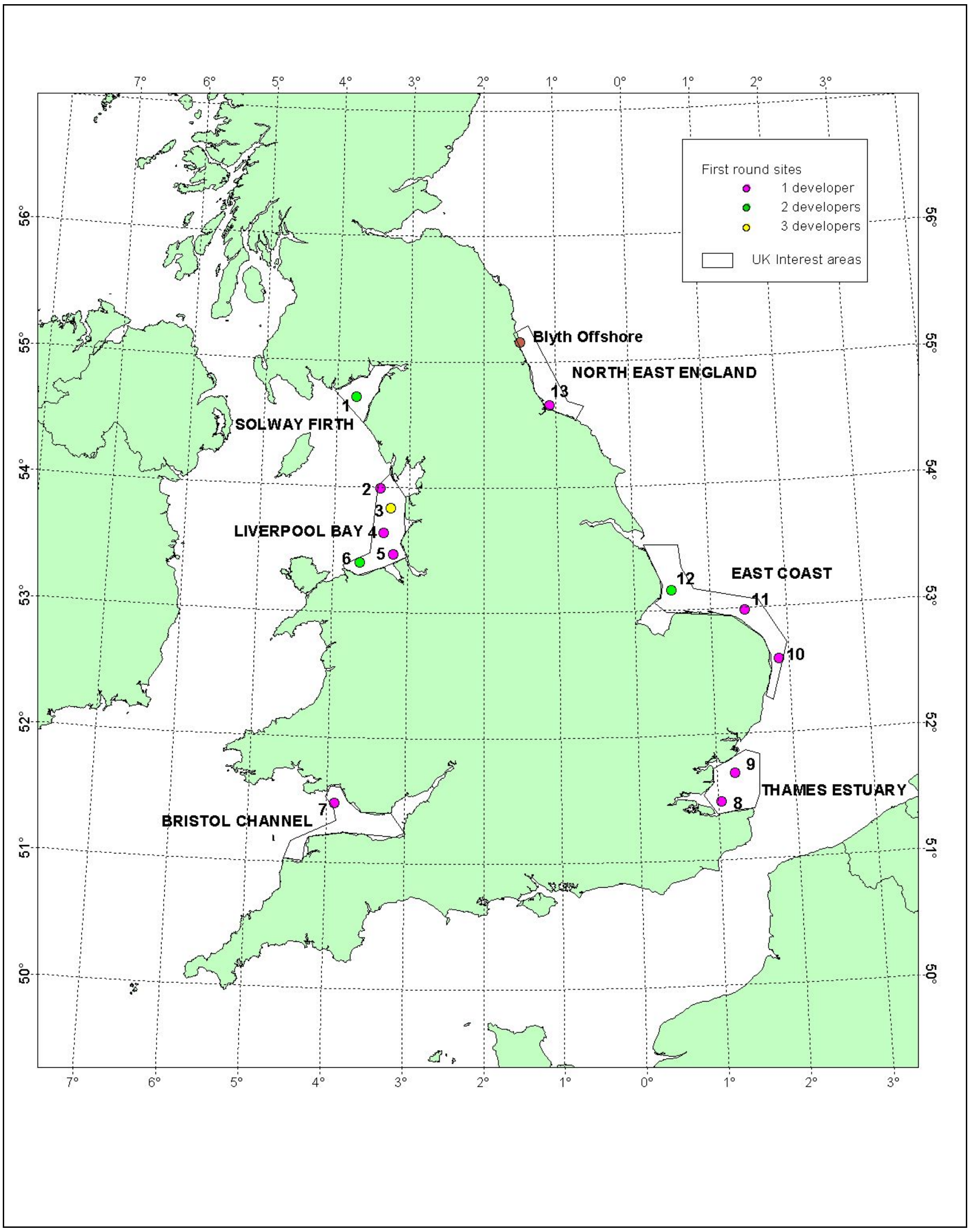


Figure 2.1 Location of UK Interest Areas and sites included in the first round of offshore wind developments.

(adapted from: http://www.crownestate.co.uk/estates/marine/wind farms/wfmap.shtml)

Table 2.1 Pre-qualification sites in the First Round

Map Ref.	Site Name	Developer	Location	Approx. Distance to Shore (km)
1	Solway Firth	a. Solway Offshore Ltd b. Offshore Resources Ltd	Off Maryport (England) Off Rock Cliffe (Scotland)	9.5 8.5
2	Barrow	Warwick Energy Ltd	Off Walney Island	10.0
3	Shell Flat	a. Shell WindEnergy Aegir Ltd b. Elsam A/S c. CeltPower Ltd	Off Cleveleys " "	7.0 7.0 7.0
4	Southport	EnergieKontor UK Offshore Ltd	Off Birkdale	10.0
5	Burbo	SeaScape Energy Ltd	Off Crosby	5.2
6	North Hoyle Rhyl Flats	a. NWP Offshore Ltd b. Celtic Offshore Wind Ltd (COWL)	Off Prestatyn Off Abergele, North Wales	6.0 8.0
7	Scarweather Sands	United Utilities Green Energy	Off Porthcawl	9.5
8	Kentish Flats	NEG Micon	Off Whitstable/Herne Bay	8.0
9	Gunfleet Sands	GE Wind Energy	SE off Clacton-on-Sea	7.0
10	Scroby Sands	Powergen Renewables Offshore Wind Ltd	Off Caister, (Middle Scroby Sands)	2.3
11	Cromer	Beaufort Consortium	Off Mundesley (Foulness)	6.5
12	Lynn Inner Dowsing	a. AMEC Offshore Wind Power Ltd b. Offshore Wind Power Ltd	Off Skegness Off Ingoldmells	5.2 5.2
13	Teeside	Northern Offshore Wind Ltd	Off NE Teesmouth & Redcar	1.5

Of particular note are sites at Map Ref. 1, 6 and 12 where applications have been granted for two developers in the same general area and Map Ref. 3 where applications have been granted for three developers in close proximity to each other. This raises the issue of possible cumulative impact between adjacent sites. It is understood that, in the case of these multiple sites, there may be some relaxation from the original requirement set by Crown Estate for a minimum of 10km separation. This is because certain impacts (eg visual or navigation) may be reduced overall if the individual wind farms are closer together.

The take-up within the generalised UK Interest Areas for the First Round is:

UK Interest Area	Number of Sites
North East England	1
East Coast (Humber to Mid-East Anglia)	4
Thames Estuary	2
Bristol Channel	1
Liverpool Bay	8
Solway Firth	2

2.1.e Beyond the First Round

It is important to note that the information collated herein is related to the first round of applications and sites announced by Crown Estate. At this stage, it is not meaningful to predict details of possible future developments. For example, in future rounds some developers may wish to apply for extensions to sites over and above their initial allocation, with new sites alongside existing ones. Future schemes may also consider sites at greater distances from the coast where the wind resource is higher and where some of the impacts (eg visual effects) may be reduced. A limit to offshore distance may be the cost of construction in depths greater than 30m, at least in the foreseeable future. In addition, grid connection will remain as an important logistical consideration.

2.1.f European Situation

Europe is presently leading the way in developing offshore wind farms with significant activity now occurring in Denmark, Sweden, Holland, Belgium, Germany, Ireland as well as the United Kingdom.

The interest in the offshore wind resource has emerged with the successful development of the technology for land based sites, together with recognition that good land sites will rapidly become exhausted.

The initial wind farms are generally regarded as research and demonstration projects for the technology. They tend to be sited in the Baltic Sea off the coast of Sweden and Denmark, and in Holland the first sites are both in a freshwater inland sea.

For all these areas the tidal influence is relatively insignificant (*Note*: the tidal range in the Baltic Sea is only a few centimetres at most) and the fetch lengths are restricted to relatively small distances (*Note*: thus reducing the capacity for generating large wind-waves). Hence, coastal process conditions related to these early developments are not directly comparable to conditions expected in UK coastal waters.

The success of these demonstration projects has allowed serious consideration of much larger wind farm parks, at greater distances off the coast, in deeper water and in more exposed conditions and where coastal processes may become more of an issue. There are presently nine active offshore wind farms in European waters with a combined installed capacity of around 90MW.

Horns Rev is one of the latest schemes being considered and will be the first Danish offshore wind farm planned for development in the North Sea. This case is perhaps the closest example yet to the sort of coastal process conditions that are anticipated at UK sites.

Appendix C provides additional details on these European developments.

2.2 Offshore Wind Farm Parameters

In the context of the present study wind farm parameters are used to define and describe the various components of an offshore development that may interfere with coastal processes. The major components are:

- Turbine foundations
- Array spacing
- Sea bed cable connections

2.2.a Turbine Foundations

The support structure for turbines forms a vital component of the offshore wind development. The design considerations are driven by the overriding requirement to avoid resonance with the periodic rotor force, and the added complication of combined hydrodynamic and aerodynamic loads. A variety of foundation types have been used for existing offshore wind farms, but often in less exposed conditions than UK coastal waters, for example the Baltic Sea. These include tripod, mono-pile and gravity-base concepts (Figure 2.2). Alternative concepts include caisson and suction bucket foundations, and for the future floating support systems are also being considered to avoid depth limitations.

In most cases, developers will consider the outcome of detailed geophysical and geotechnical site investigations before making a decision on the most suitable type of turbine foundations. Final design will also depend on water depths, environmental loading, construction costs, etc.

- Mono-piles

The present view is that the majority of first round sites will opt for mono-pile foundations, but each case is still subject to further investigations. Mono-piles are typically suited to shallow water (with maximum installation depths of about 25m) and mobile sea beds with strong sub-soil properties. Depending on the local sea bed conditions it may be necessary to either drill and grout or drive the mono-piles into the sea bed.

- Tripod

Tripod support structures have their origin in the oil and gas industry. They usually consist of a central column with three supporting piles linked by a steel frame which spreads the load of the turbine. Tripod concepts are more complicated to fabricate but provide greater structural stiffness which may result in less deformation of the tower under extreme loading conditions. They may also require shorter, smaller piles than the mono-pile solution. Tripod foundations are generally suitable in areas of deeper water and weak soil.

- Gravity

Gravity-based or caisson structures have been used successfully in several projects sited in the Baltic Sea where conditions are relatively benign. These foundations have been made of concrete or steel filled with ballast. A typical foundation for a 1.5MW turbine will need to weigh about 1,500 tonnes. Gravity foundations are typically suited to shallow water sites (normally less than 10m) where there may be no solid base (eg sandbanks). They are unlikely to prove a cost-effective solution in situations where the sea bed requires a lot of preparation to create a suitable surface upon which the structure can be placed. For some of the sites under consideration in the first round, the prefabricated foundation option may become too massive for transport on a large scale, with further constraints during installation if local currents are strong.

The typical diameter for these foundation types are listed below (OWEN, 2000).

Foundation type	Base Diameter (m)
Mono-pile	3 to 3.5
Tripod	10 to 12
Gravity base	12 to 15
Blyth Offshore (drilled and grouted mono-pile)	3.5

Presently the largest proven turbines are 2MW units, with approximately 3.5m diameter support bases. The expectation is that, by the time the first round wind farms are constructed, proven technology will be around 3 to 3.5MW, with an increased mono-pile base diameter of around 5m and gravity base diameter of around 15m. These values have been adopted in the present study.

<u>2.2.b Array Spacing</u>

In most cases, decisions on specific arrangements of the turbine arrays are yet to be made by developers. The final choice in array spacing will depend greatly on maximising energy yield from the wind farm, with slight adjustment based on local variation in sea bed conditions, depth and the need to accommodate a maximum number of 30 turbines within an area of $10km^2$. Separation between adjacent turbines is normally based on a multiple of the rotor blade diameter, D, with a typical spacing of 5 to 9D (CADDET 2000). This requirement ensures that fatigue loading from wind turbulence generated by the upstream turbines is minimised. Although array spacing will lead to greater efficiency at greater spacing, the increased cable lengths and construction costs will become an issue.

Typical Turbine Spacing	**(m)**
Across prevailing wind direction	
Minimum	300
Maximum	400
Along prevailing wind direction	
Minimum	700
Maximum	850
Blyth Offshore	200

Note: Blyth Offshore uses a blade diameter of 66m to power two 2MW Vestas turbines. The turbine spacing at around 3D is of less importance here since only two turbines make up the present wind farm, with upstream turbulence consequently limited to winds from the two directions when the turbines are aligned.

2.2.c Sea bed Cable Connections

The sea bed cable infrastructure of an offshore development consists of interconnecting cables between the turbines within the farm (perhaps running to a central converter sub-station) and one or more main high voltage export cable to the shore.

The developers that have been consulted have not yet finalised their designs relating to cable route, cable specification, number of cables to shore, depth and method of installation and burial, etc.

There are a variety of installation methods available depending on the nature of the sea bed substrate, the ecological sensitivity of the area and whether other existing cables or pipelines need to be crossed. Burial techniques include jetting/ploughing the cables into position or trenching prior to cable laying. The final installation method needs to ensure that the cable is placed in or on the sea bed in a way that minimises the possible risk of exposure from erosion and/or damage from trawling.

The main export cable, typically around 0.2 to 0.25m in diameter, will almost certainly need to be buried to avoid interference from fishing activity, shipping anchors, and possible exposure due to erosion. The interconnecting cables inside the wind farm are also likely to require burial. This will depend on the site, the water depth, and other risk factors. For example, if the site is deep enough to allow fishing boats with trawling gear, then burial is likely to be needed. However, for some sites, for example on top of a shallow sandbank, where trawling is restricted, then the developer may consider the option of laying the inter-connecting cables on the sea bed and letting it bury itself over time.

The cable route to shore will be determined on the basis of many factors. These include the chosen electrical grid connection point on shore, a suitable landfall site, the sea bed conditions, the presence of obstacles such as shipwrecks and rock outcrops, environmentally sensitive/protected areas, and other uses of the sea bed (eg oil and gas, and dredging).

Section 2.3.i briefly addresses potential effects on coastal processes of installing devices on the sea bed. In a site-specific context, these issues will need to be evaluated as part of the Environmental Impact Assessment and cable engineering design.

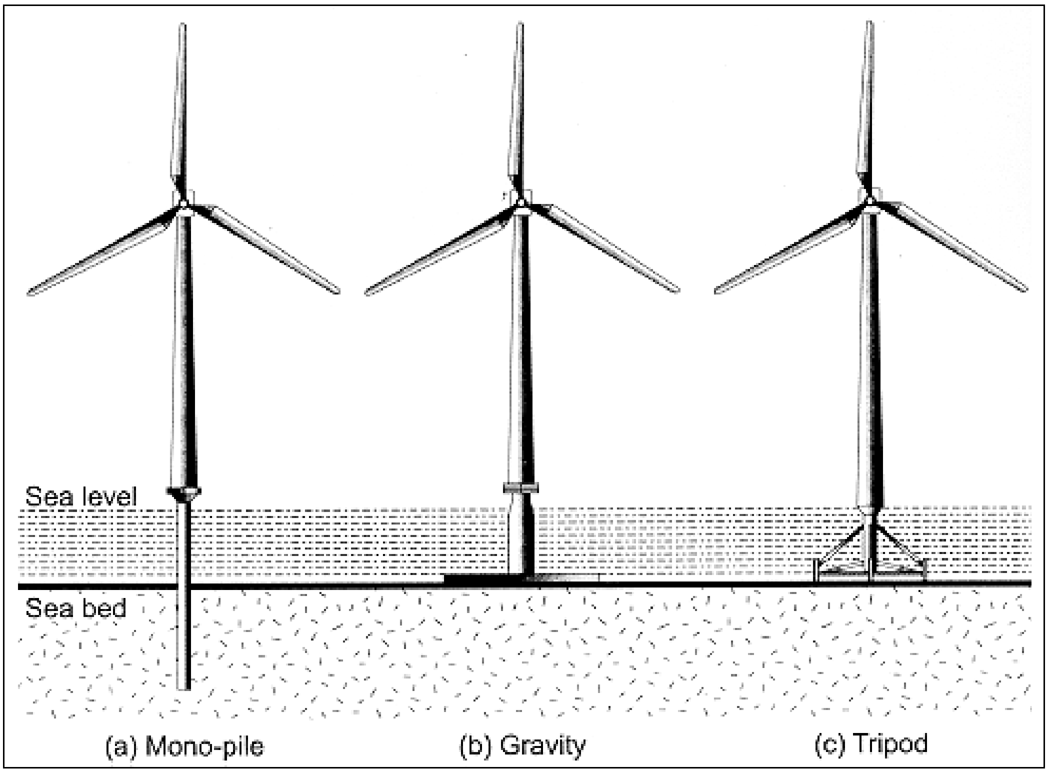


Figure 2.2 Sketch of main foundation types
(source: CADDET, 2000)

2.3 Coastal Processes

2.3.a Introduction

For a successful wind farm development there needs to be an adequate understanding of the local coastal environment in terms of:

- Designing a scheme that can cope with the environmental conditions (eg sea bed, wind, wave and tidal regimes, etc) by establishing appropriate design parameters.

- Designing a scheme that the local environment can accommodate with minimum impact, by undertaking an Environmental Impact Assessment.

Coastal processes play an important role in both of these issues. A brief overview of the pertinent features of coastal systems, related to England and Wales, is provided to set the context for this project and subsequent site specific studies.

2.3.b Coastal Systems

Coastal regions can be better understood if they are conceptualised as a series of inter-linked 'open' physical systems (often referred to as coastal cells or sediment circulation cells) which contain offshore, nearshore and shoreline elements (Figure 2.3). Sediments within these systems may be gravel, sand, silt or clay or mixtures of these grades. This material may be moved around by the action of waves and tides in a series of these linked systems.

At the simplest level, a coastal system may have a sediment source area, a region in which transport and sorting takes place and a depositional or sink area or areas. Each type of sediment may respond differently to applied conditions of waves and tides and have a different source, pathway and sink area.

Within an inter-linked system, interference within any part of one system may change the balance by causing additional deposition in one area and starvation of sediment in another. Such a change may then have a knock-on effect in an adjacent system. For example, a breakwater or a series of groynes will impede the flow of sediment along the coast, or excessive offshore dredging may trap sediment moving towards the coast. Hence, any interference to sediment movements in a coastal system must be carefully considered.

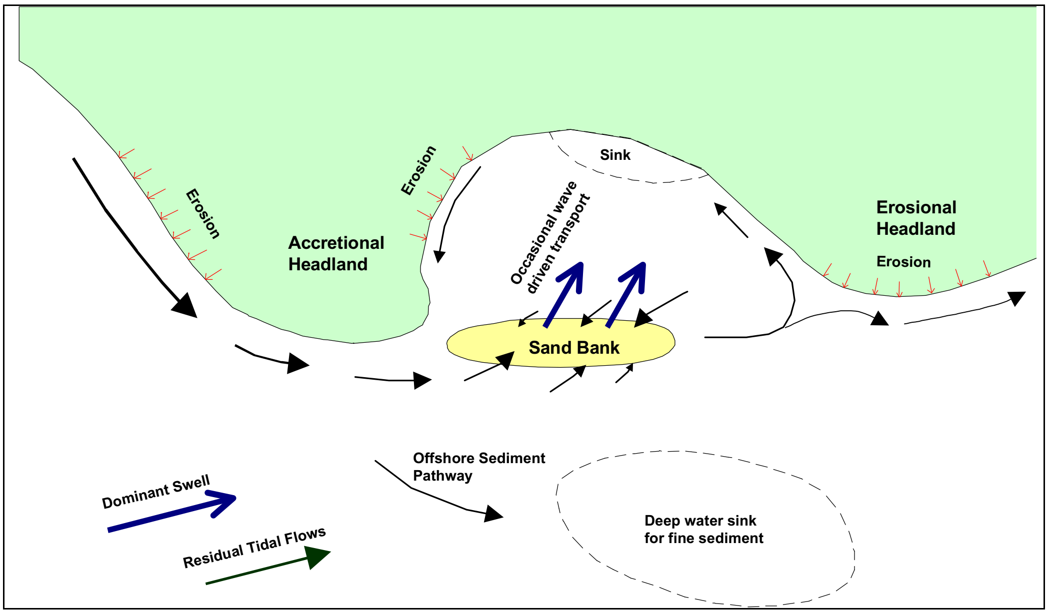


Figure 2.3 Hypothetical coastal cell

2.3.c Sediment Regime

- Sources

Mobile sediment that can take part in coastal and nearshore processes is largely derived from the erosion of cliffs or the sea bed by tidal currents and wave action. Although in other parts of the world rivers are an important source of sediment, around Britain the contribution from rivers is generally small. However, the contribution of sediments from cliffs is immense in those areas that are still unprotected by adequate sea defences and where the cliff geology is susceptible to high rates of erosion. The resulting wide variety of materials that are released by cliff erosion are transported initially by wave action in the littoral zone and then further offshore by tidal currents.

The other major source for sediment is the sea bed where action by tides and storm waves can erode and transport materials that are sufficiently unconsolidated.

- Sediment Transport

Sediments can move in the coastal environment either as suspended load or bedload. The actual mode of transport is related to particle size, density of material and the energy of the applied transport mechanism. Where this energy is applied to an erosional area then the mass transport of the released sediment creates a sediment pathway. Offshore this pathway is generally directed down the tidal current pathway, especially where this is complemented by storm wave generated currents, towards a sink area where tide and wave energy is less. In the littoral zone, transport is usually in a direction influenced by the dominant wind wave forces.
For sites in the nearshore littoral zone, the respective Shoreline Management Plans (SMP) provide an overview of the shoreline, offshore geology and geomorphology, sediment types, sediment transport regime and known sediment pathways for the net drift of sediments. These are typically parallel to the coast as long-shore transport and act in response to wave-driven currents.

Around sandbanks, sediment pathways are inherently more complex, commonly with local circulation cells (Caston,G.F. 1981., McCave,I.N. & Langhorne,D.N. 1982., and Venn,J.F. & D'Olier,B. 1983).

Cross-shore transport between the shoreline and offshore area is often associated with nesses, coastal promontories that are composed of sand or gravel (McCave, I.N. 1978). Nesses are commonly found on actively eroding coastlines or where there is a change in coastline orientation. Their formation implies a change in the rate of long-shore transport. Where there is also a high variance in the direction of wave approach, the long-shore transport may be in opposite directions on each side of the promontory causing either a littoral drift convergence or divergence. In the case of a convergence, the resulting sediment surplus creates an elongation of the ness and eventually a bar or bank linked to the coast. In some cases the ness and its connected bar can migrate along the coast in response to coastal erosion (Dugdale, R.E. 1980). Many of these shore-connected banks are to be found around the coast of England and Wales lying at a variety of low angles to the shore with a mean angle of approximately 20º.

- Sediment Deposits

Sediments can form large-scale deposits in the form of sandbanks or sand-wave fields. However, this does not imply that there will be no further movement of the sediment beyond these local sinks. On the contrary, most sandbanks and sand-waves have a highly mobile surface layer with sand movement largely in the form of megaripples. Generally, it is found that this movement is ebb dominated on one side of the bank and flood dominated on the other, resulting in the sediment forming its own circulation cell. The crest of shallow banks is often dominated by wave activity, which may also be the limiting factor to the bank height, and result in marked changes during storm periods. Some of this sediment may be lost from one or either ends of a bank and then move onward down the sediment pathway. This might mean onward to another bank or sand wave field, or to the shoreline.

The potential for any significant reduction in sediment supply to an adjacent coastal cell would only exist if there were a significant change to the circulatory flows of water and sediment around the sandbank. These typically, but not exclusively, clockwise circulations, transport sediment along the flanks, towards the ends, and simultaneously towards the crest, (Venn & D'Olier 1983) thus maintaining the bank. If there was additional sedimentation taking place on the bank this could be due to an intermittently increased supply from source, or to some impeding activity such as localised flow changes within the sub-cells. Evidence suggests that sandbanks are not necessarily closed systems (Caston 1981) and sediments may arrive at one end and some leave at the other. Such a sandbank sediment circulation model suggests that impedance will not preclude sand from still being lost from the end and thus continue to supply adjacent coastal cells, as the loss is due to inequalities of residual, water flows at the downstream end.

2.3.d Waves

Waves can have a significant impact on the coastal zone, both by themselves and in combination with tidal currents.

Waves originate from meteorological forcing and are therefore episodic in occurrence. Seasonal variations are normally pronounced and inter-annual variations may also be apparent. The wave regime in any coastal location is a combination of locally-generated seas and deep-water waves propagating into the area (eg long period swell). In general, high wave activity will occur at sites that are open to long fetches associated with prevailing wind directions. Hence, prevailing south-westerly winds and long Atlantic fetches leave the Bristol Channel exposed to peak wave activity. The North Sea and Irish Sea have more restricted fetches, and therefore (relatively) less wave energy will penetrate these areas.

As waves approach the coastline a number of modifications occur as the water depth decreases. These are:

- Shoaling and refraction
- Energy loss due to breaking
- Energy loss due to bottom friction
- Momentum and mass transport effects

These wave processes are discussed briefly in the following paragraphs.

Shoaling and refraction are usually considered together because they are both due to changes in propagation speed and wavelength as the waves 'feel the bottom', ie move from depths in which the vertical wave motions (which decrease with depth from the surface) have decayed to nothing by the time the bottom is reached to depths where they are constrained by the presence of the bottom. The effect of **shoaling** can be most simply described by following some swell waves from deep water in towards the beach. At first the propagation speed (group velocity) increases and so that the shoreward flux of energy is maintained, the wave height decreases. The waves reach their minimum height (having suffered a reduction of around 10%) when the depth is about $\lambda_0/6$, where λ_0 is the deep-water wavelength. As the waves move into still shallower water, the group velocity decreases and the height of the waves correspondingly increase, reaching their offshore value in a depth of just 1.6% of the offshore wavelength (10% of the local wavelength). At the same time the waves are becoming progressively steeper. The steepening wave profile eventually becomes unstable and wave **breaking** occurs, with consequent rapid loss of wave energy. Eventually, the whole wave motion is destroyed by breaking and frictional dissipation in the very shallow water on the beach.

If the waves propagate at an angle to the bottom gradient, one part of the wave will arrive in shallow water (and will slow down) before another. In order to maintain its integrity the wave front will have to turn, ie the propagation direction will change. This process is called **refraction**. Its main practical effect is to give rise to areas of energy convergence leading to high waves and compensating areas of energy divergence leading to low waves. When offshore banks are present, they lead, typically, to areas of strong convergence to the 'lee' of the bank. It is usually considered that discernible refraction requires the depth to be less than $\lambda_0/4$.

When the waves are under active generation by the wind, the shoaling process is modified somewhat. As the waves move into shallow water and their propagation speed decreases, the speed of the wind relative to them increases so that more energy is fed into the waves. This, combined with the reduction in wavelength discussed previously causes the waves to steepen rapidly until they become unstable and break. This process can be modelled by assuming that the extra energy fed into the waves by the wind is balanced by that lost due to breaking in such a way that the mean-square slope of the sea surface is maintained. It can be shown that the spectrum of the waves under these constraints is the TMA spectrum and this formulation has been used in the model runs in this study for mixed random seas. In the absence of wind, waves approaching a beach still steepen and ultimately break but, as discussed above, the process is delayed until quite shallow water is reached.

Energy is lost as the wave orbital velocities (which are horizontal to and fro motions near the sea bed) do work against friction. Frictional losses are small but over large areas of shallow water (depth perhaps 10% of the local wavelength) this process makes a significant contribution to the decay of the waves.

Even in deep water there is a mass transport due to waves. This is caused by the forward motion under the crests being marginally greater than the backwards flow under the troughs. As waves move into shallow water this process becomes more pronounced leading to a number of important effects. When the waves break on a beach the mass flux is sufficient to raise the quasi-steady water level at the beach. This is called wave **set-up**. Typically, the water level is raised by 10% of the significant wave height. Thus if the offshore significant wave height is 2m, the quasi-steady water level is raised by 0.2m. In a system of coastal sandbanks the mass transport due to waves can give rise to circulation systems which modify the tidal circulation and the sediment transport regime.

Spatial gradients of currents cause changes to the propagation speed of the waves which give rise to current-related shoaling and refraction effects.

The direction of waves on a beach can be very important. When there is a symmetrical and rectilinear tidal current regime, the net sediment transport path may be largely determined by the direction of the waves.

When a surface piercing structure is placed on the sea bed in the path of waves propagating to the coast, **diffraction** of waves around the structure and **reflection** of waves off the structure will occur. Offshore turbines and their foundations will interact with the wave field in this manner. For a given wave height and wave length, the extent and degree of the interaction increases with the size of the structure.

2.3.e Tidal Conditions

Unlike waves, tides are periodic in their occurrence being a response to gravitational forces from neighbouring planetary bodies, with the sun and moon having the major influence. UK coastal waters are subjected to a tidal wave that moves from the Atlantic Ocean onto the European continental shelf. This tidal wave arrives from two directions, the south-west and north-west approaches. From the south-west approaches, propagation is along the English Channel into the Bristol Channel and towards the Irish Sea. From the north-west approaches, propagation is into the Irish Sea and North Sea. As the tidal wave moves across the shallow continental shelf interactions occur with the sea bed and landforms to produce complex regional variations and tidal resonance. In particular, the tidal regime in the North Sea includes 3 amphidromic points where the tidal range is effectively zero, and the phase of the tidal wave moves around these points in an anti-clockwise direction. One of these amphidromes is close to the Danish coast. Away from the amphidromic points the tidal range increases.

Around the UK the period of the tidal wave between successive high waters is semi-diurnal at around 12 hours 25 minutes. The amplitude of the tide is generally higher around the UK and in the Baltic Sea and off the Danish Coast, with largest ranges in the Bristol Channel and Irish Sea and more moderate along the East Coast. Strongest tidal currents are normally associated with the areas with largest tidal ranges.

Variations in water depth as a function of the tide also influence the behaviour of waves, with low water conditions moving the position of sea bed interaction seaward, and high water conditions moving the position landward.

Frictional losses in tidal energy on the sea bed can result in large-scale sediment movements, and the regularity of the tidal regime provides a sorting mechanism for different sediment types.

Local obstruction to the tidal currents may create local variations in flows, and small-scale turbulence. Around structures on mobile sediments, the increased turbulent flow will contribute to the development of scour.

2.3.f Categorisation of Effects

It is important to note that the range of potential effects on coastal processes may occur locally within the wind development area and remotely as a consequence of the development. For the purposes of the present study, the effects are categorised as follows:

Local effects (near-field – the area within the development site)

- Local scour around the base of the wind turbine and the cumulative effect of multiple structures
- Effects on the local wave climate and how this might influence sediment mobility

Remote effects (far-field – the area surrounding the development site, including adjacent coastlines)

- Changes to the current flow regime, and how this may alter sediment pathways
- Changes to the wave regime, caused by diffraction, focusing effects and sheltering, and how these may impact upon sediment transport and the coastline

To investigate these issues has required identification of the potential and known offshore wind development areas, the components of these developments and the coastal process conditions associated with each site.

2.3.g UK Interest Areas

A brief overview of the elements of coastal processes applicable to UK interest areas is both beneficial and necessary to outline the relative importance of these processes around UK coastal waters. The main elements of coastal processes are waves and tides which act together in coastal waters to determine the regional and local behaviour of the sediment transport regime.

The First Round offshore sites allocated by Crown Estate are all positioned within the following UK Interest Areas:

- North East England
- East Coast (Humber Estuary to mid East Anglia)
- Thames Estuary
- Bristol Channel
- Liverpool Bay
- Solway Firth

The choice of these areas are based upon an assessment of a range of factors including suitable wind conditions, environmental sensitivities, accessible connection to the National Grid, and sea bed conditions. Appendix D provides an overview of the coastal process conditions across these 6 generalised areas.

2.3.h First Round Sites

Some general features of the coastal environments associated with the first round of offshore developments can be identified, in that:

- all sites are relatively close to the coast (an average distance of around 7km),
- all sites are in comparatively shallow water (on average 10m at low tide),
- all sites are located in areas with a high offshore wind resource and in exposed coastal locations,
- several sites are associated with sandbanks.

At these coastal locations, tides and tidal currents, waves, wave-driven currents, and wave-current interaction are likely to be the main coastal processes and, in turn, will be the driving conditions for sediment movement.

The relative importance of tides and waves can be related to the local coastal exposure and water depth, with a general rule that sediment transport is dominated by tidal currents in deep water (>15m) and by wind generated waves in shallow water (<5m) (OWEN, 1999). In addition, the proximity of the wind farms to the coast requires some consideration of inshore processes.

Further characterisation of coastal process conditions for first round sites is provided in Appendix E.

2.3.i Review of Potential Effects of Marine Structures

- Turbine Support Structures

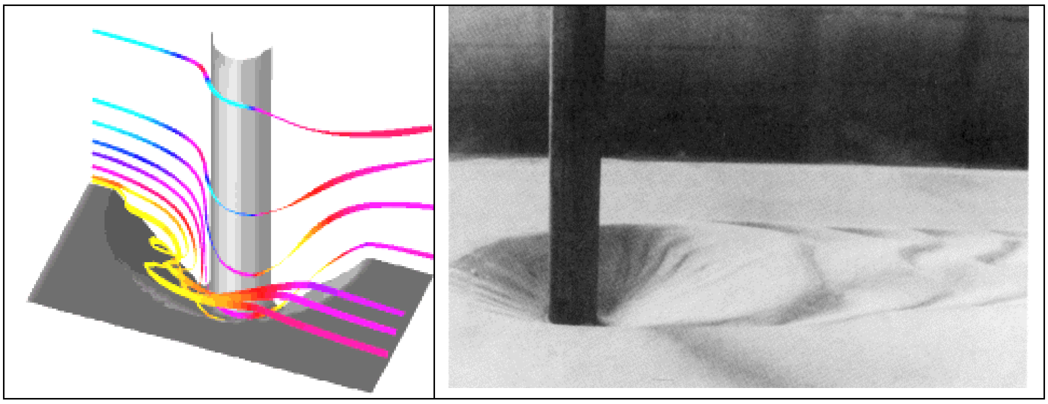
Structures that are placed in the marine environment have the potential to modify local coastal processes.

A locally modified flow and/or wave regime created by an obstruction can lead to an increase in the turbulent intensity of the flow due to the generation of vortices (wake vortices) from the structure. Even in cases where the flow speed upstream of the structure is below the value of the threshold for sediment mobility, the local increase in velocity adjacent to the structure can amplify the value of the sea bed shear stress to levels which exceed that threshold and thus induce scour.

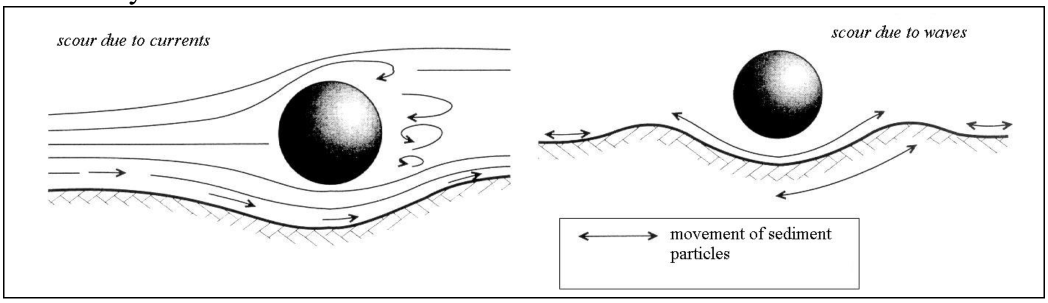
Siting of structures upon an area of high sediment mobility may result in disruption to this sediment movement, either producing excessive local scour or by the localised accumulation of sediment close by. This has the effect of starving, or diverting at least temporarily, the down-drift areas of their normal sand supply.

Siting of a wind farm on areas of the sea bed away from the main sediment pathways and depositional centres is unlikely to seriously affect sediment interchange between the offshore zone and the nearshore. However, the use of sites in areas of active erosion might accentuate that activity, or even cause such activity in closely adjacent areas.

Figure 2.4a provides a schematic of scour development around a vertical cylinder.



a. flow structure interaction and scour development around a vertical cylinder



b. flow structure interaction and scour development around an exposed cable (in cross-section)

Figure 2.4 Schematic of scour development around structures

Scour at a structure can be considered to be *general scour* and *local scour*, each process acting over different spatial and temporal scales. General scour typically occurs over longer time scales and acts over larger areas, such as the overall adjustment of the sea bed in response to a storm event. Local scour results from the immediate impact that a structure makes on the local coastal processes, such as a mono-pile structure modifying local flow patterns.

The design process for placing such a structure on the sea bed has to consider the net effect of the combined scour process and provide appropriate scour protection measures to minimise the risk of failure in the foundation.

The development of scour in the marine environment is influenced by:

- Geometry of the structure: dimensions, shape and spacing of piles
- Sediment characteristics: sediment size, specific density
- Coastal processes: tidal flows, water depth and wave regime

The following general relationships for scour can be expected:

- Scour development is usually very rapid in the initial phase after construction, with a rate which decreases until an equilibrium scour depth is reached.

- Scour development due to current and wave action is faster than scour due a current alone.
- Scour effects created by waves only are small.
- Shallow water waves with currents may even reduce scour depths, whereas breaking waves (and long period swell) with currents tend to increase scour.
- The typical shape of scour development is an inverted cone.
- Scouring has to be considered in the design process, even if there is no sediment transport in the ambient condition.

Most investigations of scour development have been undertaken using small-scale experiments in physical models with single piles. From these various investigations a range of empirically based relationships have been derived, each limited in some way to the choice and range in parameter values being tested. Typically these relationships determine a ratio between the maximum scour depth and the pile diameter (D). It is also further regarded that the extent of scour may be around 6 to 10D, and hence, if the turbine spacing is greater than this extent cumulative scour effects are unlikely.

Further details and methods to predict scour development around structures placed in the marine environment are provided in Appendix G.4.

- Marine Cables

The options for marine cables are either laying on the sea bed or burial. Cables laid upon the sea bed can cause a local vertical flow separation which re-attaches itself some 6 to 10 cable diameters downstream. Also turbulent eddies are produced from the top of the cable to form a lee wake. This wake leads to fluctuations in shear stress on the downstream side which induce scour. This process is repeated on the reverse side of the cable as the phase of the tide changes, between flood and ebb. The significance of the lee wake increases under the action of waves. Figure 2.4b provides a schematic of scour development around a horizontal cable lying on the sea bed.

2.4 Scenario Definition

It is important to recognise that the significance of any given coastal process parameter will vary from site to site, and the changes brought about by any offshore wind farm development could be positive or negative. Site specific investigations are therefore required to determine the implication of these issues and to support the successful development of any scheme.

The aim of the present study has been to consider the potential effects of offshore developments on coastal processes in generic manner. The study has required the definition of a 'reasonable worst case' and 'typical' wind farm scenario. In addition, a 'best case' scenario has also been considered so that the definition of a 'typical' scenario can be established at a suitable point between the properties of the 'best' and 'reasonable worst case'. It is important to note that the present study has developed these definitions in the context of the first round sites presently under consideration and in reference to potential impacts on current, wave and sediment conditions. It is possible that future rounds for site allocation may require some revision of these definitions.

The present definitions have then been used to describe generic scheme layouts and through the application of computational modelling, an assessment has been made on the potential effects on coastal processes.

2.4.a ‘Best Case’

The perception of ‘best’ case, in reference to its potential impact, is one that results in no change from the present (baseline) conditions.

This scenario may be defined as follows. A scheme leading to:

- No apparent change to the gross properties of the tidal regime, apart from local scale turbulence around wind turbines. Most likely a low tidal energy site.
- No apparent change in the gross properties of the wave regime, apart from local scale wave reflection off wind-turbines. Most likely a low wave energy site.
- A widely spaced arrangement of structures within the wind farm that has no perceptible cumulative interference effects.
- Local sea bed conditions that are insensitive to local changes in the tidal and wave regimes and do not respond with local scouring around the turbine base, and where cable burial is possible. Most likely an area with relict sea bed deposits.
- An adjacent shoreline that is remote and stable. Most likely a sheltered and ‘closed’ coastal sediment cell without up and down-drift linkage.
- No net effect in obstructing a sediment pathway. Most likely an area with no cross-shore transport.

2.4.b ‘Reasonable Worst Case’

The perception of ‘worst’ case, in reference to its potential impact, is one that results in the maximum undesired change from the present (baseline) conditions.

This scenario may be defined as follows. A scheme leading to:

- An apparent change to the gross properties of the tidal regime. Most likely a high tidal energy site.
- An apparent change in the gross properties of the wave regime. Most likely a high wave energy site.
- A closely spaced arrangement of structures within the wind farm that leads to cumulative interference.
- Local sea bed conditions that are sensitive to local changes in the tidal and wave regimes and respond with local scouring around structures. In

addition, the cable route prevents burial, and local armouring is required. Most likely an area with active sea bed deposits.

- An immediately adjacent shoreline that is vulnerable to erosion. Most likely an exposed and 'open' coastal sediment cell with strong up- and down-drift linkage.
- The location of the wind farm is immediately within a sediment pathway, with a consequence that modified flows and waves alter the pathway. Most likely an area with strong cross-shore transport which supplies sand to beaches and/or an area of sub-tidal longshore transport.

2.4.c 'Typical' Case

By adopting the previous definitions for 'reasonable worst case' and 'best' case, it is intended that the actual first round sites all fall within the limits of these definitions. It is also likely that each individual site holds some features of the 'worst' and 'best' cases, but with no particular site achieving an exact 'typical' status on all issues.

The definition of 'typical' case has made reference to the upper and lower limits in the range of conditions likely to be encountered, from which a typical (or average) value has been determined. Where an issue can not be averaged then a range of typical circumstances has been considered (eg sea bed profiles for the first round sites are commonly associated with a sandbank, but the average profile would loose this feature). Certain parameters are assumed to be fixed for all cases, such as the number of turbines (a restriction of 30 maximum in the first round), whilst other parameters described through Section 2.2 and Appendix E cover a range of values.

2.4.d Scroby Sands

Based on the above criteria, Powergen's site on Scroby Sands off Great Yarmouth (Site 10) is probably most comparable to the 'reasonable worst case' scenario outlined above, in terms of proximity to shore, coastal processes, sediment availability, etc. Details from an existing review of coastal process effects related to the Scroby Sands site are provided in Appendix B.

3 MODELLING

3.1 Model Selection

Computer models are already in widespread use to support a broad range of coastal investigations. These include applications for aggregate dredging, spoil disposal, cable burial, pipeline siting, dispersion from long-sea outfalls, etc. The range of issues to be addressed in the consenting of offshore wind farms is somewhat similar to those outlined above, with regulators and statutory consultees requiring sufficient information to be convinced that such schemes do not create unacceptable or 'significant' environmental impacts. Established methods and models exist to support such assessments and are considered to be generally transferable to study the potential effects of offshore wind developments on coastal processes, but importantly require site specific data for model validation in each case.

In the case of aggregate dredging, both DETR (now DTLR) and NAW are in the process of revising guidance on aggregate extraction (DETR, 2001). In particular, the physical impacts of aggregate dredging require a coastal impact study (CIS) to assess:

- Implications for coastal erosion
  - whether the dredging will interrupt the natural supply of materials to adjacent beaches (ie interference with sediment pathways).
  - the likely effect on bars and banks which provide protection to the coast by absorbing wave energy, and the potential impact on local tidal patterns and currents (ie interference with tidal dynamics).
  - the likely changes to the height of waves passing over dredged areas and the potential effect on the refraction of waves leading to significant changes in the wave pattern (ie interference with wave regime).

- Implications for local water circulation resulting from the removal or creation of topographical features on the sea bed (ie interference with tidal regime).

In addition, DEFRA, in their capacity as acting as the authority who grants a FEPA license, has outlined their requirements for an offshore wind development EIA in a guidance note (MAFF, 2001) which lists key topics which will need to be addressed. This note states that an EIA needs to consider the potential impact of the development and the extent to which any adverse effects might be mitigated or compensated in terms of the risk to (*Note:* selected here for topics related to coastal process issues only):

- Modification to the hydrography and sediment transport patterns;
- Geomorphological changes and coastal erosion.

It is also acknowledged that CEFAS has prepared additional recommendations for the scope of investigations to be included in the EIA to satisfy FEPA and CPA requirements (CEFAS, 2001).

Previous considerations on some of the modelling requirements for offshore wind farms were also reported in OWEN, 1999. The value of such modelling in any study is, however, entirely dependent on the following:

- **Knowing which processes to model** - understanding the physical processes acting within the coastal environment and the type of developments to be described in the model. This must include consideration of the importance of any 3-D issues.

- **Selecting a model that is capable of describing these processes** – ensuring the model(s) include the correct mathematical expressions to describe the relevant physical processes, and that it can be implemented at the required scale.

- **Understanding the limitations of models** - knowing what the model cannot deal with, and documenting these limitations within the ES.

- **Equipping the model with sufficient representative data** - constructing a model with good site data to describe the profile of the sea bed, the sediment types and surface roughness; describing correctly the hydrodynamic conditions of waves and tides; and identifying sediment pathways.

- **Model proving** - undertaking calibration and verification for each of the processes involved to an acceptable standard through comparison of predicted values to actual measurements.

- **Applying the model** - running the model appropriately to investigate the issues and generating the required outputs.

- **Interpreting outputs** - making sense of the information generated by the model to determine the result.

- **User Competence** - engaging the appropriate level of experience and expertise in terms of understanding coastal processes and having a proven modelling capability.

The coastal process issues to be included in modelling the areas related to the first round development sites are:

- Tidal behaviour
- Wave regime
- Combined effect of wave-current interaction on sediment transport

Appendix F provides a brief review of models suited to addressing each of these issues.

3.2 Configuration of Models

From the range of applicable 'coastal area' models described in Appendix F the study has adopted the use of the DHI MIKE21 two-dimensional depth-averaged modelling system to provide an integrated description of tidal, wave and sediment transport processes. MIKE21 offers a modular approach that allows all of the identified coastal process issues to be investigated, based on a single modelling system. Similar modular approaches are also provided by Delft3d and by Telemac. The project makes no claims to solely endorse MIKE 21.

3.2.a Schematic Layouts

Modelling studies have been based on an idealised coastal layout with an offshore boundary 20km from the coastline and taken to an approximate depth of 30m where a deep-water wave condition can be applied. The extent of the model in the along coast direction is specified at 10km either side of the development. This layout is considered to offer a sufficient size to contain the dimensions of all first round wind farms in terms of distance from coast, scale of development and water depth.

A further generalisation is the inclusion of a sandbank for siting the offshore wind development. This feature is consistent with around 50% of the first round sites, and may reflect highly mobile sediment environments. The dimensions of the sandbank are based on a uniform elliptical shoal with a length, width and height typical for the UK coast.

The full extent of the conceptualised domains are described with a model grid with a uniform dimension of 45m, which is used to provide a regional / far-field description of the wave and tidal regimes. Within the regional grid is a nested 15m grid used to resolve the flow field around the sandbank. Within the 15m grid is a further nested high resolution local / near-field model covering the extent of the wind farm and with a grid dimension of 5m. The features of the offshore development are described in the near-field model at this detailed resolution.

3.2.b Representation of Offshore Wind Developments

Within these conceptualised domains a 'reasonable worst case' and 'typical' scenario wind farm configuration have been conceptualised and a series of modelling studies undertaken to determine the potential effects on coastal processes brought about by the presence of the offshore wind development.

Figure 3.1 provides a view of the layout adopted for the 'reasonable worst case' domain. The offshore wind development is introduced into the 5m grid on a nearshore sandbank located approximately 1.5km off the coast, with an arrangement of 30 turbines in 3 rows of 10 and with a consistent separation of 300m.

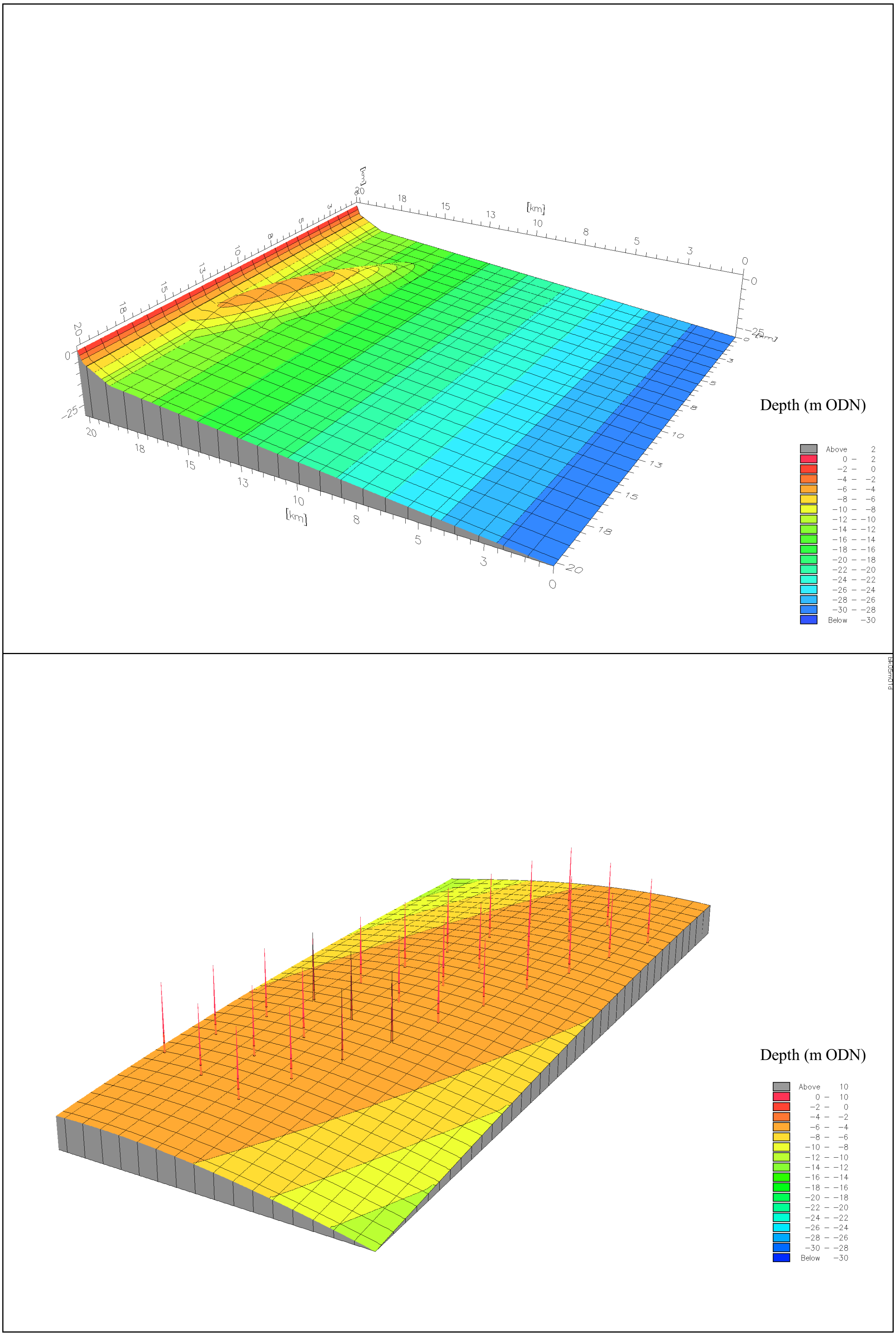


Figure 3.1 Schematic layout of ‘reasonable worst case’ scenario (top: full domain of regional model, bottom: wind farm configuration on near-field model)

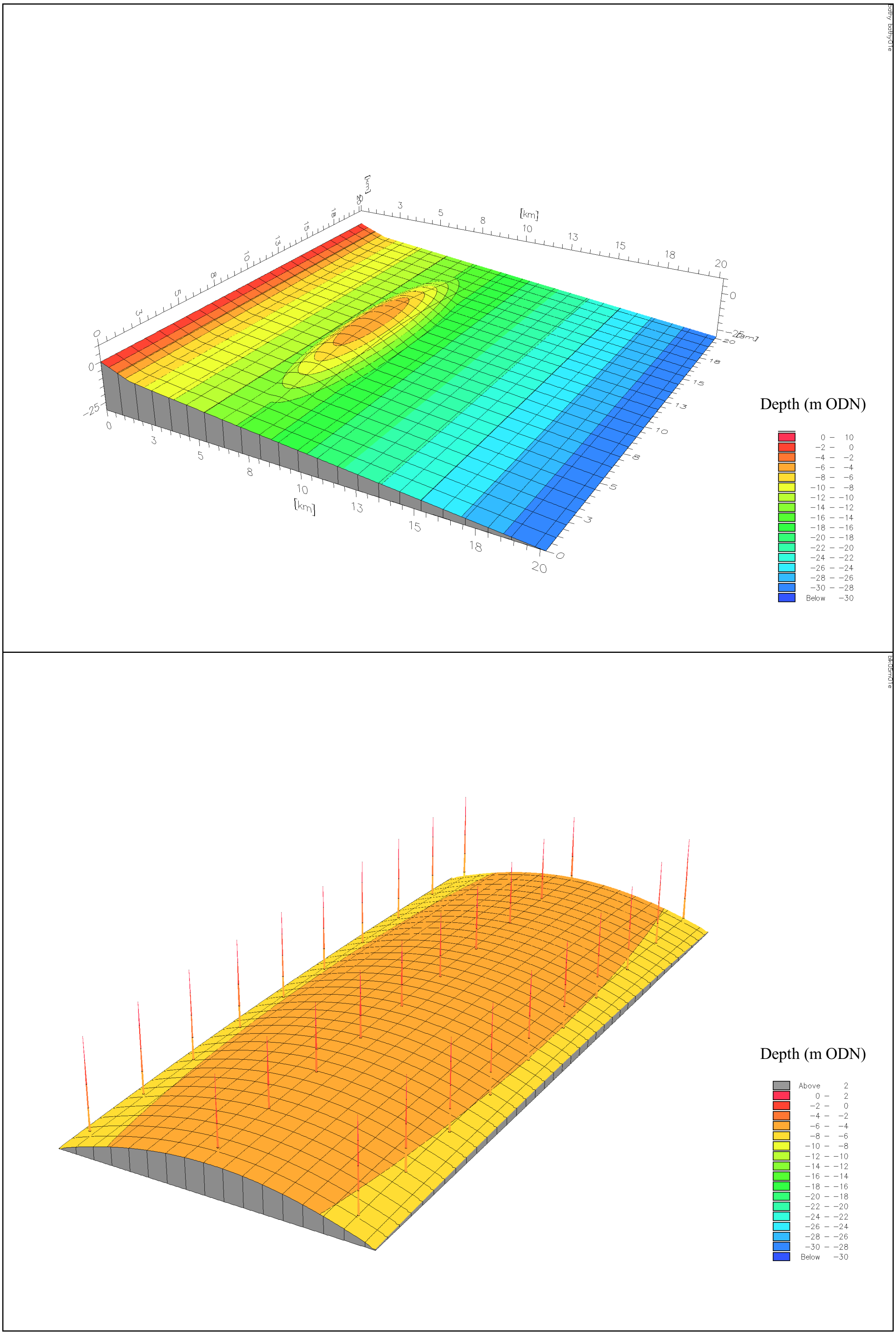


Figure 3.2 Schematic layout of 'typical' scenario (top: full model domain of regional model, bottom: wind farm configuration on near-field model)

Figure 3.2 illustrates the layout for the 'typical' scenario. The offshore wind development is introduced into the 5m grid on an offshore sandbank located around 7km from the coast, with an arrangement of 30 turbines in 3 rows of 10 with a separation of 700m between rows and 400m within a row.

Of note in the 'reasonable worst case' is the choice of angle for the sandbank relative to the shoreline. The intention here is to provide direct linkage between the coast and the sandbank by rotating the elliptical bank by an angle of 20°. This is determined to be representative for shore connected banks, which lie at a low angle to the shore, with a mean value of approximately 20°.

To verify that these two conceptual layouts provide a generalised description of the sites associated with the first round of wind farms, representative cross-section profiles have been derived for each of these sites and compared with those from the conceptualised layouts. These comparisons are illustrated in Figure E.1 and E.2.

3.2.c Hydraulic resistance from the turbine support structures

The forces transmitted to the turbine support structures by waves and currents have the effect of removing some energy from the marine environment. This energy would otherwise have been consumed in wave and tidal current mass flow, which, in turn, would provide a sediment transport capability. The resistance offered by the turbine support structures has been calculated in terms of wave and current forces and has then been subsequently expressed as an equivalent sea bed resistance term in the regional wave and tidal models. This has the effect of locally reducing the wave and tidal current energy due to the resistance offered by the structures. This issue is separate from any wave scattering and reflection that may occur locally around the structures. Local effects upon the waves have been studied by using a fine grid wave model capable of directly predicting the diffraction, scattering and reflection effects due to the structures as described in Section 3.2.a.

3.2.d Tides

Tidal conditions have been modelled using two schemes each operating with and without the presence of the offshore development:

- MIKE21-NHD (Nested Hydro-Dynamic) operating across the 45, 15 and 5m grids.
- MIKE21-HD (Hydro-Dynamics) operating at 45m, with pier resistance to account for sub-grid structures (configured to account for hydraulic resistance of structures of 5m diameter). Further details on the basis of this approach are described in Appendix G.1.

Open boundaries apply a 6m range semi-diurnal tide (an average value for the majority of sites, but also a high value for several of the proposed nearshore sites), with an appropriate phase lag to enable flows to be generated with a speed of around 1m/s. Figure 3.3 illustrates the tidal regime applied to each scenario.

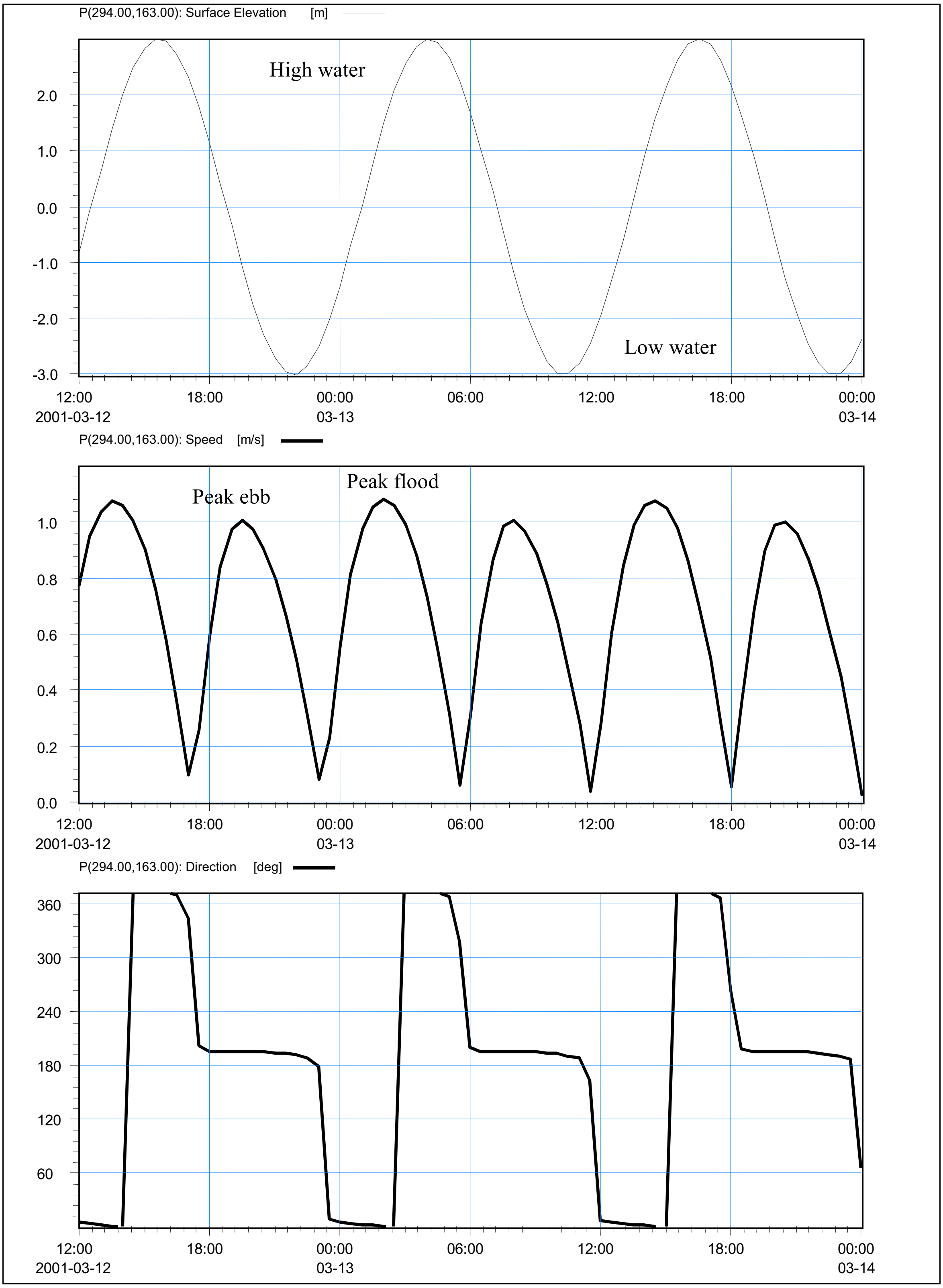


Figure 3.3 Time series of tidal boundary conditions
(top: water level, middle: flow speeds, bottom: flow direction)

3.2.e Waves

Regional / Far-field wave studies

Wave conditions in the regional domain which extends from the offshore boundary, across the development site, and up to and including the nearshore area, are described using the MIKE21 NSW (Nearshore, Spectral Wind-Wave) model. This model takes account of the processes of shoaling, refraction, bed friction, wave breaking and wave growth due to local wind fields.

Offshore boundary conditions are based on characteristic 'deep water' wave conditions for UK coastal waters, derived with reference to the Department of Energy Wave Climate Atlas for the British Isles (DOE, 1991), along with other available wave data (both modelled and measured).

For the purpose of the present study, wave climates representing 10 and 50% probability of exceedence were adopted, since this provides a representative long-term wave climate, rather than an extreme survival design event. The 10% exceedence probability condition was taken to be a significant wave height (Hs) of 2.5m with a mean period (Tz) of 7s, with the corresponding values for the 50% exceedence threshold taken as Hs = 1m and Tz = 5s. Wave tests also made allowance for variations according to tidal state with three water levels taken at high water, mid-tide and low water, as well as directions normal to the boundary of -30, 0 and +30° for the 'reasonable worst case' scenario, and -30, 0° for the 'typical' scenario. (*Note*: since the latter domain is symmetrical, then a result from +30° is identical to the -30° case)

The regional wave model uses a grid size of 45m, but does not account for reflection and diffraction, which makes it unsuitable for representing the influence of the structures directly. Consequently, the hydraulic resistance that the structures offer to the waves has been represented in the MIKE21 NSW model as an equivalent sea bed friction effect, with a value calculated to transmit the same force as that exerted by the structure. This friction effect is achieved by locally increasing the sea bed friction on a grid cell occupied by the structure.

The equivalent sea bed roughness factors for the 50% and 10% exceedence waves have accordingly been calculated as 0.08 and 0.19, respectively, and applied on one cell of the 45m model grid to represent each turbine. These roughness values are between one and two orders of magnitude larger than would arise by considering the size of the sea bed sediment alone. Appendix G.2 provides additional detail on the procedures used in determining the additional roughness terms.

Local / Near-field wave studies

The array of support towers as a whole will locally scatter waves and it is probable that the reflected wave components from each structure will interact. As a result, reflected wave heights immediately adjacent to the towers may be larger than those applying to one structure acting alone. The extent of the interaction will be a function of the spacing of the structures and the wave period. This consideration is relevant to predicting the amount of scour that may occur adjacent to the structures.

Local wave influences exerted by the structures have therefore been predicted in more detail by using the MIKE21 Boussinesq Wave Model (MIKE21 BW). In this model, it is possible and practical to use dry land cells to represent the structures. MIKE21 BW model is capable of describing the influences of reflection and diffraction, as well as refraction and shoaling. This wave model has been configured to cover the area of the wind farm at the level of the 5m grid.

Appendix G.3 describes the validation of the near-field wave model, MIKE21 BW.

<u>3.2.f Sediment Transport</u>

To investigate the influence an offshore development might make on sediment transport and pathways the MIKE21 Particle Transport (MIKE21 PA) model has been used. This model calculates the fate of particles moving within a tidal and wave regime, with the properties of particles defined to exhibit the behaviour of sand-sized sediments. This includes particle size, fall velocity and a critical Shields parameter.

Scenarios have assumed a hypothetical source of sediment introduced into the model at a position on the lower boundary. The source position is based on an assumed pathway running close to the shore, which for the 'reasonable worst case' also directly crosses the sandbank and hence the wind farm area. The model has been run for each of the 'far-field' regional models for cases with and without the wind farm development.

The advantage of using this type of approach is that the movement of the particles is in direct response to the forcing conditions of waves and tides. Where waves and tides are modified by the offshore development, the net effect of such changes is accumulated as the particles are transported across the model domain with these effects also accumulated through time. Hence, the net effect of any moderate change to coastal processes will be clearly demonstrated.

3.3 Consideration of Effects on Coastal Processes

The results from the modelling studies have been examined in detail to quantify the level of changes that may be brought about by the introduction of a wind farm development into the coastal domain.

3.3.a Changes to the tidal regime

Far-field tidal results

A sequence of tests have been carried out using a standard tidal event for each of the conceptualised domains to determine the potential effect an offshore wind development makes on the regional flow regime. The method of representing the wind development is achieved in two ways:

- Directly within the 5m near-field model.
- Indirectly within the 45m far-field model using hydraulic resistance (pier resistance option in MIKE21 HD).

Far-field model results from the 'reasonable worst case' scenario are presented in Figure 3.4 at the time of peak flood flows, and Figure 3.5 for peak ebb flows. In terms of regional scale variation, the perceived modifications to the flow regime are marginal and considered to be insignificant.

Results from the 'typical' scenario are presented in Figure 3.6 for the equivalent flood phase of the tide.

To assist quantification of these changes a range of positions in the model have been examined. These positions have been selected at a fixed distance of 1km off the coast and at various sites in the lee of the wind farms. Table 3.1 provides a summary of the maximum observed changes across these points.

Table 3.1 Maximum changes to the regional tidal regime predicted for the 'reasonable worst case' and 'typical' scenario for a transect 1km off the coast.

Parameter	**Speed**		**Time of greatest change**	**Direction (degrees)**	**Water Level**	
	(m/s)	**(%)**			**(m)**	**(%)**
Reasonable worst case	0.0024	0.72	mid tide	0.50	0.0011	0.017%
Typical	0.0005	0.11	mid tide	0.22	0.0002	0.001%

It is important to note that these changes are at a level which is at the limits of accuracy of conventional monitoring equipment, and hence the calibration accuracy of models developed for site specific studies.

Figure 3.7 presents the flood tide result for the 'typical' scenario using the pier resistance approach. In general, the values predicted using this method are consistent with the previous case, but with a slightly larger reduction in speeds. This is a consequence of the approach which provides a local increase to the sea bed friction which slows down the flow and across the dimension of a 45m grid cell. The former case provides a direct obstruction in the 5m grid, which diverts the flow around the obstacle.

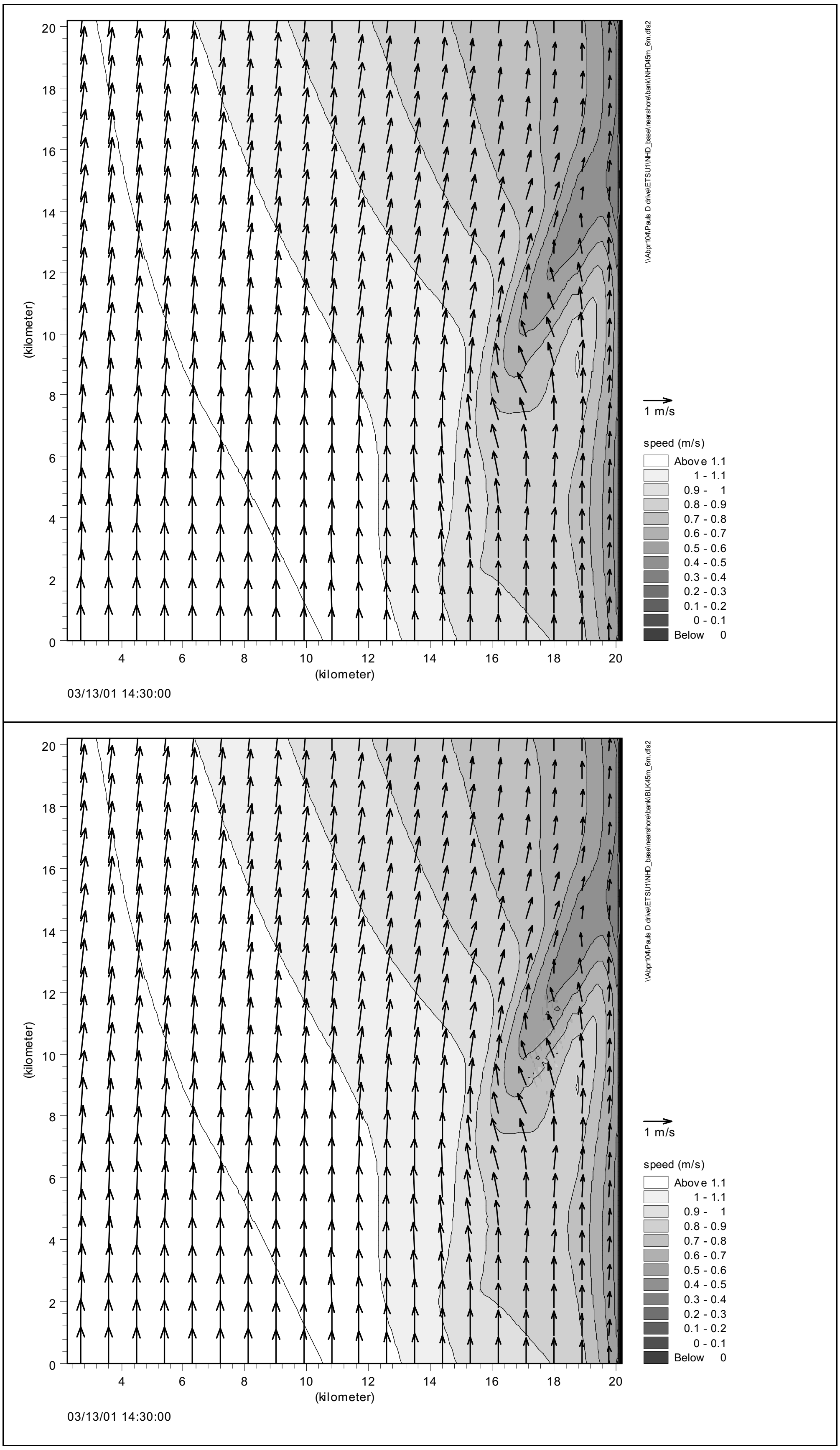


Figure 3.4 'Reasonable worst case' predicted regional flow patterns, peak flood flows.
(top: no wind farm, bottom: wind farm in 5m model of NHD)

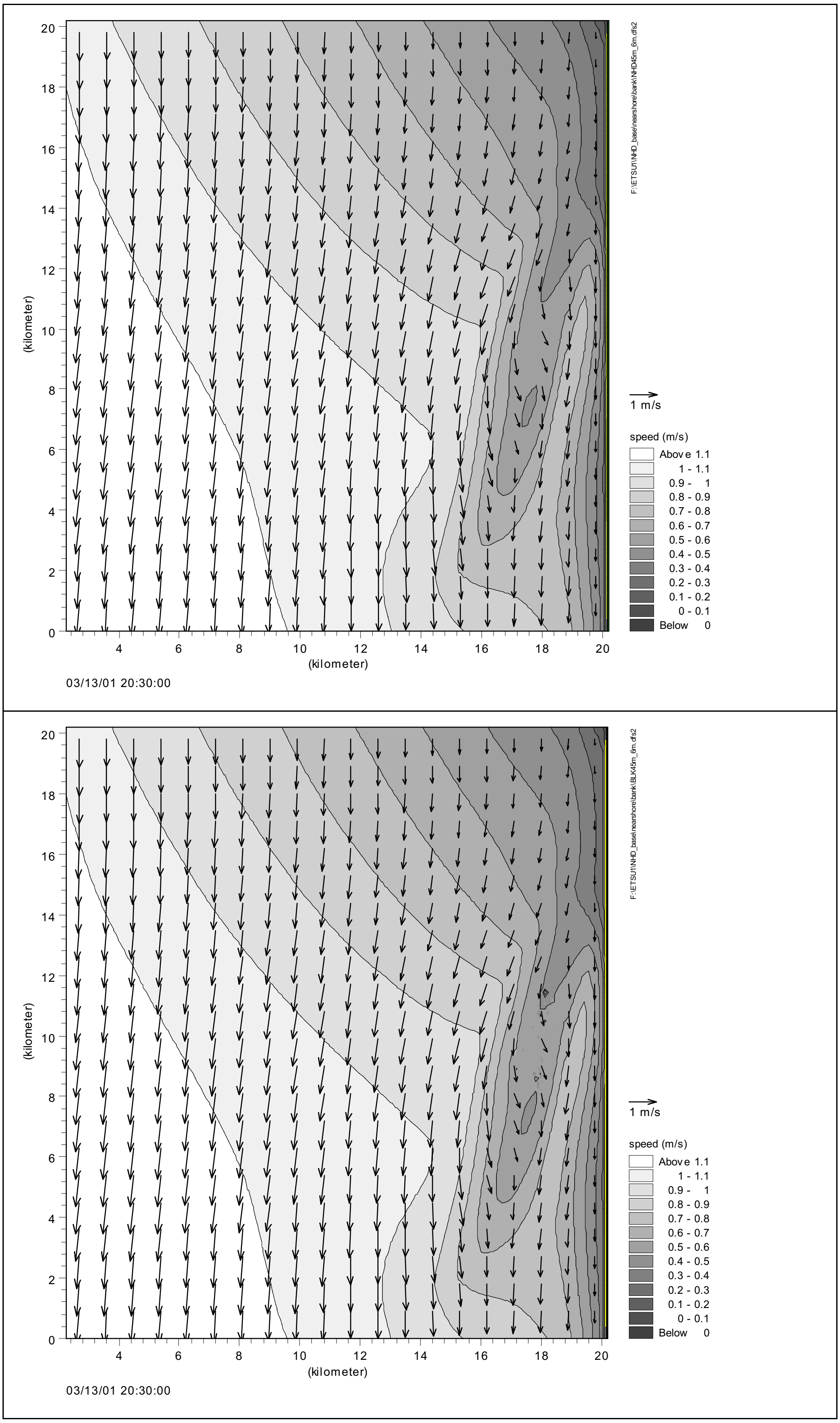


Figure 3.5 ‘Reasonable worst case’ predicted regional flow patterns, peak ebb flows.
(top: no wind farm, bottom: wind farm in 5m model of NHD)

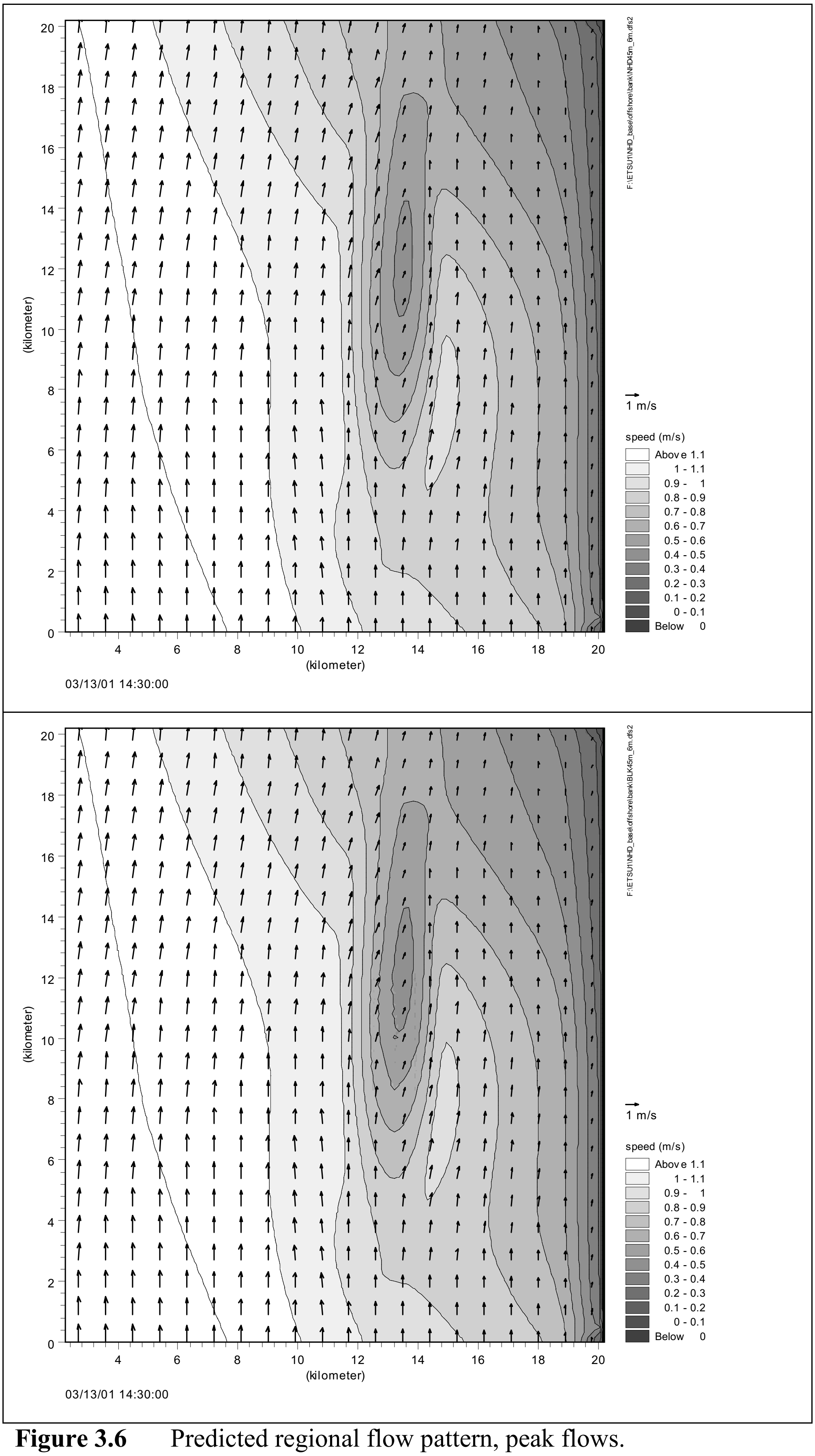


Figure 3.6 Predicted regional flow pattern, peak flows.
(top: no wind farm, bottom: wind farm in 5m model of NHD)

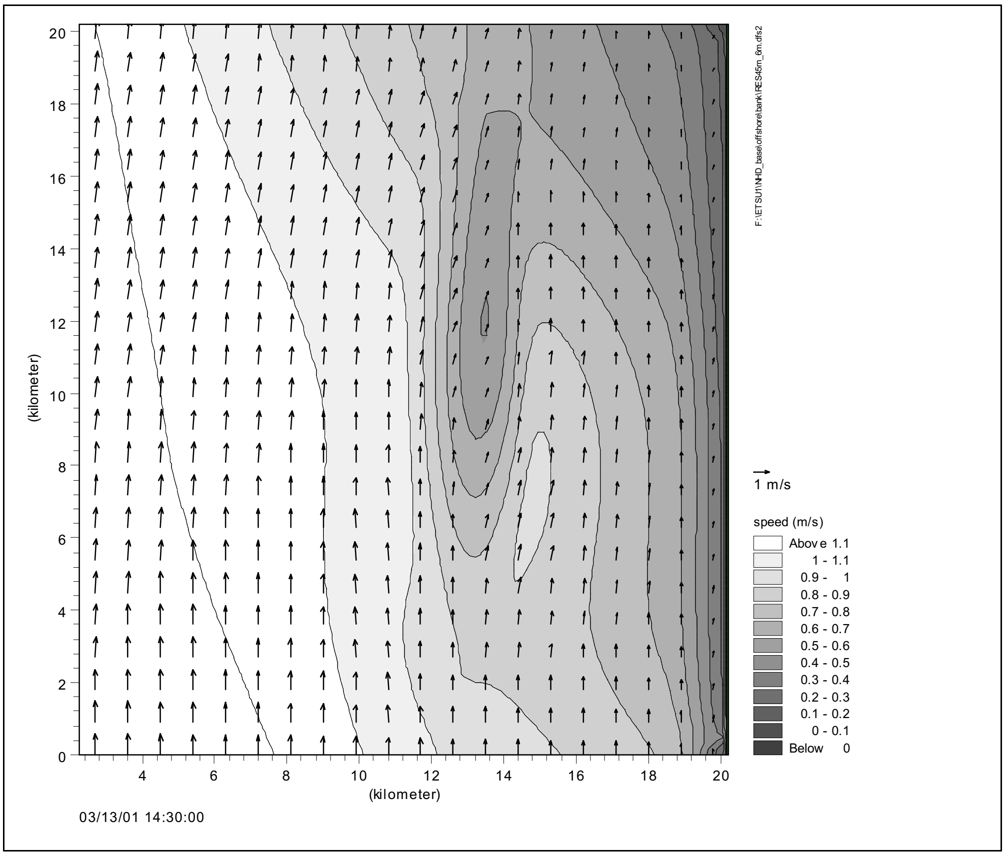


Figure 3.7 Predicted regional flow pattern, peak flows. (wind farm in 45m model of NHD, represented with pier resistance)

From the assessment of the far-field model results it appears that the regional flow patterns appear to remain unchanged when the wind farm is in position.

Near-field tidal results

Figure 3.8 illustrates the predicted flows for the ‘reasonable worst case’ for peak flood and ebb events. The presence of the array of wind turbines can be clearly seen by the appearance of ‘wakes’ in their lee superimposed on the more general flow patterns, with a moderate overlap between adjacent wakes.

Quantification of the local changes to the flows has been considered by sampling the results at various locations. The average reduction in local flow speed appears to be <0.1m/s, at peak flows of 1.2m/s, with a marginal change in direction of <0.5º. It is stressed that these changes are only local to the individual structures.

Similar results are obtained for the ‘typical’ scenario, and are presented in Figure 3.9. Due to the wider spacing across the array of support structures overlap in adjacent wakes is no longer apparent. Quantification of the local changes in the flow pattern suggests speeds are reduced locally by <0.05m/s, at peak flows of 1.2m/s, and direction by <0.4º.

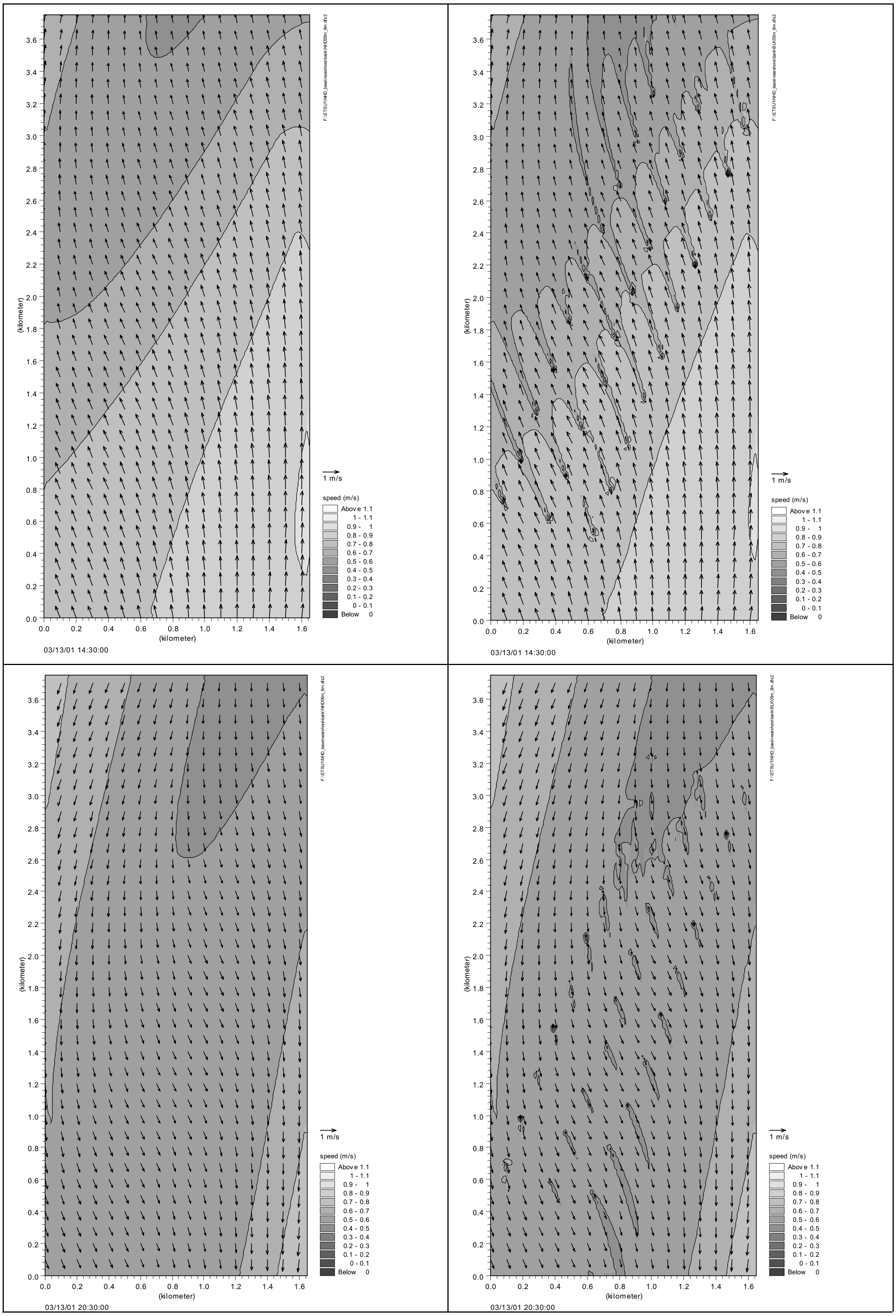


Figure 3.8 'Reasonable worst case' predicted local flow patterns, peak ebb & flood flows. (left: no wind farm, right: wind farm in 5m model of NHD)

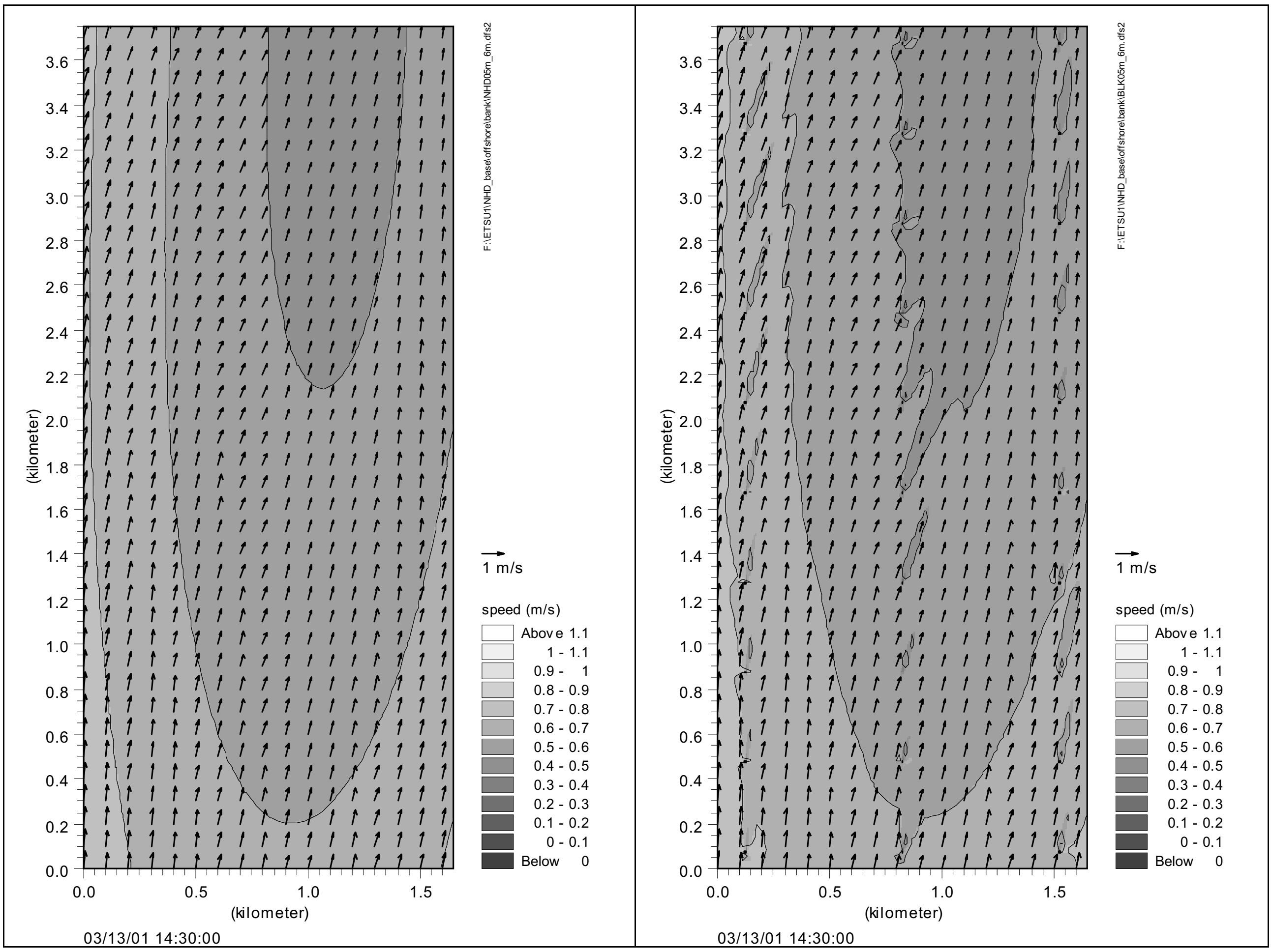


Figure 3.9 'Typical' case predicted local flow patterns, peak flood flows. (left: no wind farm, right: wind farm in 5m model of NHD)

3.3.b Changes to the wave regime

Far-field wave results

The modification to the regional wave climate has been examined by considering a range of scenarios with and without the offshore development. A common transect line has been used to sample model output in a consistent way, positioned at a distance of 1km from the coast. The results along this transect have then been examined to provide an assessment of the potential changes in the wave regime relative to the coast.

Figure 3.10 provides an illustration of the reduction in wave heights as a consequence of the wind farm for a direct wave approach. The wind farm has a clear influence in reducing wave heights, but the level of reductions are small, and in the order of 1 to 1.5%.

Tables 3.2 and 3.3 present the predicted average reductions in wave height along the transect for the 'reasonable worst case', for 50% and 10% exceedence wave conditions, respectively. Included in these tables are values for the change in wave directions brought about by the offshore development. A positive value denotes a clockwise rotation of the wave direction, whereas a negative value denotes an anti-clockwise rotation of the wave direction.

Table 3.2 Reduction in predicted wave heights for 'reasonable worst case', 50% wave exceedence values

Input Wave Height (m)	Tidal State	Input Wave direction (degrees)	Average Reduction in Wave Height (m)	Changes to Wave Direction relative to input (degrees) max & min
1.0	LW	-30	0.0107	1.7 -1.0
1.0	LW	0	0.0103	1.0 -1.2
1.0	LW	+30	0.0106	0.5 -1.0
1.0	MID	-30	0.0125	1.7 -1.0
1.0	MID	0	0.0158	0.6 -0.5
1.0	MID	+30	0.0148	0.3 -0.4
1.0	HW	-30	0.0031	0.1 -0.1
1.0	HW	0	0.0036	0.1 -0.1
1.0	HW	+30	0.0034	0.1 -0.1

Table 3.3 Reduction in predicted wave heights for 'reasonable worst case', 10% wave exceedence values

Input Wave Height (m)	Tidal State	Input Wave direction (degrees)	Average Reduction in Wave Height (m)	Changes to Wave Direction relative to input (degrees) max & min
2.5	LW	-30	0.0021	0.2 -0.2
2.5	LW	0	0.0020	0.2 -0.2
2.5	LW	+30	0.0020	0.1 -0.2
2.5	MID	-30	0.0129	0.2 -0.3
2.5	MID	0	0.0120	0.3 -0.2
2.5	MID	+30	0.0133	0.2 -0.2
2.5	HW	-30	0.0081	0.1 -0.1
2.5	HW	0	0.0099	0.1 -0.1
2.5	HW	+30	0.0092	0.1 -0.1

The outcome of this analysis is that the 'reasonable worst case' scenario has the effect of reducing the wave height by 0.0158m (1.58%) for the 50% wave exceedence condition and 0.0133m (0.53%) for the 10% wave exceedence condition. The largest variations in wave direction occur at low water and mid-tide with values of <2º, with the majority of changes less than 1º. It is important to note that the levels of change determined by this analysis are beyond the accuracy limits of any monitoring equipment. It is also suggested that these changes are not significant by themselves.

Figure 3.11 provides a prediction of reduction in wave height for a comparable event to Figure 3.10, but for the 'typical' scenario. For this arrangement, the wave approach is aligned directly with the array and produces a more discernible level of interaction.

Tables 3.4 and 3.5 present predicted average reductions in wave height along the transect for the offshore 'typical' scenario, for 50% and 10% exceedence wave conditions, respectively.

Table 3.4 Reduction in predicted wave heights for 'typical' scenario, 50% wave exceedence values

Input Wave Height (m)	Tidal State	Input Wave direction (degrees)	Average Reduction in Wave Height (m)	Changes to Wave Direction relative to input (degrees) max & min
1.0	LW	-30	0.0015	0.2 -0.6
1.0	LW	0	0.0011	0.4 -0.4
1.0	MID	+30	0.0029	0.2 -0.7
1.0	MID	-30	0.0031	0.2 -0.2
1.0	HW	0	0.0008	0.0 -0.1
1.0	HW	+30	0.0008	0.1 -0.1

Table 3.5 Reduction in predicted wave heights for 'typical' scenario, 10% wave exceedence values

Input Wave Height (m)	Tidal State	Input Wave direction (degrees)	Average Reduction in Wave Height (m)	Changes to Wave Direction relative to input (degrees) max & min
2.5	LW	-30	0.0001	0.1 -0.1
2.5	LW	0	0.0001	0.1 -0.1
2.5	MID	+30	0.0021	0.1 -0.3
2.5	MID	-30	0.0015	0.2 -0.2
2.5	HW	0	0.0022	0.1 -0.2
2.5	HW	+30	0.0022	0.1 -0.1

The study hypothesis is based on the premise that if the 'reasonable worst case' has no significant impact on coastal processes, then the 'typical' scenario will have even less impact. The results presented in Table 3.4 suggest that waves are reduced by only 0.0031m (0.31%) for the 50% wave exceedence, which is a lower level than the equivalent values from the 'reasonable worst case'. Similarly, results from the 10% wave exceedence runs illustrate a reduction of only 0.0022m (0.09%). For wave direction also, the changes are even smaller than those calculated for the 'reasonable worst case' scenario.

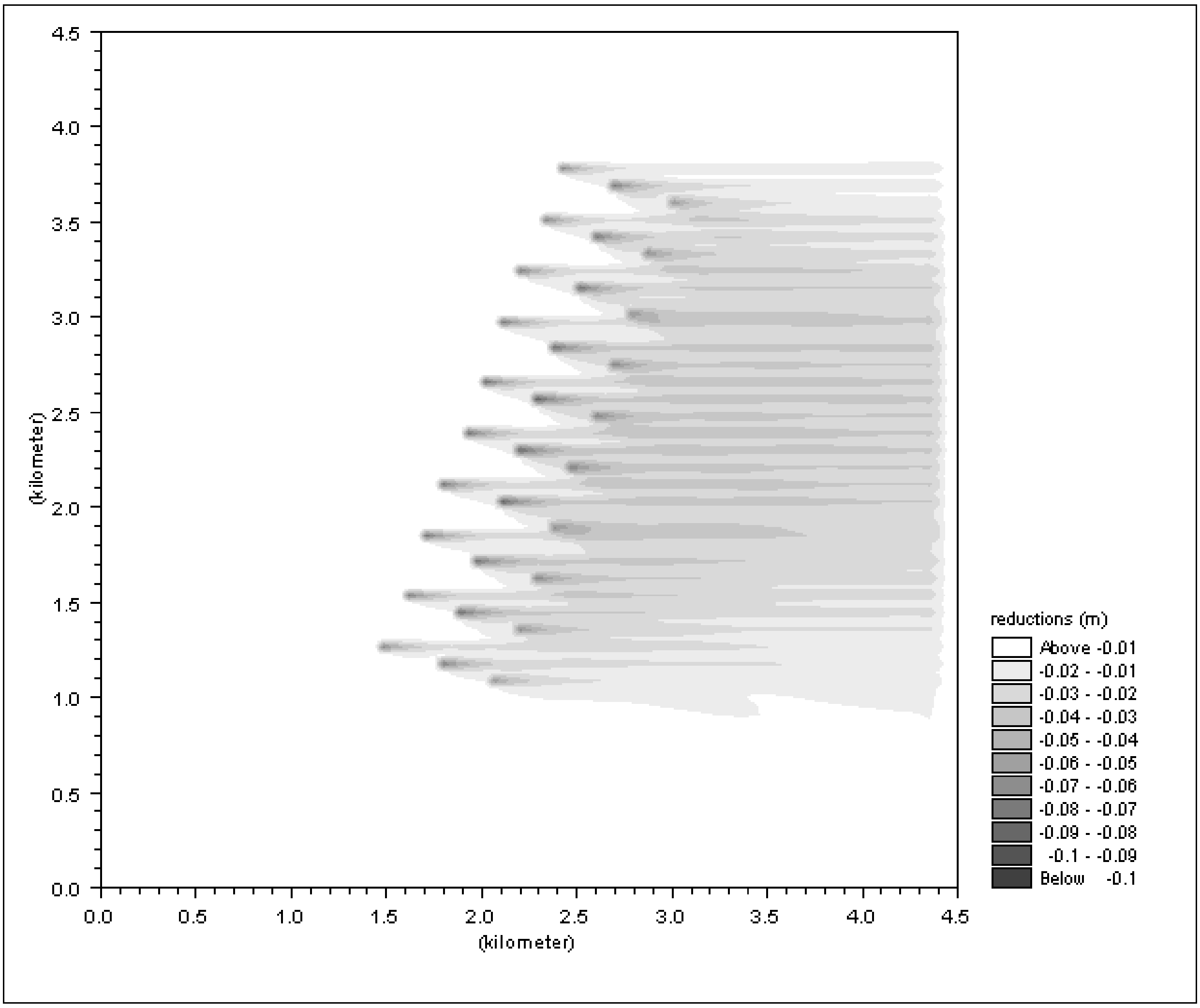


Figure 3.10 Predicted reductions in wave heights behind the 'reasonable worst case' wind farm for an incoming significant wave height of 1m and mean period of 5s. Water level is 5m ODN and the wave field enters the model from due left.

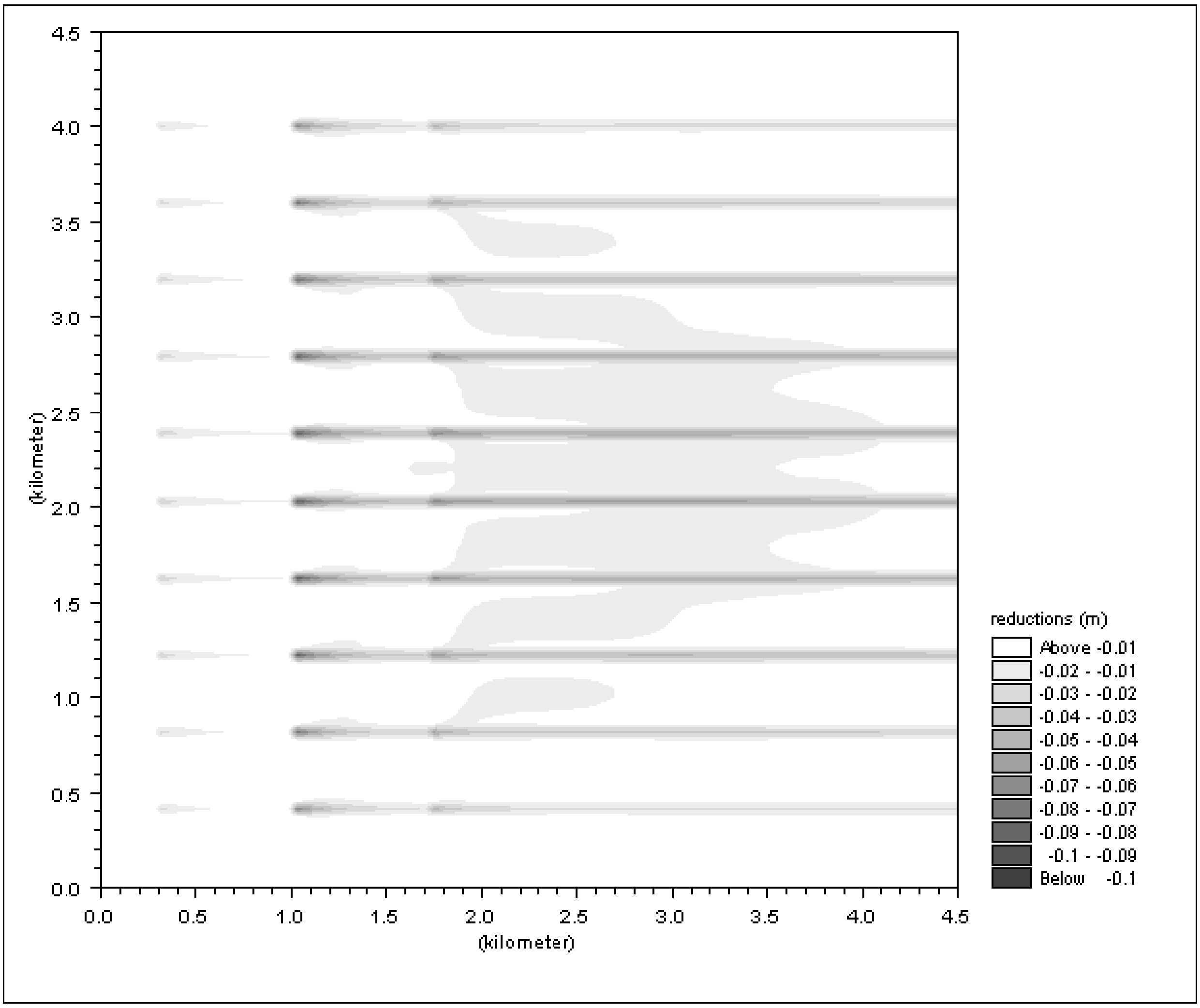


Figure 3.11 Predicted reductions in wave heights behind the ‘typical’ wind farm for an incoming significant wave height of 1m and mean period of 5s. Water level is 5m ODN and the wave field enters the model from due left.

Near-field wave results

MIKE21 BW has provided a prediction of wave heights adjacent to the support structures in a wind farm array.

Figure 3.12 illustrates the pattern of wave heights predicted by the 5m grid model for the 'reasonable worst case'. The water level is +1m ODN, providing on top of the sandbank, a depth of at least 5m. The wave train is represented by a random time series characterised by a TMA spectrum, having a peak period of 6.5s. This peak period corresponds with a mean period of approximately 5s. The wave train enters the model from the left and the contours show the amplification of the incoming wave height, which has an incident 1m significant wave height.

Towards the top of the figure, deeper water off the oblique sandbank provides an area where wave heights are greater than 1m, this is partly as a function of wave reflection off the sandbank into deeper water and partly due to a small contribution of wave reflection off the structures. Across the sandbank the shallower depths provide areas where waves energy is dissipated through bottom friction, and waves heights are subsequently reduced.

The largest amplification ratio is 1.84, adjacent to one of the structures at the top end of the model, where the water depth for this example was 5.11m. This amplification is larger than that applying to a single cylinder in a regular wave train with similar characteristics, suggesting that in the array of support structures, some interaction between reflections from several structures probably takes place. The idea that interactions between the structures does occur is corroborated by the corresponding amplification of just over two for a regular wave train rather than a random time series, applied to the full array of structures at 300m centres.

For the 'typical' case wind farm, it would be reasonable to suppose that the wave height amplification would be lower than that for the inshore case. This is because the structures are more widely separated in the offshore generic design than they are for the inshore scenario. There are three rows of structures at 400m centres in each row, with the rows 700m apart. The same random time series that was applied to the inshore array was again used as an input to the offshore array. The resulting maximum amplification of wave heights due to reflections was 1.58, rather lower than for the inshore array. The initial premise concerning lower wave height amplifications was therefore validated. Figure 3.13 shows the pattern of wave heights predicted by the grid model. The tendency of the structures to scatter the waves individually, without interacting, can clearly be seen.

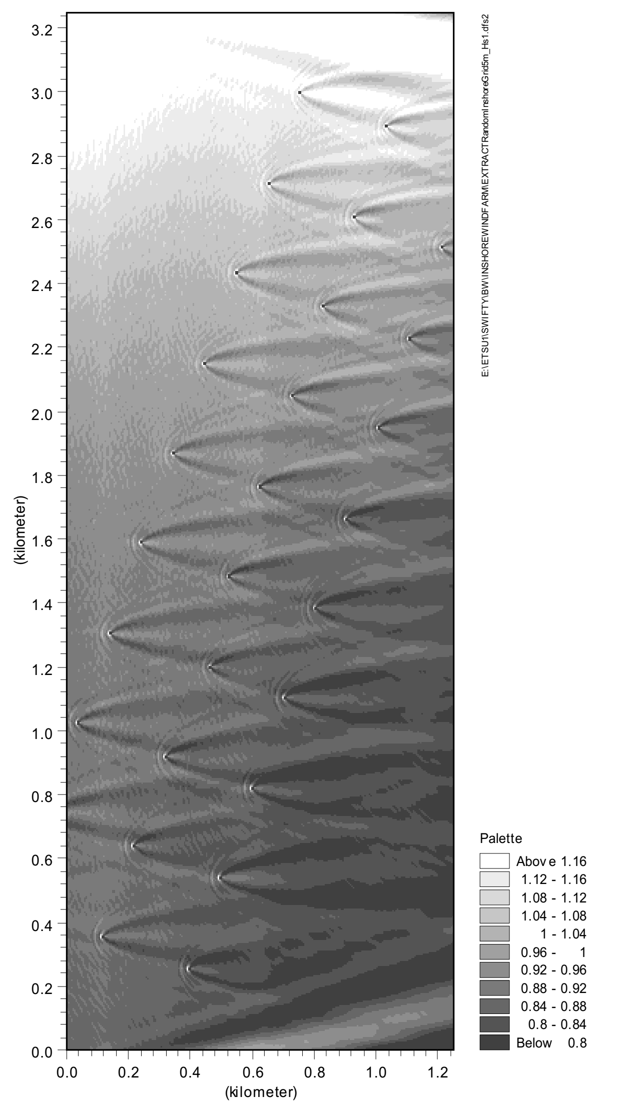


Figure 3.12 Predicted wave heights for the 'reasonable worst case' scenario. Incoming significant wave height is 1m and the peak period is 6.5s.

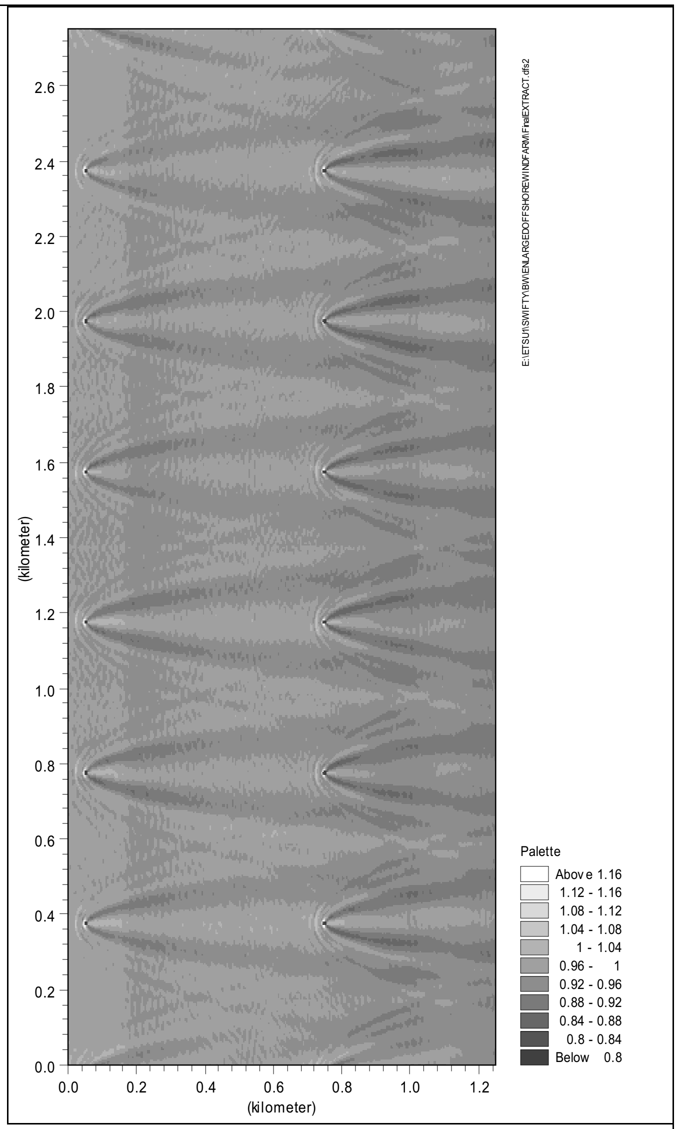


Figure 3.13 Predicted wave heights for the 'typical' scenario. Incoming significant wave height is 1m and the peak period is 6.5s.

3.3.c Changes to the sediment regime

Combined near-field and far-field sediment results

The changes in tidal and wave regimes predicted by the modelling suggest some local and minor modifications to each process but limited to the near-field. There is no overall change in the regional behaviour or significant effects remote from the development site. Hence, the implications for significant changes along the coastline are not anticipated.

To investigate how these minor changes might effect the net movement of sediments across the site a sediment particle model has been applied. This model predicts the transport pathway for medium sand particles in response to pre- and post-development predicted flow regimes and combined across the near and far field grids.

Figure 3.14 illustrates the pattern in net deposition in plan and in section for a series of events with and without the wind farm for the 'reasonable worst case' using a hypothetical sediment pathway which passes directly across the wind farm area. The results pre- and post-development appear very similar. Examining the deposition area adjacent to the wind farm indicates that the position of maximum concentration changes by around 300m, with a change in peak concentration of around 3%, but the overall position of the depositional area is unchanged. In section, the overall concentration of net sediment deposition remains relatively unchanged, however small variations are evident across Transect 1 where some deposition is displaced over small distances.

Figure 3.15 illustrates the pattern in net deposition for a series of events with and without the wind farm for the 'typical' scenario and applying the same hypothetical pathway used for the 'reasonable worst case'. The results with and without the wind farm are again very similar, with the position of the maximum concentration changing by only 45m and the peak concentration by 1.4%. Again, the overall depositional area is unchanged.

In natural systems there is usually frequent and relatively continuous change going on over the surface of a sandbank, such as the change of shape, position and orientation of sand waves responding to changes in water flows resulting from seasonal effects and from surges. Local accumulations appear due to these same influences and disappear, though the time scale, whilst sometimes short, are not regular.

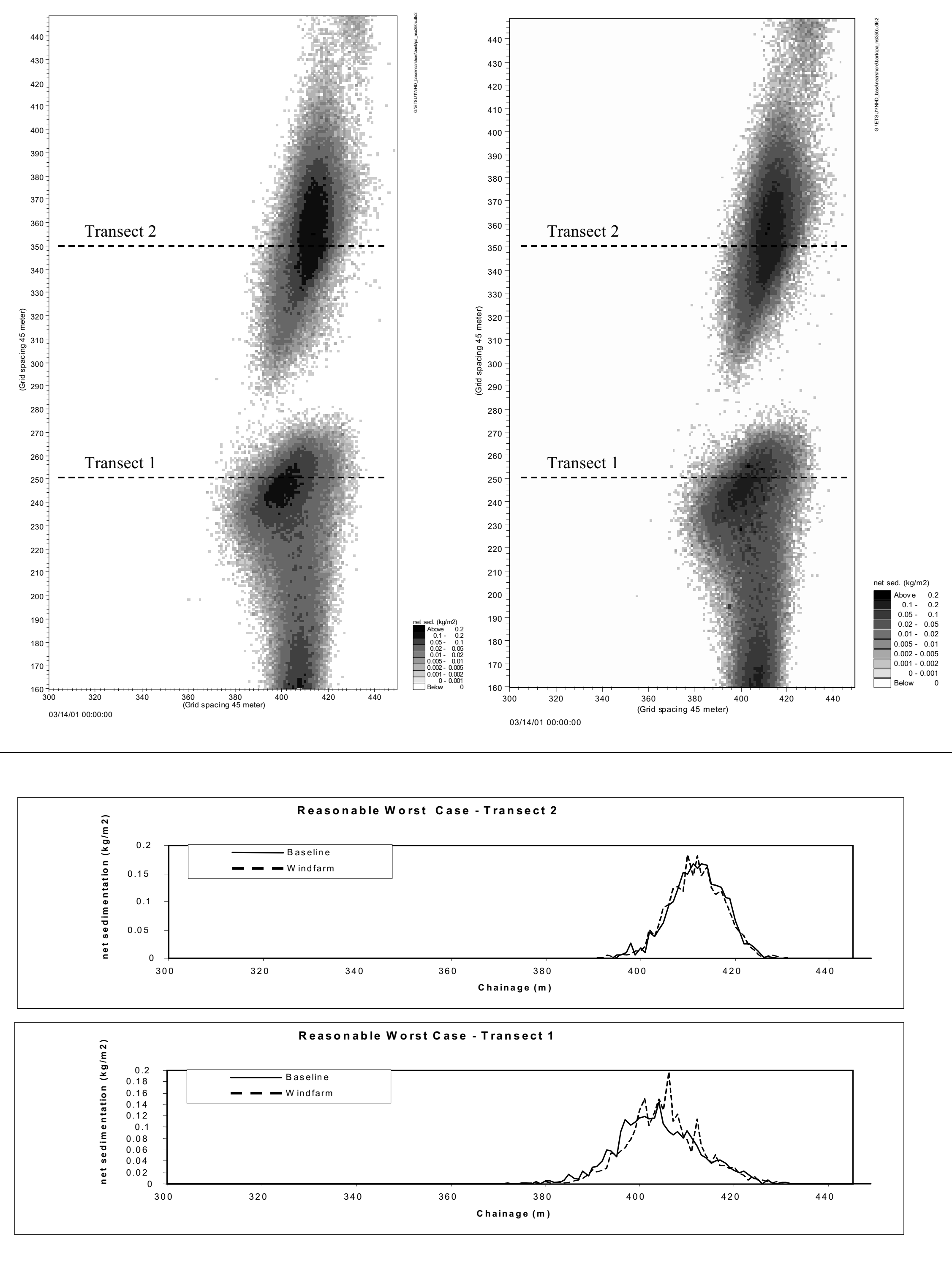


Figure 3.14 Net deposition from sediment pathway across 'reasonable worst case' scenario (top left: no wind farm, top right: wind farm, bottom: transects to compare changes)

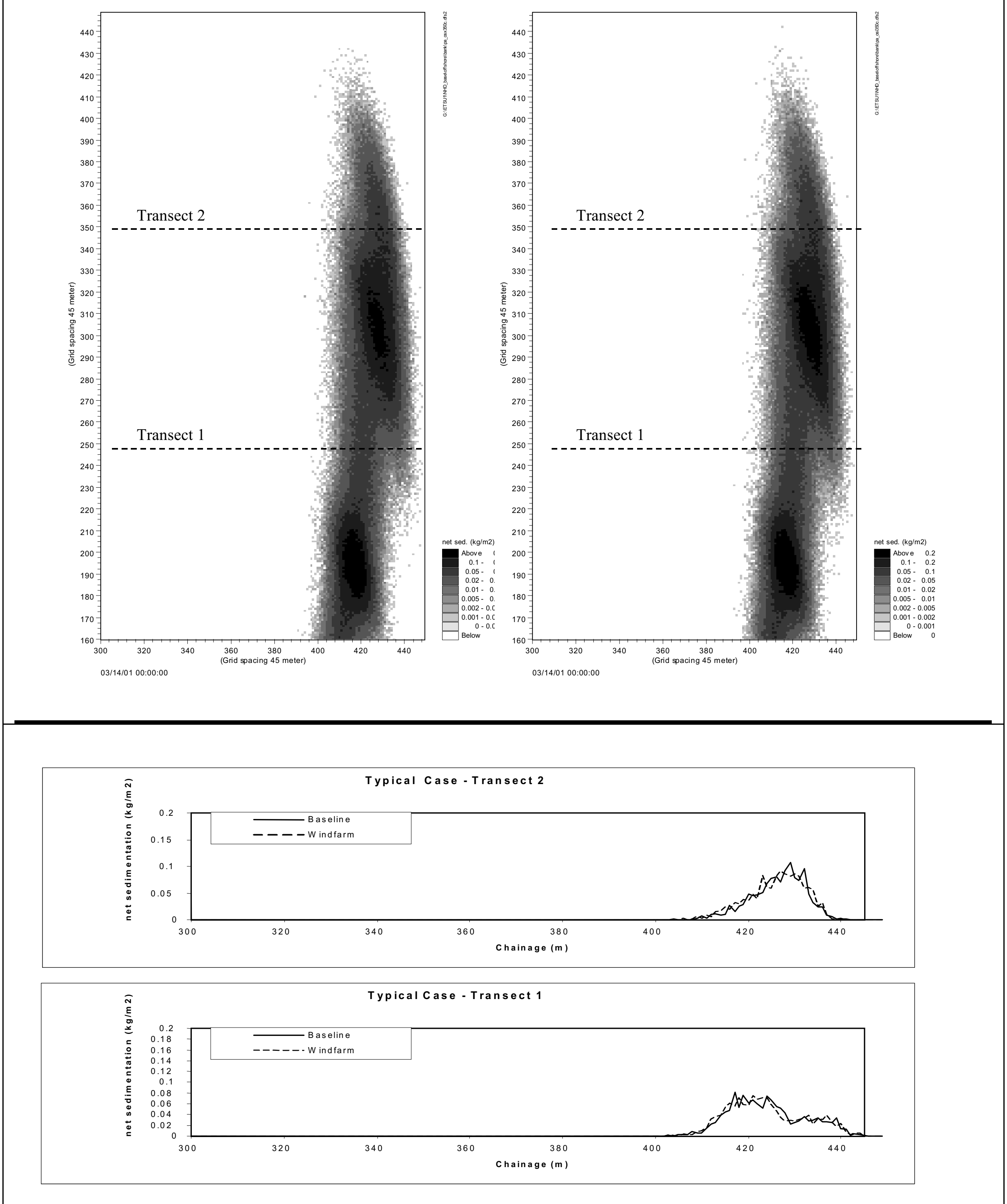


Figure 3.15 Net deposition from sediment pathway across 'typical' scenario (top left: no wind farm, top right: wind farm, bottom: transects to compare changes)

Scour

Predicting the absolute development of scour is presently beyond the capability of conventional 'coastal area' modelling schemes, such as the one described previously. Scour development is a complex process involving three-dimensional (3-D) turbulent flows and complex boundary layer interactions. In shallow water and exposed coastal locations the form of turbulence will include both tidal and wave effects. The accuracy of predictions based on conventional models is questionable, even if a 3-D model was applied. A highly detailed grid scheme would be required to resolve both the turbine support structure and the form of turbulent wakes. Such a high level of resolution which would make it impractical to resolve multiple turbines separated by long distances.

Whilst the conventional modelling approaches are unsuited to predicting scour formation they can still be considered suited to investigating general flow patterns, waves and sediment transport processes. As such they simplify the fine scale turbulent flow effects with turbulent closure schemes acting at a grid cell level. Furthermore, in conditions where scour may become an issue, it is anticipated that design considerations will address this and provide suitable scour protection for the structure. In such cases, the problem of scour and erosion of sediments adjacent to the structures is eliminated.

To assist such design requirements, a practical approach to estimating scour development is to consider the scale and shape of the structure to be placed on the sea bed, along with the predicted coastal process conditions provided by 'coastal area' models and use this information in a suitable empirical approach to estimate scour depths. Section 2.3.i provides an overview of the main issues for marine scour and Appendix G.4 provides further details on estimating scour development, along with examples from similar types of installations.

4 GUIDELINES FOR SITE SPECIFIC STUDIES

4.1 Overview

Whilst the present study has demonstrated that hypothetical 'reasonable worst case' and 'typical' scenarios appear to have no major influence on regional coastal processes there still remains the requirement for an adequate understanding of the local coastal environment for a successful wind farm development, in terms of:

- Designing a scheme that can cope with the environmental (wind, wave, tidal, sea bed) conditions (ie by establishing design parameters).
- Designing a scheme that the local environment can accommodate (ie the subject of the EIA).

Detailed design considerations will therefore need to be carried out for each of the proposed wind farm sites to determine the likely hydrodynamic loads on structures, which in turn may also influence the choice of suitable foundation types. In addition, local scouring around foundations are likely to be a main concern, and may also pose a problem for the stability of the structure. Scour protection around the base may therefore be required.

The various stages of the wind farm development will also need to be evaluated as part of the EIA through pre-construction, construction, operation and decommissioning phases. The present study has focussed on examining the potential effects related to the operation phase. In the construction and decommissioning phases, the site work on turbines and cables has the potential to cause some short-term, temporary effects on the sea bed and to raise suspended sediment levels.

A coastal process study is therefore required to describe the site specific coastal processes. An outline for such a study is provided below which is expected to include:

- initial review of site characteristics to determine the level of existing information and present knowledge, including:
  - appraisal of coastal process parameters
  - identification and evaluation of existing data and models
  - identification of data gaps
  - scope for new surveys
  - requirements for further studies and modelling
- additional data collection
- detailed investigations, with consideration of
  - baseline description (pre-development)
  - construction issues
  - operation and maintenance
  - decommissioning phase

Where modelling is used, sufficient site data is required to configure and calibrate the model to ensure predictions can be made with a good level of reliability and confidence.

CEFAS has prepared a guidance note for EIA requirements in respect of FEPA and CPA requirements (CEFAS, 2001), and refers to the scope to a coastal impact study.

The coastal process study needs to consider the issues of waves, tides and sediments in an integrated manner since these process are not independent of each other, especially in shallow water. This has important implications for any data collection programmes, particularly when the data is to be used to support site specific modelling. The main elements for such investigations are described in the following sections.

4.2 Waves

The wave conditions at each site should be investigated to support the design and operational considerations as well as the EIA studies. These requirements involve the detailed evaluation of wave statistics at the development site. The difficulty is that because wind farms are positioned in coastal areas in shallow water and often on sandbanks, sufficient measured data are not usually available.

An initial broad assessment of the characteristic 'offshore' wave conditions in UK coastal waters can be made with reference to the Department of Energy Wave Climate Atlas for the British Isles (DOE, 1991).

4.2.a Information requirements

- Design conditions

The initial stage in considering the importance of waves at the development site is to determine the exposure of each area to offshore waves.

In conventional practice, offshore structures are designed to withstand a set of 'design conditions' with a specified average recurrence interval. The recurrence interval or 'return period' is often set at 50 years, although other periods may be selected. The set of conditions will usually include the following:

- sea state, usually specified by the significant wave height, Hs.
- characteristic wave period of the design sea state, usually specified as the mean zero-crossing period, Tz.
- crest-to-trough height of the highest individual wave.
- period of the highest individual wave.
- height of the highest individual crest.
- 'still water level' (ie the combination of surge, wave set-up and tide).

- maximum surface elevation taking into account the wave crest and the still water level.

In addition, the maximum current speed including surge-induced currents will usually be required. This has a bearing on the severity of scour around the structure and indirectly influences the wave height. In fact, many of the elements of the design set are interdependent, for example: during severe conditions the local still water level is the most important determinant of wave height.

- Operational statistics

In addition to the design criteria, there is usually a need for operational statistics so that installation, operation and maintenance activities can be planned economically. For installation activities the concern would be to assess the likelihood of conditions which would prevent the operation of installation vessels and other equipment. For maintenance activities the requirement would be for accessibility statistics. That is, the proportion of the time during which the turbines are accessible for maintenance.

- EIA requirements

Sufficient data to provide a baseline description of waves propagating across the development site and towards adjacent coastlines. Post-construction analysis of wave regime to determine the potential changes in wave patterns and the consequence to sediment transport.

4.2.b Description of the local wave climate

The wave climate local to the wind farm site can be determined in a number of ways.

- From measured data

If there is a long data set of wave measurements at the site, then the problem is comparatively straightforward. The data set should ideally consist of measurements made at regular intervals of, say, 3 hours and extend over many years. From this data many of the design and operational statistics can be determined using standard techniques. Unfortunately, this is rarely the situation and other approaches have to be considered.

If short-term measurements are available in the general area of the wind farm these can be used to validate output of wave generation models.

- From modelled data

There now exist several databases for predicted offshore wave conditions (eg Met Office Wave model, NESS, etc) which have been determined from historical wind data. These databases contain wave information from the past 3 decades and constitute an enormously valuable resource. Their main benefit is that they offer complete, or almost complete, data coverage over a vast area. They also include information on wave direction which is important for the process of transforming the predicted waves to the local (wind farm) position.

The conventional approach is to select an offshore data point as near as possible to seaward of the site and to use a local wave model, which includes appropriate shallow-water processes, to transform the offshore waves to the location of the wind farm.

A problem is that offshore data are representative of deep water waves and are only available at resolution of grid points in the model. A typical resolution may be in of the order of 30km. Moreover, the coast constitutes one boundary of the model and the first points to seaward may not be wholly reliable.

- Data validation

The recommended procedure is to validate data from a possible offshore grid point against short-term measurements, where these are available, to ensure that the data are accurate. Another possibility is to undertake some measurements at the site and validate the transformed data against these. This has the advantage that the data validation is carried out at the actual site.

4.2.c Transformation of the Offshore Data Set

If the bathymetry of the site is relatively simple, then offshore waves can be transferred to the development site using standard shoaling and refraction equations, along with some procedure to allow for energy loss due to wave breaking and sea bed friction. Such a scheme is fairly rapid to apply and has the advantage that the whole offshore data set can be transformed to provide design and operational criteria from the statistics of the transformed data.

However, in general the bathymetry of the site is unlikely to be simple enough for this approach. In such cases it is advised to use a detailed gridded bathymetry to resolve such variations and as input to a wave transformation model. Appendix F provides some further details on these types of models, which need to include the following processes, as a minimum:

- Refraction and shoaling
- Wave/current interaction
- Loss of energy by wave breaking
- Loss of energy by seabed friction

This is a subset of the physical processes which affect waves in shoaling water. Other processes, such as diffraction around large features, may be necessary, especially in situations where the ratio of the pile diameter to wavelength is >20%. The results of such models will provide a regional (far-field) description of the wave regime and quantification of incident waves arriving at the development site.

Most wind farm sites (certainly those in the first round) are located in shallow water where non-linear wave processes become important. To represent these effects requires the use of a local wave model which may not be economically feasible for the whole offshore data set. Under these circumstances, the statistics of the offshore data set should be used to determine a number of design sea states which seem likely to produce the worst conditions in the wind farm area, including a number of directions of approach. The local wave model will need to include a description of the main structural elements of the offshore wind farm which stand in the path of waves, so that reflection and diffraction effects brought about by these structures is described.

The near-shore model is then applied using just these sea states. It must be remembered that with this approach operational statistics for the whole data set are not available.

4.2.d Effects of the turbine support structures on waves

This study has shown that for general wave conditions (those occurring many times each year) and for slender structures (mono-piles) the effects of the placement of the turbines is minimal. For site specific investigations this conclusion requires further validation against actual data to support the EIA and design considerations. For more extreme conditions and for larger structures (eg gravity bases) the effect on the wave climate within the wind farm due to scattering from the support structures may lead to a local patterns of standing waves which will need to be evaluated for operational statistics.

4.3 Tides

As with waves, design and operational limits brought about by tidal variations are also required, as are details of the tidal regime for the EIA. In shallow water, the tidal regime also exerts a direct influence on local waves by depth variations through the tidal cycle and by wave-current interactions.

4.3.a Information requirements

- Design conditions

For design purposes the peak tidal currents and largest water elevations are of main interest. Peak water elevations will also need to consider the contribution from non-tidal surges, which may result in either positive or negative variations to the tidal level. The dependency between peak water levels and peak waves also becomes important in determining wave loads and slamming forces.

- Operational limits

For operational and maintenance purposes, sufficient water depths and calm conditions are required for installation and access. Some first round sites are in especially shallow areas which may limit access during periods of very low tide.

- EIA requirements

A description of the tidal regime is required for pre- and post-construction phases to determine the potential changes in flow patterns and how these might effect both the wave and sediment regime.

4.3.b Description of the local tidal regime

As with waves, the tidal regime local to the wind farm site can be determined in a number of ways.

- directly from measured data
- from a calibrated model

Tidal variations tend to follow a regular pattern of repeating cycles with their major influence determined by the position of the sun and moon. These variations create daily high and low water events and periods of spring and neap ranges.

Most useful measured data sets will cover the full range in tidal variation and be of suitable quality to enable harmonic analysis to be performed. If sufficient data exist to quantify the spatial and temporal variation in tides across the development site then design parameters can be determined directly.

The spatial variation in water elevations over short distances can be expected to be minimal, but the temporal variations will show changes over daily, monthly and annual cycles. To determine design extreme water levels normally requires a succession of peak annual tides.

The horizontal spatial variation in currents is likely to be high in areas where the profile of the sea bed is irregular, especially where major bedform features are present, such as sandbanks. Vertical variations in flow tend to respond to sea bed boundary friction, surface wind stresses, wave orbital motion and, in deeper locations, stratification.

A baseline description of the tidal regime will need to consider all naturally occurring variations likely to be present in the planned operational period of the development, as well as considering how the tidal regime might respond to climate change and sea level rise.

4.3.c Effects of the turbine support structures on tides

This study has shown that for the range of tidal conditions representative for the First Round site the effects of the placement of the turbines on regional flow patterns is minimal. For site specific investigations this conclusion requires further validation against local data and performed using suitable modelling methods to support the EIA and design considerations.

4.4 Sediment Regime

The sediment regime is characterised by the mobile material which is transported through the marine environment in response to waves and tides. The mobile material may either move as bedload and form bedform features or be transported in the water column as a suspended load. The particular mode of transport can be related to the amount of energy in the water column and the properties of the local sediment.

4.4.a Information requirements

- Design considerations

For design purposes knowledge on the stability of the local sea bed is required to determine foundation requirements and give consideration to the potential for scour around the base of structures.

- EIA requirements

For the EIA, information on changes to the sediment regime is required in terms of any modification of sediment pathways, increases to suspended sediment concentrations, changes in erosion and deposition patterns and the consequence of the development on nearshore sediment transport.

4.4.b Description of the local sediment regime

A description of the sediment regime can be formed with reference to both:

- site measurements
- calibrated sediment transport model

The geology and sediment types for each site will need to be investigated thoroughly, as conditions of sediment type, thickness and mobility and underlying bedrock conditions will vary appreciably from site to site. Such data will assist decisions on methods for cable installation and any protective works, and, in combination with hydrographic data, enable quantification of localised sediment disturbance and subsequent transport of any sediment plume.

Site investigations will need to consider the usual issues related to offshore developments, including geotechnical studies from borehole and grab samples to determine local soil conditions, type and thickness. In addition, a variety of geophysical methods need to be used, including reflection seismic profiling to ascertain both local and a more regional view of sediment type and thickness. There will be a need to obtain an accurate baseline bathymetry for the area and the use of side-scan sonar to help in the identification of mobile bedforms. If it is suspected that there is appreciable mobility then there might be a need for a repeat series of bathymetric and side-scan surveys to attempt to measure this mobility particularly where there are large bedforms that, by their mobility, cause large variations in local water depth.

A baseline of suspended sediment concentrations is also required, along with quantification of the sediment type taken into suspension by increased turbulence around the structures.

Any sediment transport model needs to consider the combined influence of waves and tides on sediment transport.

A baseline description of the sediment regime will need to consider all naturally occurring variations likely to be present in the planned operational period of the development, as well as considering how the sediment regime might respond to climate change and sea level rise effects.

4.4.c Effects of the turbine support structures on sediments

The findings of this study suggest that, in the short-term, there is likely to be little disruption to the overall sediment regime as a consequence of the offshore development, except for the potential for local scour effects. In most cases, scour protection is anticipated as part of the design around the base of any structures which will eliminate scour problems.

The results of the present investigations and from an understanding of the causes and scale of sandbank dynamics, it is considered unlikely that long-term, major geomorphological change will occur as a result of turbine structures, though a consideration of scales is an important issue. For example, if a small feature such as a sandwave is seen as a geomorphological feature then these are likely to change in response to natural conditions and be locally modified due to scour as they pass through and by a structure.

4.5 Marine Cables

There are a number of methods currently available for protection against scour around cables laid across the sea bed. These include protective aprons, rock dumping, mattresses, cable trenching, concrete saddles, cable anchors, and flow energy reduction devices.

However, it is anticipated that, for the majority of cases, the main export cable will be buried to nominal depth below sea bed of around 1 to 2m to ensure adequate protection from fishing activity, anchoring and possible erosion. Burial depth specification requires a risk assessment as part of the engineering design process. Embedment is likely to be carried out by ploughing or jetting. At the shore ends, and in areas of very hard sea bed including rock, it may be necessary to trench the sea bed.

Ploughing involves cutting the sea bed with a plough towed by a vessel. As the sea bed is cut, the cable drops in above the plough cut and the sea bed then falls back into place. Thus, the plough both buries and back-fills, giving instant cover and protection. However, it is difficult to use a plough in areas of hard sea bed such as boulder clay.

Jetting tools, which can be free-swimming or tracked, cut a trench into which the cable is laid. Tidal current movement then causes backfill with sediment over a period of time. Jetting tools can be used in areas of hard sea bed, including some soft rocks. If a cable has to be installed through an area of rock, a trench in the rock may be cut and the rock back-filled.

4.5.a Potential Effects of Installation

Ploughing and jetting would cause local, short-term, temporary disturbance of the sea bed. With ploughing, the sea bed settles back in place on top of the cable as it is laid. The depression formed by jetting will often naturally backfill with sediments moving in under the influence of tidal currents though monitoring will be necessary to ascertain its effectiveness. Typically, a corridor of up to 5m width of the sea bed may be disturbed during installation of the cable. The width of disturbance is dependent on the depth of burial, the size of the plough 'footprint', and the installation technique. If natural burial is slow or unlikely to occur then the jetted trench will need to be filled with appropriate non-erodable materials.

Some effects may arise during installation from increased suspended sediment in the water column. For example, jetting may distribute significant volumes of sediment into the water column. Heavy particles will settle down quickly over the immediate surrounding area, but finer particles will travel further from the disturbed area, swept by tidal currents. Therefore, indirect impacts could extend beyond the 5m installation corridor. However, assuming appropriate ploughing or jetting techniques are used, any effects should be short-term and relatively small, resulting in little impact on coastal processes. Suspended sediment plumes can cause small, localised increases in turbidity and oxygen demand in the water column. Resettlement of particulate matter could cause short-term alterations to the physical characteristics of the sea bed, but recovery of the original sea bed geomorphology is usually relatively rapid, especially in areas characterised by relatively strong tidal current conditions.

In many cases, suspended sediment levels can be expected to be already high due to the current regimes, storm activity, areas of mobile surface sediment and fisheries activities, particularly beam trawling, along the cable route. Therefore, the increase in suspended sediments above background levels will be short-term and probably not significant, but this will need to be confirmed as part of the EIA.

In those instances, in particular when jetting is employed, when displaced sediment is likely to become a substantial part of the suspension sediment load and be dispersed down tide, the potential near-field and far-field concentrations will need to be ascertained in relation to the normal background suspended sediment concentrations(SSC) as part of this EIA. This will allow judgements to be made as to any short or long-term effect upon the morphology or benthos in the suspension corridor.

Trenching and cable laying in harder substrates is more difficult, and the potential effects on the sea bed morphology are generally greater, than for jetting or ploughing. The sediment types in which a cutting wheel is normally used (ie hard substrates) may take some time to naturally infill, causing longer-term, but localised disturbance to the sea bed.

Where rock scour cable protection is required, the effects upon sediment transport in the local area should also be considered to ensure that the rock does not adversely affect the local rate of coastal erosion/accretion.

- Pipeline or Cable Crossing

The export cable route may involve crossing an existing pipeline or another cable. Post-lay jetting for cable burial at pipeline/cable crossings can result in a potential impact in terms of temporary and localised re-suspension of sediment and disturbance to the sea bed morphology. Where post-lay burial cannot be undertaken, rock armouring may be used or concrete mattresses may be installed over the cable crossing to ensure protection. This could raise the sea bed profile by approximately 0.25 to 0.5m at the location of the crossing, which may encourage limited accumulation of sediment at the crossing. Thus, the impact on sea bed topography, although long-term, can be expected to be small and very localised.

- Shore-end/Landfall

In most cases, the cable would be trenched across the foreshore and beach. Following installation of the cable, the trenches would be back-filled with the removed beach materials (eg sand, shingle, cobbles), with care taken to restore material to previous profiles where differences have been found during excavation.

Given the dimension of the cable, and assuming a well-planned and professionally executed installation operation, there should not be any significant effects on the beach profile and mobility of coastal sediments.

4.5.b Operation and Maintenance

Once buried, the cable is not expected to have any significant impact on sediments or sea bed morphology. As mentioned above, mattresses installed at pipeline/cable crossings would cause a localised, long-term impact on sea bed topography, and may also result in some sediment accumulation but this impact is not expected to be significant.

Maintenance of the cable is not anticipated. However, if a fault occurred which necessitated repair, the cable would have to be excavated, repaired and re-buried. Potential impacts would be similar, but on a much smaller scale, to those during installation.

4.5.c Decommissioning

The typical quoted design life of a submarine cable is 25 years, but in practice cables may remain in use for a period of many years after their original design life has expired.

Once buried, the sea bed above the cable should settle and signs of disturbance should be difficult to distinguish. Following decommissioning, in principle the cable could be secured at either end and abandoned *in situ*, with its location remaining marked on charts as an obstacle. Alternatively, when no longer operational, the cable could be recovered to remove it as a potential obstacle and to realise any remaining value. Removal, probably by dragging or excavation, would cause disturbance to the sea bed. However, as with cable installation, impacts are likely to be local and short-term, but this will need to be confirmed on a case-by-case basis.

5 CONCLUSIONS & RECOMMENDATIONS

5.1 General

The study has carefully considered the potential effects on coastal processes likely to be caused by the installation of an offshore wind farm development and associated infrastructure, including cabling to shore. This has been done in the context of first-round sites around the UK coast announced by the Crown Estate on April 2001, although it is also considered that the general findings of the present study will be more widely applicable to other offshore environments. The properties of these first round wind farms and their respective coastal environments are reviewed in Section 2.2 & 2.4 and Appendix E. This information has provided the context for the generic studies.

As yet there are no large-scale offshore wind farms in similar environments anywhere in the world, although some are planned for installation in 2002 and beyond. This means that there is, as yet, no project experience to draw on.

The present study has therefore developed a definition for 'reasonable worst case' and 'typical' wind farm scenarios and has assessed the potential effects these may have on coastal processes, primarily through the application of existing computer models, and with reference to theoretical considerations.

The following parameters define the "reasonable worst case" and "typical case" scenarios which have been established from a review of the first round development proposals.

Parameter	**Reasonable Worst Case**	**Typical Case**
Morphology of development site	Nearshore sandbank, angled to shore	Offshore Sandbank, parallel to shore
Distance from shore	1.5km	7.0km
Size of wind farm	30 turbines in $10km^2$	
Foundation type	Mono-pile, 5m diameter	
Array Spacing	3 rows of 10, each separated by 300m	3 rows of 10, 700m between rows, 400m spacing within a row
Tidal Conditions	Macro tidal environment, 6m tidal range	
Wave Inputs	10% wave exceedance (Hs = 2.5m, Tz = 7s) 50% wave exceedance (Hs = 1.0m, Tz = 5s) approach angles -30, 0 & 30° normal to coast	
Sediment Environment	Sandy substrate	

From the scenarios investigated in the present research, it is concluded that the changes in current, wave and sediment conditions brought about by the presence of the wind farm are unlikely to be significant in the far-field, with only very small influences determined in the near-field.

5.2 Applicability of Existing Computer Models

The work has involved a brief review of existing wave, current and sediment models available for use in coastal impact studies, such as for marine aggregate extraction and engineering design. It is concluded that selected, established models can be adapted and applied with confidence to the assessment of the interaction between an offshore wind farm and the coastal processes.

However, these same models do not provide a direct means to predict scour development due to various simplifications in their numerical schemes, eg turbulence closure assumptions. Indirectly, these models can provide appropriate input parameters into a well-chosen empirical relationship to predict scour development.

From the range of applicable models considered, the study adopted the use of the DHI MIKE21 coastal modelling system to provide an integrated description of tidal, wave and sediment transport processes. Similar systems are available from other suppliers (Delft3d, Telemac, etc) and the project makes no claims solely to endorse MIKE21. The present study uses nested model grids with increasing resolution ranging from 45m down to 5m to capture the conditions around (far-field) and inside (near-field) the offshore wind farm development.

The potential effects of an offshore wind farm on coastal processes have then been evaluated in relation to a representative set of tide, wave and sediment transport conditions. This involved a suite of model runs with and without the offshore wind development in place.

5.3 Reasonable Worst Case

The 'reasonable worst case' scenario is defined with a site located on a nearshore sandbank around 1.5km from the coast, with an array of 30 turbines in 3 rows of 10 and with a constant spacing of 300m. It is intended that this arrangement creates the opportunity for greatest potential effect on coastal processes that may be brought about by any of the planned offshore wind developments.

From the test performed, the modelling results show that the changes in current, wave and sediment conditions brought about by the presence of the wind farm are unlikely to be significant in the far-field, with only very small influences determined in the near-field.

For example, the changes in tidal current predicted to occur along a nearshore transect line are less than 1% in speed and 0.5° in direction. Similarly, wave heights were found to reduce by around 0.5 to 1.5% under typical conditions (10% and 50% exceedence levels). The largest variations in wave direction, occurring between low water and mid-tide, were of order 1 to 2% or less.

Changes inside the wind farm are likely to be of some importance in relation to local scour, with some discernible interaction apparent as a result of the close spacing of the wind farm array. For example, changes in current speed in the lee of turbine structures have been estimated to be less than 0.1m/s at peak flows of 1.2m/s. Incoming wave conditions show some amplification immediately adjacent to the turbine structures due to reflection effects. For the ‘reasonable worst case’, the modelling also suggests some interaction between reflections from several structures, but their magnitude is small.

Sediment pathways do not appear to be changed significantly as a result of placing the wind farm in their way. The model results showed that the overall sediment depositional area effectively remained unchanged, although the position of maximum concentration appeared to have moved by several hundred metres, with a change in peak concentration of around 3%.

Whilst the research has shown that the impacts on the regional geomorphology are likely to be negligible, the physical environmental impacts on the scheme, in terms of current and wave loading, are likely to be more severe. In addition, problems associated with locally generated scour around the foundations of turbines will need to be dealt with, probably involving appropriate protection measures.

5.4 Typical Scenario

The ‘typical’ scenario is defined with a site located on a sandbank around 7km from the coast, with an array of 30 turbines in 3 rows of 10 and with a separation of 700m between rows and 400m within a row. This arrangement is considered to be representative of the majority of sites in the first round.

The changes in current, wave and sediment conditions brought about by the presence of the wind farm are even smaller at a regional level than for the ‘reasonable worst case’.

From the tests performed, the changes in tidal current predicted to occur along the near-shore transect are far less than 1% in speed and 0.5° in direction. Changes in wave height were found to be of order 0.5% or less and changes in wave direction were similarly small. Changes in the immediate area of the wind farms are only likely to be of importance in relation to local scour.

Model results show that sediment pathways do not appear to be changed significantly, being even less than for the ‘reasonable worst case’.

Furthermore, for both scenarios, the level of changes that have been determined for each coastal process parameter is probably at the limits of accuracy of conventional monitoring equipment. In terms of wave measurements, detecting small changes presents a challenging problem for a variable that is derived from statistical methods.

5.5 Cables

The installation of submarine cables across an offshore wind farm and to shore has the potential to cause some short-term, temporary effects on the seabed and on suspended sediment levels. However, the impacts are unlikely to be significant provided the cable lay operations are carried out professionally so as to minimise disturbance. Pipeline/cable crossing may result in very localised accumulations of sediment, but the impact is unlikely to be significant. These issues will need to be confirmed on a case-by-case basis as part of the EIA. This will involve desk review but in some cases may also require well-specified computer modelling.

5.6 Previous and Proposed Offshore Wind Farm Studies

A number of site specific studies have been carried out in recent years in support of offshore wind farm development projects and consider the issue of coastal processes, for example in Denmark (eg Middelgrunden and Horns Rev) and UK (eg Blyth and Scroby). Conclusions were broadly similar to those of the present study. These studies also suggest some minor influences on the current, wave and sediment conditions immediately around the turbine structures, but no significant impacts on the regional geomorphology.

Model calculations carried out for Horns Rev (up to 80 turbines, with mono-pile foundations) suggest that local currents in the vicinity of the turbines may be reduced by at most 2% after installation of the wind farm.

The newly proposed development off the Dutch coast at Egmond aan Zee is another demonstration offshore wind farm project, to be located some 8 km offshore. As part of this development, there will an extensive monitoring and evaluation programme. This will include, among other things, regular measurement over a 3-year period of the 3-D erosion pattern around mono-piles and an assessment of the effectiveness of seabed protection techniques. The monitoring programme will also look at potential changes further afield in currents, suspended sediment and geomorphology. Such data will be invaluable in developing a greater understanding of ‘actual’ coastal process effects resulting from an offshore wind farm development. It is therefore recommended that, when such schemes are built and monitoring commences, this new information is reviewed in the context of the present study to complement the conclusions of these investigations.

5.7 Site-Specific Studies

The present study has addressed idealised wind farm scenarios in order to identify the key issues and assess potential impacts in a generic way. In a site-specific situation, the local character of current, wave and sediment conditions, in combination with the specific wind farm location, dimension and orientation plus the turbine array and support structure, will need to be evaluated to predict the actual significance of the effects on coastal processes.

Whilst the research has shown that the potential impacts on the environment are likely to be negligible, the environmental impacts on the scheme, in terms of wave and current loading, are likely to be far more severe. In addition, the problems associated with locally generated scour (around the turbine structures) will need to be dealt with adequately. To investigate these issues will involve a combination of desk-study, field survey and computer modelling work. Chapter 4 provides general guidelines for undertaking such site-specific studies.

5.8 Recommendations for Further Research

5.8.a Endorsement of Study Conclusions

The present study has reviewed, at a generic level, the potential effects of offshore wind developments on coastal process. The research has drawn on experience from other coastal applications as well as referring to existing theoretical considerations for a number of key issues, including:

- Representation of structural resistance in tidal currents
- Representation of structural resistance in waves
- Wave scattering and diffraction effects resulting from cylindrical structures placed in a wave environment
- Scour development

As yet there are no large-scale offshore wind farms in similar coastal environments anywhere in the world to provide supporting evidence to fully endorse the findings of the present research, although some are planned for installation in 2002 and beyond. It is therefore recommended, that when such schemes are built and monitoring commences, the resulting information is reviewed in the context of the present study to further validate the conclusions of these investigations.

For example, scour effects in a mobile sediment environment remains as an issue where scientific understanding is still developing. At present there are no universally recognised methods to assess scour, either inclusive or exclusive of standard coastal process models. Observations of scour are therefore of extreme interest around any offshore installation and such monitoring will prove invaluable in developing techniques for quantifying this effect and for assisting the selection of appropriate scour protection methods.

5.8.b Future Direction of Offshore Wind Industry

The present study has focused on the issues related to the first round of offshore developments and generalised the main coastal process parameters to define hypothetical scenarios for the two test cases. If the offshore wind industry moves towards deeper water and expands into larger schemes then it is recommended that the present research is extended and adapted to encompass any new issues.

6 REFERENCES

Caston, G.F. 1981. Potential loss and gain of sandbanks in the southern Bight of the North Sea. Marine Geol. 41. 239-250.

CEFAS, 2001. Offshore Wind Farms: Guidance note for Environmental Impact Assessment in respect of FEPA and CPA requirements. October 2001.

DETR, 2001. Draft Marine Minerals Guidance Note 2: Guidance on the Extraction by Dredging of Sand, Gravel and Other Minerals from the English Sea bed. Consultation Paper. February 2001.

DOE, 1991. Department of Energy - Offshore Technology Report. Wave Climate Atlas of the British Isles. OTH 89 303. HMSO.

DTI, 1999. New and Renewable Energy – Prospects for the 21st Century.

DTI, 2002. Guidance Notes. Offshore Wind farm Consents Process. April 2002.

Dugdale, R.E. 1980. Nearshore sandbanks and foreshore accretion on the south Lincolnshire coast. East Midlands Geog. 7(2) 50, 49-63.

ETSU, 2000. An assessment of the environmental effects of offshore wind farms. ETSU W/35/00543/REP. (prepared by Metoc plc.)

Halcrow, 1996. Wave Climate / Scour Study. Sarah Jane Offshore Wind Monitoring Station. Study Report. Powergen plc. September 1996.

IEA CADDET Centre for Renewable Energy, 2000. Electricity from Offshore Wind. Danish Energy Agency & IEA CADDET Renewable Energy Technologies Programme. October 2000.

MAFF, 2001. Proposals for Offshore Wind Farm Developments: General Guidance Notes.

McCave, I.N. 1978. Grain size trends and transport along beaches: Examples from Eastern England. Mar. Geol. 28. M43 – M51.

McCave, I.N. and Langhorne, D.N. 1982. Sandwaves and sediment transport around the end of a tidal sandbank. Sedimentology 29, 95-110.

OWEN, 1999. Offshore wind resource assessment and metocean data. EPSRC. 1 November 1999.
(http://www.owen.org.uk/workshop_2/ws2_final.pdf)

OWEN, 2000. Structure and foundations design of offshore wind installations. EPSRC. 1 March 2000. (http://www.owen.org.uk/workshop_3/ws3_final.pdf)

Venn, J.F. and D'Olier, B. 1983. Preliminary observations for a model of sandbank dynamics. In Sundermann,J. and Lenz, W. (Eds) North Sea Dynamics, Springer-Verlag, Berlin, 472-485.

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APPENDIX A CONSULTATION

A number of stakeholder groups were contacted by letter at the outset of the project to request their input to the study. These included the key consenting authorities and statutory bodies as well as European developers and first round pre-qualified lease holders.

In response to this request, input to the study was received from the following groups, either by letter, e-mail or meeting. However, it is stressed that any views and judgements expressed in this report are those of the authors.

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APPENDIX B REVIEW OF SCROBY SANDS STUDY

B.1 Background

Outside of Blyth Offshore, there has been previous interest in the UK to develop offshore sites on Scroby Sands and Gunfleet Sands. Both of these sites are now actively being pursued since the announcement made by Crown Estate on their successful site allocation. Detailed studies have already taken place to support the development on Scroby Sands which includes an investigation on wave climate and scour (Halcrow, 1996). This is an important study as it represents the first set of investigations related to a wind farm development in UK coastal waters. The location of the proposed wind farm is on Scroby Sands, a site around 4km from the coast north-east of Great Yarmouth, Norfolk. Scroby Sands is a linear bank with its longest dimension orientated approximately north to south along the axis of the tidal currents. Water depths range from 5 to 10m, depending on position and the state of the tide.

B.2 Waves

The issue of waves has been developed from consideration of an offshore (or deep water) wave climate derived from an available long-term data set. This wave data has then been transformed to the study site through the application of a refraction/diffraction model. The area modelled extended about 20km from the coast and around 30km along the coast, with a grid size of 200m. The model area includes the wind farm, but does not encompass the deep water wave position. This assumes that the wave climate at the northern, southern and eastern edges of the shallow-water model area is substantially similar to the wave climate at the location of the offshore grid points. This may be an over-simplification since the offshore depth is not 'deep water' when compared with typical sea-state wavelengths of 50 to 100m, and moreover the bathymetry between the offshore grid points and the edges of the coastal model is complex.

The transformation model was run for a number of depths corresponding to various combinations of tide and surge elevations. Wave statistics for the various water depths were assembled for two positions in the wind farm area.

A separate near-field wave study attempted to elucidate the effect an array of mono-pile wind turbines might have on the wave field in the local area. This was achieved using a local model covering an area of 1.8 by 2.2km, and resolved with a grid size of 5m. The model was run with and without an array of 50 mono-piles, represented by 5 by 5m squares, and with a monochromatic wave boundary. The 'without' case shows the effect of wave refraction over the bank, while the presence of the mono-piles results in the superposition on this basic pattern of a smaller-scale pattern of increased spatial variability resulting from the interference of waves scattered by the mono-piles with the incident wave field. For monochromatic regular waves, changes in wave heights of order 10% occur in the vicinity of the wind farm. Overall it is considered that areas of reduced activity compensate the areas of increased wave activity.

Unlike a monochromatic wave, a real sea-state consists of wave energy spread over a range of frequencies and propagation directions and is often described as random waves. The scattering and interference pattern shown by the monochromatic wave results will be considerably smoothed, whereas the spatial variability due the presence of the wind farm will in fact be indistinguishable from the natural spatial variability of the waves.

The effects of scattering should be limited to the immediate vicinity of the wind farm. The only other effect is due to blocking of wave energy which the report estimates will lead to a reduction of wave height of order 1% to the 'lee' of the wind farm.

It is noted that no similar near-field tidal flow or sediment modelling was undertaken.

Compared to presently proposed wind farm designs, the study made 'conservative' assumptions in modelling the wind farm in respect of number of turbines (50) and turbine spacing (around 250m). In addition, the turbine base diameter was represented in the model by a 5 by 5m grid square which is suggested to provide a greater impact on the current flow and wave conditions that a cylindrical shape.

B.3 Wave/current interaction

The report notes that the mono-piles will affect the waves in their immediate neighbourhood (this is assumed to mean within a few pile diameters) so that the interaction of quasi-steady currents (ie tidal, surge and wave-induced currents) with the wave orbital velocities will be enhanced. This will lead to greater scour effects.

B.4 Sandbank Morphology

Importantly the study provides some considerations of the recent morphology of the sandbank system. This is of great importance to any developer planning a scheme in such a dynamic environment. Scroby Sands is one bank amongst a system of sandbanks where rapid changes can occur in the profile of the sea bed. It is highly likely that any sandbank accumulation of sand will alter its height and lateral position in time. Often this lateral movement is one of sinuous alteration about a mean axis whilst the highest elevations of some sandbanks are found to move their position along the axis of the bank. The sandbank systems play an important role in the coastal zone as they provide a level of protection for the shoreline against waves. Any changes to the crest height, width and position of the bank will affect the amount and strength of the wave activity that reaches the shoreline.

B.5 Scour

Scour problems are reviewed and tentative estimates developed with the use of theoretical analytical relationships. It is clear that these methods give only approximate results, and further work in this area was recommended. The report comes up with a scour depth of order 8.4m for a 5m diameter mono-pile.

Around the base of the wind mast the anecdotal evidence suggests that scour around this 1m diameter pile is approximately circular with a depth in the same order as the pile diameter.

Recommendations for scour protection are made. The studies also suggest that there should be no significant interruption to sediment pathways.

B.6 References

Halcrow, 1996. Wave Climate / Scour Study. Sarah Jane Offshore Wind Monitoring Station. Study Report. Powergen plc. September 1996.

APPENDIX C EUROPEAN DEVELOPMENTS

C.1 Demonstration Projects

Real offshore developments commenced toward the end of the 1980's with three research and demonstration projects at Nogersund, Vindeby and Medemblik. It is noted that all three projects were realized under not too extreme conditions, being either in the Baltic Sea or an inland freshwater lake.

The world's first offshore site came into operation in 1991 and was a single turbine mounted on a tripod foundation and placed on a solid rock platform a distance of 250m off the Baltic coast of Sweden at Nogersund. It is understood that the scheme was later abandoned in 1998.

The world's first offshore wind farm (ie multi-turbine development) came into operation later in 1991 with a configuration of 11 turbines mounted on a sandy sea bed using box caisson foundations. This development is a distance of around 1.5km from the Danish coast, north of Vindeby. The site is still in operation.

The Lely offshore wind farm is sited in a freshwater lake near Medemblik in Holland, and became operational in 1994. The site includes 4 turbines supported on steel mono-pile foundations drilled into a sandy bed at a distance of around 800m from the shore of the lake.

The first offshore wind farm in UK coastal waters is Blyth Offshore, a site which became operational in December 2000. The development is also regarded as a demonstration project for the UK. The site comprises two turbines located on North Spit, a small rocky shoal around 1km offshore from Blyth Harbour off the coast of north-east England. The turbines are mounted on mono-pile foundations drilled into a permanently submerged rock platform.

Further developments have proceeded in Holland, Sweden and Denmark with Middelgrunden coming into operation in January 2001. At this site 20 turbines have been erected on Middelgrunden Shoal, a site 2km outside of Copenhagen harbour. Gravity foundations have been used for these turbines and arranged to follow the elliptical arc of the shoal. The site is a former dumping ground for harbour sludge. This development is presently the largest offshore wind farm in the world and has the capacity to generate enough electricity to power 3% of the Danish capital's energy needs.

For each of these existing developments appropriate design and environmental considerations have been made to ensure successful projects.

The present sites off the coast of Sweden and Denmark are all within the Baltic Sea, and those in Holland are in a freshwater lake. For all these areas the tidal influence is relatively insignificant (*Note*: the tidal range in the Baltic Sea is only a few centimetres at most) and the fetch lengths are restricted to relatively small distances. In addition, the issue of ice loading is of some relevance to the design calculations.

The Environmental Impact Assessment (EIA) for such developments have included consideration on the impact brought about by the physical presence of the wind turbines. This normally includes comment on changes to waves, flows and sediment movements.

For the Danish site at Middelgrunden, the EIA (MWTC, 2001) includes a consideration of changes in water flow brought about by the development. It is commented that *'the foundation's influence on water flow in Øresund has been analysed. The analysis shows that the water flow will be reduced by maximum 0.005%.'* For design considerations, the calculated wave parameters are a significant wave height of 3.8m and a period of 6s, providing a wavelength of around 40m.

For Blyth Offshore the tidal range is relatively large (4.6m spring tide equating to a macro tidal environment) and the fetch distances equate to the width of the North Sea. Hence, the influence from tidal currents and waves becomes more significant, whereas problems from ice do not feature as a design issue.

Table C.1 provides summary details for the known offshore wind farms across Europe which have come into operation.

C.2 Future Developments

The success of the demonstration projects has allowed serious consideration of much larger wind farm parks, at greater distances off the coast, in deeper water and in more exposed conditions.

Ireland, Belgium, Germany and the Netherlands are also expressing serious intent in developing their offshore resource. Proposed projects include:

- Mouth of the Western Scheldt River, Holland, 100MW
- Ijmuiden, Holland, 100MW
- Horns Rev, Denmark, 160MW
- Læso, Denmark, 150MW
- Omo Stalgrunde, Denmark, 150MW
- Gedser Rev, Denmark, 15MW
- Rodsand, Denmark, 600MW
- Lillgrund Bank, Sweden, 48MW
- Barsebank, Sweden, 750MW
- Kish Bank, Ireland 250MW+
- Arklow, off County Wicklow, Ireland 200MW+

Utilising megawatt-plus class machines, these projects will generate higher volumes of electricity from the more constant wind regimes experienced at sea and are likely to play a major role in power generation in the future.

C.3 Horns Rev

Horns Rev is the first Danish offshore wind farm planned for development in the North Sea, with construction due to commence in 2002. This case is perhaps the closest example yet to coastal process conditions similar to those at the proposed UK sites. The designated area for the wind farm is around 15km off Blåvands Huk, which is the most westerly point on the Danish coast, at a site south of the Horns Rev reef. Water depths across the wind farm are between 5.8 and 17.5m, with a micro-tidal range of around 1.2m. This low tidal range is a function of the short distance to an amphidromic point (a position of zero tidal range) which is close to the west coast of Denmark. The average wave height is between 1 to 1.5m, occurring in response to westerly winds. The estimate for the significant design wave height is 5.4m (1:100-year return period). Tidal currents are generally shore parallel with a peak flow of up to 0.5m/s, increasing to 0.8m/s during storms. The sea bed is composed primarily of medium-fine sand (D_{50} = 0.370mm). Up to 80 turbines will be arranged in a grid pattern, spaced a distance of 560m apart, using mono-pile foundations with local scour protection. The EIA for the development comments that *'there will be minor local impact on current, wave conditions and sediment transport in the immediate area of the foundations. This is not expected to have a significant effect on the benthic fauna. Construction of the wind farm will not affect the regional wave conditions, currents or sediment transport along the coast of Jutland at Blåvands Huk and Skallingen. Thus no impact is expected on the marine biology of the international protected areas.'* (ELSAM, 2000). In addition *'the location of the wind turbines will affect the currents in their immediate vicinity. However, the effects will only be of a very local nature. Model calculations show that the total current velocity is reduced by 2% at most before and after the establishment of the offshore wind farm.'* (DHI, 1999). For waves, there is a predicted reduction in height of around 3%, immediately leeward of the development, with no influence expected to occur along the Jutland coast.

C.4 References

Middelgrunden Wind Turbine Co-operative, 2001. Environmental Impact Assessment of the wind farm at the Middelgrunden Shoal. January 2001.

ELSAM, 2000. Horns Rev Offshore Wind Farm. Environmental Impact Assessment of Sea Bottom and Marine Biology. March 2000.

Danish Hydraulic Institute. 1999. Horns Rev Wind Power Plant. Environmental impact assessment of hydrography. Baggrundsrapport nr. 8. Project no. 50396-01.

Table C.1 Inventory of known built Offshore Wind Farms

Site Name	Country	Year online	Sea Location	Distance offshore (m)	Water depth (m)	Number of turbines	Installed Power (MW)	Foundation type
Nogersund	Sweden	1991 (ceased 1998)	Baltic	0.25	7	1	0.22	Tripod
Vindeby	Denmark	1991	Baltic	1.5 to 3	2 to 6	11	4.95	Gravity (Box-caisson)
Lely (Medemblik)	Holland	1994	Inland lake	0.8	5 to 10	4	2	Driven Mono-pile
Tunø Knob	Denmark	1995	Baltic	6	3 to 5	10	5	Gravity (Box-caisson)
Dronten	Holland	1996-7	Inland lake	0.4	5	28	16.8	Driven Mono-pile
Bockstigen	Sweden	1998	Baltic	4	6	5	2.75	Drilled Mono-pile
Blyth Offshore	UK	2000	North Sea	1	8	2	1.5	Drilled Mono-pile
Middelgrunden	Denmark	2001	Baltic	2		20	40	Gravity (concrete caisson)
Utgrunden	Sweden	2001	Baltic	8	7 to 10	7	10.5	Driven Mono-pile
Yttre Stengrund	Sweden	2001	Baltic			5	10	Drilled Mono-pile
Horns Rev	Denmark	Proposed site	North Sea	15	5.8 to 17.5	80	160	Mono-pile

APPENDIX D A REVIEW OF COASTAL PROCESSES WITHIN UK INTEREST AREAS

A brief review of the general coastal process characteristics for each of the six UK Interest Areas is provided to illustrate regional variations and highlight key issues related to each area.

D.1 North East England

This area extends south from Blyth to Tees Bay and is exposed to North Sea conditions. Water depth increases rapidly from the shoreline and within 10km is around 70m deep in some places. The area is subject to intense wave activity due to the deep water near to the shore; this is particularly the case from the north and east where there are long wind fetches.

The sea bed is composed of a thin veneer of loose unconsolidated sediments usually <1m in thickness. They vary from coarse, sometimes gravelly sands near the coast to fine sands further offshore. The only signs of the offshore mobility of these sediments are thin, ephemeral sand streaks running over bedrock with the direction of movement variable but largely southward. However, the shoreline and nearshore sediment movement is principally towards the south. Where there is little or no sand cover then the bedrock seafloor consists of Jurassic mudstones and subordinate sandstones. These would be subject to scour if a wind farm were built upon them.

There is a meso tidal range, and maximum tidal velocities inshore are about 1m/s. Further offshore the velocities decline to around 0.35m/s. Tidal stream directions are largely shore parallel in a NW/SE direction.

D.2 East Coast (Humber Estuary to mid East Anglia)

This area extends over an approximately 10km coastal strip between the mouth of the River Humber, and excluding The Wash, as far as Southwold, Suffolk. Water depths are generally shallower than the region off the North East of England, and are typically greater than 25m, except for local channels such as the Inner Silver Pit, and between some sandbanks.

In the sea area off the Lincolnshire coast the unconsolidated sediments are thin gravels or sandy gravels. Similarly along the north Norfolk coast thin gravels or sandy gravels overlie Chalk bedrock. However, off the eastern shore of Norfolk and Suffolk sand is found often accumulated into large sandbanks such as Haisborough Sand, Hammond Knoll, Scroby Sands, Holm Sand and Newcome Sand. In the channels between them, gravels, sandy gravels and coarse sands are found. There are many indications of the high mobility of these sediments in the form of sand ribbons, streaks, mega-ripples and sand waves largely indicating transport pathways into the sandbank complex from both the north and the south.

There is a macro tidal range, with maximum tidal velocities of approximately 1 to 1.5m/s falling off further offshore to 0.75m/s. The strongest flows are largely shore parallel.

Wave activity can be vigorous particularly from the north and east though the offshore sandbanks do afford some protection to the shoreline.

There are sediment pathways between both shore and offshore while much of the sediment that makes up the beaches is being derived from the highly erodible cliffs along the Yorkshire / Lincolnshire / Norfolk and Suffolk coastline.

Further offshore, similar sea and sea bed conditions apply with many more sandbanks and inter-bank channels. The principal sediment is a fine to medium sand often displaying features indicative of high mobility. A great number of deep, glacially eroded channels, refilled with glacial outwash and marine trangressive sediments are also located in this offshore zone. Sea bed conditions are therefore variable with susceptibility to erosion or scour also being very variable.

D.3 Thames Estuary

This area is contained within the outer reaches of the Thames Estuary as far seaward as approximately 1° 40'E. Water depths vary between 3m (above CD) on the drying sandbanks to a maximum of 30m below CD in the eastern reaches. The bathymetry mostly shows water depths of 10 to 20m within the channels between the sandbanks.

This area represents a drowned section of the palaeo River Thames and its tributaries, that in most places has been infilled by sediments ranging from gravels through to silts and clays. For example, Kentish Flats are bordered on the north by the Palaeo Thames and are underlain by the infilled PalaeoSwale itself a tributary of the Palaeo Thames. Overlying much of these sediments are fine to medium sands accumulated into a series of sandbanks with their surface sediments being highly mobile usually in a clockwise direction about the banks. The thickness of sediments are therefore highly variable ranging from <1m up to 35 or 40m. For instance, the Gunfleet Sands, in the northern part of the estuary, or the East Middle Sands on the Kentish Flats.

Tidal currents are strong, particularly in the inter-bank channels where velocities up to 1.5m/s are found. The main tidal flow direction is approximately parallel to the sandbanks, except in the southern sector where it runs east/west.

There are parts of this area where sediment is exchanged with the shoreline and large quantities of sediment finds its way into this area from the erosion of surrounding cliffs (eg Isle of Sheppey) and from further east in the North Sea.

Overall, the Thames Estuary can be regarded as a highly mobile, depositional area.

Bedrock of the stiff to very stiff, London Clay, and in a few places of chalk and Tertiary sands is found at the sea bed in several areas particularly at the north and south sides. Wave activity from the east, north and the west is most effective but is not so severe as that experienced further to the north within the North Sea.

Further seaward in the Southern Bight of the North Sea, the London Clay bedrock is often close to or at the sea bed. Several more north-south trending sandbanks are found there and these provide some additional protection to the main estuarine area to the west.

D.4 Bristol Channel

This area lies in the middle reaches of the Bristol Channel, between Penarth and Weston-super-Mare in the east and Hartland Point and the Mumbles in the west. Water depths are generally greater than 25m except in the central parts where depths up to 40m are to be found.

Loose, unconsolidated sediments are generally thin less than 1m except where sandbanks are situated and in Swansea Bay and Bridgwater Bay. Here a thickness of over 15m can be found. Like the Thames Estuary, the Bristol Channel is essentially the drowned palaeovalley of the River Severn. The main channel has been largely exhumed but the tributary valleys still contain some sediments. Silts and sands are found in Swansea Bay and Bridgwater Bay, gravels in some places towards the sides of the channel and fine to medium sands in the major sandbanks of Helwick, Scarweather, Nash, Holm and Culver Sands. These banks show a high surface mobility in a clockwise direction. Sediment movement is flood dominated in the nearshore zones and ebb dominated in the central axis. This mobility is indicated by megaripples and E/W aligned sand ribbons and streaks.

The whole of the Bristol Channel is a macro-tidal environment with the largest tidal ranges of all UK Interest Areas, with currents and tidal amplitudes also amongst the highest in the world. Tidal velocities in excess of 2m/s can occur at peak ebb in the channel centre and 0.7m/s in Swansea Bay where mean tidal amplitudes of 6.2m are recorded.

Wave activity is very powerful at times, particularly from the west and south-west where there is an uninterrupted fetch of over 3,000 miles across the North Atlantic. Wave heights up to 5m have been recorded at the Scarweather Light Vessel.

Where little or no sediment covers the bedrock then mudstones of Jurassic age constitute the seafloor. These would be liable to erosion if a wind farm were to be built up on them. Around the margins harder rocks constitute the cliffs and seafloor and these provide little towards the sediment budget of the shore or seafloor. However, on the north side some glacial tills are found that do provide some sediment input. Further to the west towards the Isle of Lundy, the water deepens appreciably and the seafloor of Jurassic mudstones, sandstones and limestones is covered by mobile sands formed into sand waves.

D.5 Liverpool Bay

This area lies across Liverpool Bay between Llandudno, Wales and Barrow in Furness, Cumberland, all within the Irish Sea. Water depths show a gently, westward sloping seafloor down to approximately 35 to 40m.

Recent sediments cloak the whole area and largely consist of up to 2m of rippled sand in the more coastal regions with these becoming more gravelly or muddy sand with various amounts of silts and clays, further offshore. Underlying these surface materials is often a 1 to 2m layer of sandy gravels. Within Liverpool Bay lies a deep cut, palaeo-channel formed initially by the fluvial and glaciofluvial drainage from the Rivers Mersey, Dee and Ribble. Lying within this largely infilled channel, recent sediments can be up to 50m in thickness but elsewhere they are usually around 10m in thickness. Underlying these sediments and much of the area is the firm to stiff, Irish Sea glacial till. This till is also found in many places around the bay and sometimes constitutes the cliffs and a potential source material for the beaches of the area. On the south side of the bay are found hard rock cliffs of Carboniferous limestone around Colwyn Bay and Llandudno and sandstones and mudstones on the west side of the River Dee. These will not provide much sediment to the nearby sea area or the beaches. Alongshore and nearshore sediment movement is largely to the north on the west facing section and to the west on the north facing section of the bay.

Tidal amplitudes are high particularly on the southern side of the bay and tidal currents range up to 1m/s, falling off further offshore. Except around headlands and in the river estuaries the tidal currents are approximately aligned east-west.

Sand movement, largely due to this tidal activity is indicated by the presence of sandwave fields in the outer parts of the area. These sand waves have their crestlines aligned approximately north-south, are up to 2m in height and between 10 to 20m in wavelength. They are found particularly off the Ribble Estuary and west of 3º 30'W.

Wave activity is particularly intense from the west and north west though the bay is partially sheltered from other wind wave directions.

D.6 Solway Firth

This area lies within the Solway Firth with its outer limits at approximately St Bees Head and the south-eastern extremity of Wigtown Bay. Water depths vary between 0 and 1m (above CD) on the sandbanks of the Firth and decline gently towards the west to around 35m (below CD).

Unconsolidated recent sediments are principally rippled sands, or gravelly sands nearer to the coast or in patches in the channels between the sandbanks. As in Liverpool Bay, the sands are sometimes underlain by a layer of gravelly sand of between 1 and 2m in thickness. Once again this area is dominated by a deep palaeo-channel cut originally by the fluvial and fluvioglacial action of the rivers Lyne, Eden, Annan, Nith and their tributaries. Within the Solway Firth, this channel has been largely in-filled with up to 20m of gravels, sands, silts and organic clays. Within the Firth are a number of sandbanks that exhibit interconnected sediment pathways and a highly mobile surface layer that is essentially clockwise around the principal banks. Underlying these more recent sediments is a thick, firm to stiff, glacial till. This deposit is also found around the coast and, as such, this unit does provide sediment by cliff erosion for the shoreline. Because of the interconnected pathways linking this shore zone with the sandbanks some of this material supplies the offshore areas. The Solway Firth is essentially a sediment depo-centre with silts and clays being deposited in the quieter backwater areas, and sands in the bay. Longshore drift is essentially towards the east. Some areas around the Firth have a hard rock cliffed section of Carboniferous mudstones and siltstones or Permo-Triassic mudstones or sandstones. These will provide only a very little material to the recent sediment budget.

Tidal amplitudes are high within the Firth and current velocities up to 1.75m/s are found within the channels between the sandbanks. Further to seaward these velocities fall off to < 1m/s. The principal flow directions are NE/SW in the Firth. These veer to a more east-west direction in the outer reaches of the area.

Wave activity is particularly intense from the south-west, but the southerly and westerly directions are also important.

Further offshore, beyond 4ºW, the sandy floor that either overlays the glacial till or Perm-Triassic mudstones and sandstones is found as a number of large sandwave fields with N/S crestlines, 2m amplitude and 10 to 20m wavelengths. The thickness of this recent sediment layer of sands, silts, and organic clays, is up to 40m in the palaeo channel, less so on its flanks.

APPENDIX E CHARACTERISATION OF FIRST ROUND SITES

The general characteristics for each of the first round offshore wind developments, and their respective coastal regimes, have been compiled from published information and, wherever possible, supplementary details provided by the various developers. The following parameters have been considered:

- distance to shore
- water depth
- sea bed profiles
- tidal conditions
- wave conditions
- sediment regime

This information has then been examined to establish upper and lower limits for a range of wind farm parameters, as well as calculation of typical values. A further comparison has also been made against equivalent values derived from the Blyth Offshore wind farm, which was commissioned in late 2000.

E.1 Distance to Shore

The following table is based on general information released as part of the Crown Estate announcement in April 2001 relating to first round wind farm sites:

Distance to shore	(km)
Closest development	1.5
Furthest development	10
Average distance	7
Blyth Offshore	1

This information reflects the shortest distance between the coast and the closest turbine in the wind farm. *Note:* these distances may be subject to small changes following detailed site investigations, environmental assessments, etc.

E.2 Water Depth

A general assessment of likely water depths over the wind farm development areas has been made using the appropriate Admiralty Chart for each site. All depths are quoted relative to CD and are taken as a depth characteristic for each development area.

Water Depth	(m)
Shallowest site	2.0
Deepest site	20.0
Average depth	7.5
Blyth Offshore	6.0

E.3 Sea bed Profiles

Similarly, a general assessment of a typical profile across the sea bed from the adjacent coastline has been made using the appropriate Admiralty Chart applicable to each site. (*Note*: gradients are quoted for sub-tidal regions).

Gradient	
Shallowest gradient	1:1,100
Steepest gradient	1:270
Average gradient	1:815
Blyth Offshore	1:160

(*Note*: the steepest gradient is associated with the site closest to the coast)

Figure E.1 illustrates the comparison between the derived 'reasonable worst case' profile and profiles from first round sites intended for development at a distance of less than 6.9km from the coastline (average value). The general shape of the 'reasonable worst case' profile is consistent with these sites, offering a steep nearshore profile, the presence of a nearshore sandbank (the intended platform for the wind farm) of a suitable dimension and tailing off over a shallower gradient into deeper water. The intention for the 'reasonable worst case' scenario is to create a profile similar to Site 10, but position the sandbank closer to the coastline to generate the 'reasonable worst case'.

Figure E.2 illustrates a comparison between the derived 'typical' profile and profiles from first round sites intended for development at a distance of more than 6.9km from the coastline (average value). The general shape of the 'typical' scenario is very consistent with these sites, with a moderate nearshore profile, the presence of a sandbank of a suitable dimension further offshore (the intended platform for the wind farm) tailing off into a shallower gradient to deeper water. The 'typical' scenario profile matches well with most sites, and especially Site 9.

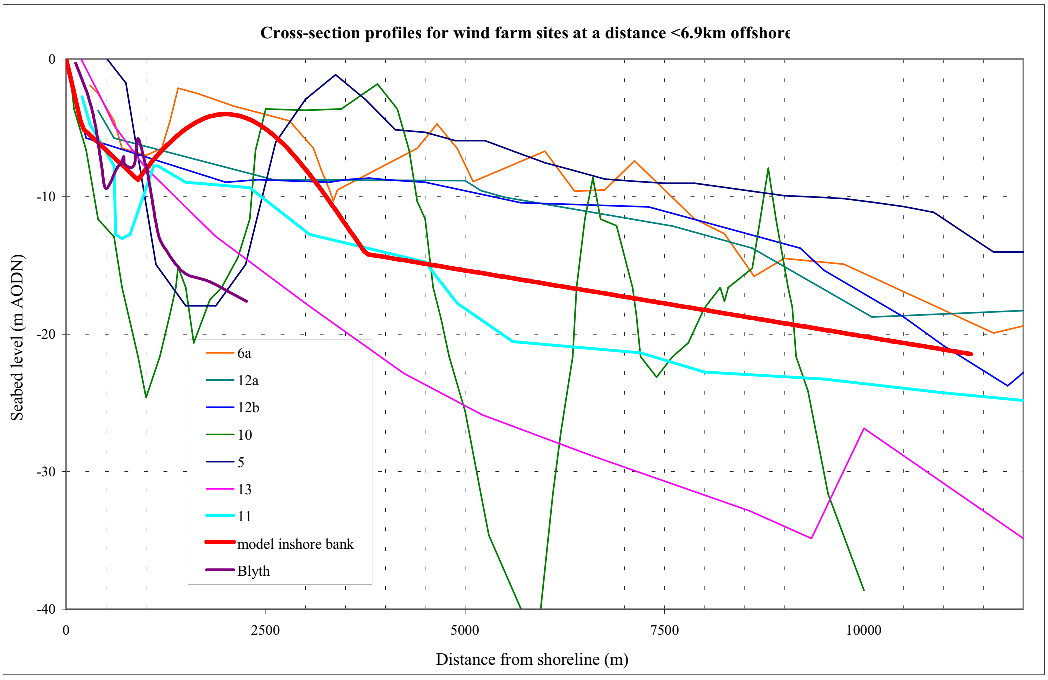

Figure E.1 Cross-section profiles for nearshore sites

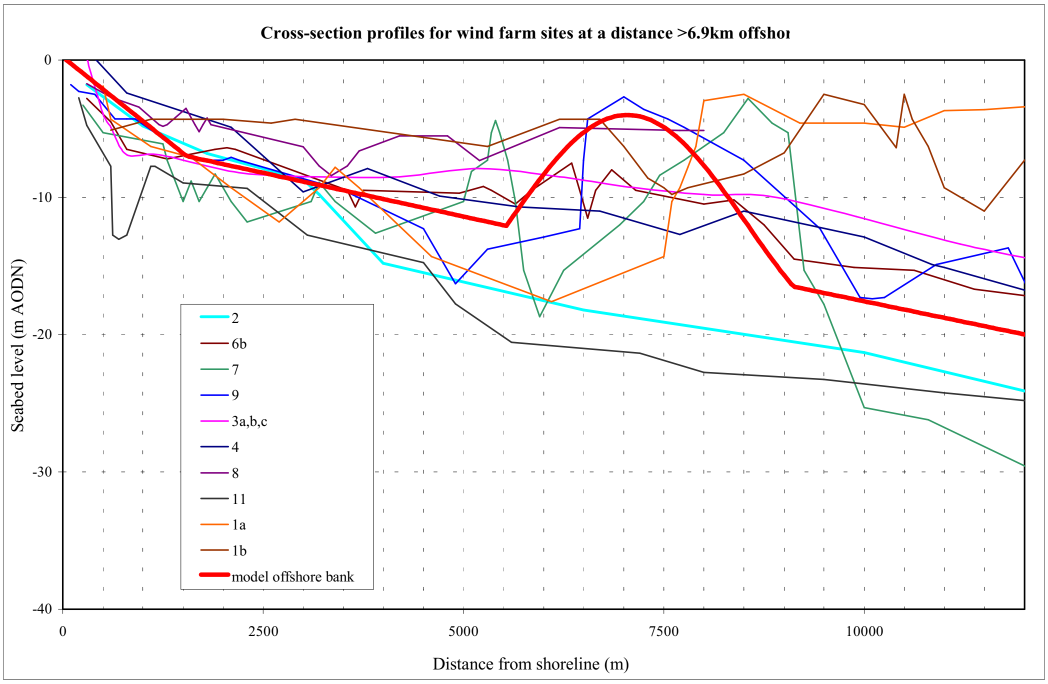

Figure E.2 Cross-section profiles for offshore sites

Since a site with a steep seaward gradient will reach deeper water over a shorter distance, then it is perceived that wave conditions at such sites have the potential to be more severe than at sites in areas of shallow gradients where wave shoaling (and perhaps also wave breaking) will already have reduced wave energy.

E.4 Tidal Conditions (Water Levels)

Reviewing the tidal conditions applicable to first round sites provides the following information for water level variations. (*Note:* These values have been derived from the nearest tidal station indicated on the relevant Admiralty Chart).

Tidal Range	Spring (m)	Neap (m)
Largest (Bristol Channel)	8.9	4.2
Smallest (East Coast)	1.9	1.0
Average	6.4	3.3
Blyth Offshore	4.2	2.2

Tidal range can also be expressed in semi-quantitative terms based on various thresholds:

Micro	< 2m
Meso	2 to 4m
Macro	> 4m

The adoption of this terminology would characterise the majority of sites with a macro tidal range for spring tides. The comparison to be made here is that existing built offshore wind farms across Europe are all at sites where the tidal range is minimal and equivalent to a micro-tidal regime.

E.5 Tidal Conditions (Currents)

Reviewing the tidal conditions applicable to first round sites provides the following information for tidal currents (*Note:* These values are derived from the nearest tidal stream position indicated on the relevant Admiralty Chart). Tidal streams are typically shore parallel around the UK coast.

Tidal Flows	Spring (knots)	Neap (knots)
Largest (Solway)	3.9	2.2
Smallest (Tees)	1.0	0.5
Average	2.1	1.2
Blyth Offshore	0.7	0.3

(1 knot = 0.514m/s)

E.6 Wave Conditions

Reviewing the wave conditions applicable to first round sites provides the following approximate information for 'offshore' wave conditions. The numbers quoted are the sea-state parameters of significant wave height and mean wave period associated with annual exceedence probabilities of 10% and 50%

Wave conditions	10% exceedence	50% exceedence
Largest (Bristol Channel)	3.5m, 9s	1.5m, 7s
Smallest (Thames)	1.5m, 6s	0.5m, 5s
Average	2.3m, 7s	1.0m, 5s
Blyth Offshore	2.0m, 7s	1.2m, 5s

E.7 Sea Bed Sediments

The proposed first round developments are typically located in shallow water areas, many of which are associated with sandbanks. A *general* view on likely seabed conditions around the UK is available from various British Geological Survey (BGS) publications. These include both maps and, in some areas, detailed reports. The maps which cover the whole of the UK continental shelf area, are part of the 1:250,000 Series and each are published usually in three separate sheets:

Seabed Sediments & Holocene
Quaternary Geology
Solid Geology

The range of potentially mobile sea bed sediment types indicated at the first round sites includes:

S (sand), msG (muddy sandy Gravel), gmS (gravely muddy Sand), G (Gravel), sG (sandy Gravel), mS (muddy Sand).

The most typical sediment type and the one adopted in the present study is S (sand).

It should be noted that this data provides only a first approximation of the type of material likely to be found in these areas. For any site-specific application it is recommended that a detailed site investigation is carried out to determine more precisely the local sediment type, thickness, texture and mobility.

APPENDIX F REVIEW OF MODELS

The types of computer models which are applicable to describing coastal process issues have been drawn from various guideline publications (HR Wallingford 1994, 1996a and 1996b).

F.1 Tidal Behaviour

There is a wide availability of hydrodynamic models suited to describing tidally driven flows in shallow water coastal environments. The most common of these models are reviewed in HR, 1996a. In the context of examining the potential effects of offshore wind development on coastal processes, such models need to be able to incorporate a definition of the wind farm in terms of its presence on the sea bed. This can be achieved in one of two ways:

a) Directly - by choosing a grid scheme that can resolve the scale of the structure (*Note*: requires a fine grid scheme at a scale equivalent to the dimensions of the turbine).

b) Indirectly - through parameterisation of the effects (hydraulic resistance) that a single turbine creates and implementing a local friction coefficient within the development site at turbine positions, to account for sub-grid scale processes (*Note*: avoids the need for a fine grid scheme).

Note: The latter approach has been applied to a range of studies supporting the successful development of offshore wind farm sites around the coast of Denmark.

The minimum requirement is to be able to determine the influence the wind turbine structures may have on the tidal flow patterns and resolve this on a 2D-horizontal grid scheme.

Applicable models include:

Delft3d - hydrodynamic module (structured finite difference curvilinear grid)
MIKE21/ 3 - hydrodynamic module (HD & NHD) (structured finite difference uniform grid, with optional nesting)
Telemac - hydrodynamic module (unstructured finite element grid)
DIVAST - hydrodynamic module (structured finite difference uniform grid)

F.2 Wave Regime

Similarly, there is a wide availability of wave models suited to describing coastal wave environments. The most common of these models are reviewed in HR, 1996b. In the context of examining the potential effects of offshore wind developments on coastal processes, such models need to be able to incorporate a definition of the wind farm in terms of its presence on the sea bed and through the water column.

As waves travel towards shallower depths, it may become appropriate to use more sophisticated models that not only include wave refraction and shoaling, but also diffraction (around vertical structures), reflection and energy dissipation (eg wave breaking).

Diffraction becomes important when the diameter of the vertical obstruction, D, is more than approximately 20% of the wave length, L, ie $D/L > 0.2$ (Isaacson, 1979).

The minimum requirement is to be able to determine the likely changes to waves passing through the wind farm site and the degree to which significant changes in the wave pattern occur (ie interference with wave regime).

To determine characteristics of the local wave regime will normally require the transformation of a 'deep water' (offshore) wave to the wind farm site. Models that provide this functionality include:

Delft3d	- HISWA wave model
MIKE21	- NSW wave model
COWADIS	- wave refraction model from Electricité de France
LINDAL	- wave refraction model from Applied Wave Research

To examine the local wave behaviour and interactions may require a wave disturbance model that incorporates diffraction and reflection, and preferably accounting for irregular waves. An example of the types of models that provide this functionality include:

MIKE21	- BW Boussinesq wave model (accounts for irregular waves)
MIKE21	- EMS (elliptic mild slope) wave model (regular waves only)
DIFFRAC	- Delft Hydraulics
ARTEMIS	- Electricité de France (elliptic mild slope wave model, based on linear wave theory)
CGWAVE	- University of Maine

F.3 Combined Effect of Wave-Current Interaction on Sediment Transport

Within the nearshore areas the process of wave-current interaction and wave driven currents will be the dominant mechanism contributing to sediment transport. An essential requirement for any model will, therefore, be the incorporation of wave-current interaction within any sediment module.

Applicable models include:

Delft3d	- sediment transport module

MIKE21 - sediment modules (ST, MT and PA)
Telemac - sediment module

Note: these modelling techniques can not fully deal with the complexities of describing the development of local scour in a practical manner for circumstances where waves and currents interact. At present, it is more common for scour to be investigated using an analytical approach. Further details on methods to determine scour are provided in Appendix G.4.

F.4 References

HR, 1994. Guidelines for the use of computational models in coastal and estuarial studies. Review of models in use for engineering studies. SR 390. HR Wallingford.

HR, 1996a. Guidelines for the use of computational models in coastal and estuarial studies. Flow and sediment transport models. SR 456. HR Wallingford.

HR, 1996b. Guidelines for the use of computational models in coastal and estuarial studies. Wave transformation and wave disturbance models. SR 450. HR Wallingford.

Isaacson M., 1979. Wave induced forces in the diffraction regime. In: Mechanics of Wave-Induced Forces on Cylinders.

APPENDIX G THEORETICAL CONSIDERATIONS

The hydrodynamic forces transmitted to the turbine support structures by waves and currents have the effect of removing some energy from the marine environment. This energy would otherwise have been consumed in wave and tidal current mass flow, which, in turn, would provide a sediment transport capability.

The resistance offered by the turbine support structures can be predicted in terms of wave and current forces and expressed as an equivalent seabed resistance term in the regional wave and tidal models. In the area coastward of the wind farm this has the effect of reducing the wave and tidal current energy due to the resistance offered by the structures.

The far field resistance issue is separate from any wave scattering and reflection that may occur locally around the structures. Local effects upon the waves have been studied by using a fine grid wave model, capable of directly predicting the scattering and reflection effects in the near field of the structures.

G.1 Structural resistance in tidal currents

The MIKE21 Hydrodynamic Model (MIKE21HD) provides a method to describe the current-induced drag force acting on a structure by equating this force with an equivalent bed shear stress contribution (DHI, 2000), as follows:

$$\tau \cdot \Delta x \cdot \Delta y = n \cdot F$$

where

$\tau =$ equivalent shear stress

$\Delta x, \Delta y =$ grid spacing dimensions

$n =$ number of structures allocated to one grid cell

$F =$ the drag force acting on one structure, that is:

$$F = \rho C_d A V^2 / 2$$

$\rho =$ density of water

$C_d =$ drag coefficient

$D =$ diameter of the structure that is exposed to the tidal current flow

$V =$ depth - average current speed

This method ignores any contribution made by lateral forces which is reasonable because these forces are oscillatory in nature and are of equal and opposite sign. Consequently, when averaged over time, the two opposite forces cancel each other and make a zero net contribution. Furthermore, at peak tidal current flow, the Reynolds' number for steady flow past the structures is of the order of 10^7. Under these circumstances, the in-line force is likely to be approximately 2½ times the fluctuating lateral force. The latter cannot therefore contribute greatly to the total force acting on the cylinder.

References (G1)

Danish Hydraulic Institute, 2000. MIKE 21 Coastal Hydraulics and Oceanography. User Guide – Hydrodynamic Module (Reference Manual).

G.2 Structural resistance in waves

Wave loading on structures conforms to two regimes, which, to a certain extent, overlap with each other.

The first regime occurs when the structure is large enough to significantly scatter the incoming waves. Under these circumstances, the wave loading lies in the *diffraction* regime. The equivalent inertia force transmitted to the structure by the waves is then less than that which would accrue if the scattering effect were to be ignored. The limit at which diffraction exerts a significant influence upon wave forces depends on the diameter of the structure (D) compared to the length of the incoming wave (L). For D/L values of more than 0.2, diffraction becomes increasingly important.

The second wave-loading regime is one in which the structure has little scattering effect upon the incoming waves. This regime applies to values of D/L of less than 0.2. Under such circumstances, the wave force is usually expressed as the sum of drag and inertia forces. The drag force is due to separation of the fluid flow around the form of the cylinder and is associated with the velocity of flow of the water particles in the wave. The inertia force is generated by the influence of the cylinder upon the acceleration of water particles, resulting in a transfer of fluid momentum. Under such circumstances, the normal procedure would be to apply *Morison's equation* to predict the wave force. This solution expresses the wave force as the sum of drag and inertia forces. Force transfer coefficients are combined with the predicted velocity and acceleration of water particles in the wave, to generate the total predicted wave force. The velocities and accelerations have to be predicted using an appropriate wave theory.

In the present study, wave periods of 5 and 7s have been identified as appropriate to waves with a probability of exceedence of 50 and 10%, respectively. The associated wave heights (H) are 1 and 2.5m. In turn, the length (L) of such waves would be approximately 30 and 46m, respectively, in a typical water depth of 5m, on top of the sandbank. Furthermore, in deep water, the wavelengths would be 39 and 77m, respectively. Since a cylinder diameter (D) of 5m has been identified as a reasonable design value for the turbine support structure, it follows that diffraction is not likely to exert a significant influence upon the associated wave force, although some local reflection will of course occur near the structure. The smallest D/L value applying under the average wave conditions considered here is 0.17 – a value which is less than the threshold of 0.2 for diffraction effects to become important in the far-field.

The wave forces have therefore been predicted using Morison's equation. There are many references available concerning the prediction of regular wave forces; one of the most comprehensive is that by Sarpkaya and Isaacson (1981), which also covers the diffraction regime of wave loading.

- Appropriate wave theories

Before applying Morison's equation, the velocities and accelerations of the water particles in the wave have to be predicted using a suitable wave theory. The most popular theory is *small amplitude wave theory.* This theory is also frequently known as the *Airy solution*, or *linear wave theory*. It is based upon the assumption that the amplitude of the wave is very small and that consequently, higher order terms in the solution may be neglected – hence the name *linear theory*. In linear theory, water particle velocities and accelerations are exactly 90 degrees out of phase with respect to each other. Linear theory is really only appropriate when the wavelength is small in relation to the water depth and the wave height is also small when compared to the wavelength.

Le Méhauté (1976) provided a comprehensive review of the appropriate suitability ranges for linear and other wave theories. According to his work, linear theory would not be appropriate to the present study for waves on the shoal, because their wave heights would render the assumption of small amplitude inadequate. The result of using linear wave theory under such circumstances is that the wave force would be underestimated. This is particularly true of the wave with a 10% exceedence probability. The more the wave departs from the spirit of the linear assumption, the more inaccurate the predicted wave force becomes. Consequently, in the present study, a non-linear, finite amplitude wave theory was required for predicting the forces on the structures.

For the present purposes, the *stream function* series solution originally developed by Dean (1965) was used, for predicting the water particle velocities and accelerations in the wave, as well as the surface profile of the wave itself. The stream function solution satisfies analytically all of the governing equations and the boundary conditions, with the exception the Bernoulli condition at the free surface. This condition is satisfied by a suitable numerical procedure. Unlike the linear wave solution, the maximum drag and inertia forces are not 90 degrees out of phase with each other in the stream function solution. As the amplitude of the wave increases to the breaking point, the maximum drag and inertia forces gradually come closer together in phase. It is possible for them to be only a few degrees apart in a steep near-breaking wave.

- **Structural resistance in waves**

MIKE21 NSW predicts the dissipation of wave energy due to sea bed friction using a quadratic friction expression:

$$dE/dt = -c_{fw}/(8g\sqrt{\pi})\cdot[\omega H_{rms}/\sinh(kd)]^3$$

where

$E = H_{rms}{}^2/8$

ω = wave frequency

H_{rms} = root mean square wave height

k = wave number

d = water depth

c_{fw} = the wave friction factor

Nielsen (1992) provides more detailed empirical information concerning modelling the sea bed roughness and the energy dissipation in circumstances where bed forms are present.

In the present study, the wave friction factor has been increased for those model grid cells containing a structure, by increasing the wave friction factor, so that the shear force on the bed is equal to the force exerted upon the actual structure, by the wave.

In the present study, Morison's equation has been used to predict the wave forces, based upon a linear sum of the drag and inertia forces acting upon the vertical cylinder:

$$F\cdot dz = [\rho C_d DV|V|/2 + \rho(\pi D^2/4)C_m dV/dt]\cdot dz$$

where

ρ = density of water

C_d = drag force coefficient for the structure in waves

C_m = inertia force coefficient for the structure in waves

V = horizontal water particle velocity in the wave at a height z above the seabed

D = diameter of the cylindrical structure

dV/dt = $\partial V/\partial t + (u+U)\partial u/\partial x + 0.5w\partial w/\partial x$: generalised acceleration term in the force equation

u, w are the horizontal and vertical water particle velocities in the wave

U = the wave-induced current

The total force on the structure is obtained by integrating from the sea bed level up to the mean water level, in the case of linear wave theory. Using stream function solution, the integration is obtained by properly integrating up to the free surface of the wave. The usual practice is to adopt an inertia force coefficient of 2.0 in the wave force calculations. This is the theoretical solution for time-dependent rectilinear flow past a cylinder, but experience has indicated that it can also be validly applied to making wave force predictions. For the present study, a drag force coefficient of 1.0 has been adopted. However, for the wave and cylinder configurations being considered here, the wave inertial forces dominate loading. Therefore, the choice of drag force coefficient is not so critical.

- **Values of wave force predictions**

Table G.1 compares predicted wave forces on the 5m diameter towers, calculated using linear and stream function series solutions in combination with Morison's equation, for two example cases. The linear solution is very similar to the stream function result for the 50% exceedence wave condition. However, for the 10% exceedence wave, linear theory significantly underestimates the wave force, when compared with the stream function approach. The linear wave solution can therefore result in a subsequent underestimate of the resistance offered by the structures to the wave flow. Results obtained using the stream function solution have therefore been adopted for the present study.

It is worth noting that the present predictions indicate that maximum wave forces are dominated by the inertia component and that the drag force does not make a great contribution. The Morison equation is known to be somewhat less reliable when the contributions from drag and inertia are similar in magnitude. For an inertia-dominated loading case, the Morison solution provides a more reliable estimate of maximum wave forces. Consequently, the present results are unlikely to be adversely affected by any doubt concerning the reliability of the Morison equation.

Table G.1 Wave force predictions for waves of 50 and 10% exceedence probabilities

Prob. (%)	Hs (m)	Tz (s)	Stream Function Waves: kN force			Linear Waves: kN force		
			Max Drag	Max Inertia	Max Total	Max Drag	Max Inertia	Max Total
50	1.0	5.0	6.5	154.5	154.8	4.8	153.3	153.3
10	2.5	7.0	76.6	368.7	389.4	34.2	294.7	294.7

Note: the above calculations are based upon a sea bed level of –4m ODN, for a generic structure situated upon the sandbank. The water level in this example is 1m ODN. The diameter of the cylinder is 5m.

References (G2)

Danish Hydraulic Institute (DHI). 1998. Nearshore Spectral Wind-Wave Module. Reference Manual. Release 2.7.

Dean, R.G. 1965. Stream function representation of nonlinear ocean waves. Journal of Geophysical Research, Vol.70, pp. 4561-4572.

Le Méhauté, B. 1976. An Introduction to Hydrodynamics and Water Waves. Published by Springer-Verlag, Dusseldorf.

Nielsen, P. 1992. Coastal Bottom Boundary Layers and Sediment Transport. Published by World Scientific – Advanced Series on Ocean Engineering. ISBN 981-02-0472-8.

Sarpkaya, T. and Isaacson, M. 1981. Mechanics of Wave Forces on Offshore Structures. Published by Van Nostrand Rheinhold. ISBN 0-442-25402-4.

Swift R. and Dixon, J. 1987. Transformation of regular waves. Proc. Instn. Civ. Engrs. Part 2, 83, June, 359-380.

G.3 Validation of near-field wave model

In order to investigate the near-field wave processes a two-dimensional wave model has been configured capable of predicting the interaction between reflected waves from all the structures in the wind farm. The model was first tested by comparing the results for one tower against the analytical solution by Havelock as reported by Longuet-Higgins and Cartwright (1957) and subsequently amplified by Sarpkaya and Isaacson (1981).

The first means of validating the two dimensional scattering model from an analytical perspective is to investigate the variation in the resultant wave pattern immediately up-wave of the front face of the cylinder. In this case, the cylinder is being considered as a plane surface-piercing reflective obstacle. The incident wave field interacts with the waves being reflected from the front face of the cylinder. The resultant free surface elevation may be integrated over time to obtain a prediction of the wave height variation along a line normal to the crests of the incoming waves.

Referring to Weigel (1964), the following equation presents the resultant water surface elevation in front of a structure that is partially reflecting an incoming wave field:

$$\begin{aligned}\eta(x,t) = {} & (1/2)(H_i + H_r)\cos(2\pi x/L)\cos(2\pi t/T) + \\ & (1/2)(H_i - H_r)\sin(2\pi x/L)\sin(2\pi t/T)\end{aligned}$$

The subscripts *'i'* and *'r'* refer to the incident and reflected wave fields, respectively. The symbol '*H*' constitutes the wave height; *'T'* is the period and *'L'* is the wavelength. The equation may be simplified to the following format:

$$\begin{aligned}&\eta(x,t) = A(x)\cos(2\pi t/T) + B(x)\sin(2\pi t/T), \text{ where}\\ &A(x) = (1/2)(H_i + H_r)\cos(2\pi x/L)\\ &B(x) = (1/2)(H_i - H_r)\sin(2\pi x/L)\end{aligned}$$

The square of the resultant free surface elevation may then be expressed as:

$$\begin{aligned}\eta^2(x,t) = {} & A^2(x)\cos^2(2\pi t/T) + B^2(x)\sin^2(2\pi t/T) + \\ & 2A(x)B(x)\cos(2\pi t/T)\sin(2\pi t/T)\end{aligned}$$

Integrating the above expression over one wave period and then dividing by wave period T, the following results apply:

$$\begin{aligned}&\overline{\eta^2(x)} = H_i^{\,2}[(1+\alpha)^2/4 - \alpha\sin^2(2\pi x/L)]/2, \text{ where}\\ &\alpha = (H_r/H_i)\end{aligned}$$

The RMS value of free surface elevation therefore fluctuates spatially between the following limits:

$$\sqrt{\overline{\eta^2}}(\max) = (1+\alpha)H_i/(2\sqrt{2}), \text{ for } x = 0, L/2, L, 3L/2, \text{ and...}$$
$$\sqrt{\overline{\eta^2}}(\min) = (1-\alpha)H_i/(2\sqrt{2}), \text{ for } x = L/4, 3L/4, 5L/4 \text{ etc.}$$

The resultant wave height can therefore be expected to vary between limits represented generically by the above solution, with a spatial frequency of one-half of the wavelength.

Description of the analytical solution for wave scattering by a single structure

Longuet-Higgins and Cartwright (1957) (L-H&C) used an analytical solution due to Havelock for the reflection of waves from a vertical, circular cylinder. Their results were initially used in the present study to obtain a preliminary idea of whether scattering effects would influence the wave field in the vicinity of the wind farm. In addition, it was necessary to investigate the field of wave heights due to the interference of these scattered waves with the incident waves. Since the L-H&C results were not in a convenient form to do this, it was decided to undertake some further calculations based on Havelock's solution as reported in L-H&C and in Sarpkaya and Isaacson (1981). This work has been used to prepare Tables G.2 to G.6 and Figures G.1 and G.2.

Consider a train of plane sinusoidal waves (the *incident* wave field) which propagates past a vertical circular cylinder. The waves are characterised by their amplitude, *a*, equal to half the crest to trough height *H*, and their period *T*. Using the wave period and the water depth it is possible to calculate the length λ of the waves and the wave-number, *k*, defined by $\frac{2\pi}{\lambda}$. The Havelock solution allows calculation of the field of waves, which are scattered from the cylinder. These waves have the same period as the incident waves and they radiate out from the scattering cylinder forming a circular pattern. As they radiate out from the cylinder their amplitude decays, and conservation of energy requires that they decay in proportion to $\sqrt{\frac{1}{r}}$ as *r*, the distance from the mono-pile becomes large (several mono-pile radii). This is quite a slow rate of decay. The scattered waves then interfere with the incident wave field to give a pattern of waves whose amplitudes vary with position.

The question being addressed is whether this interference pattern (which is due to the presence of the turbines) is significantly different from the incident field which exists in the absence of the turbines.

The 'relative amplitude' of the reflected waves is controlled by *ak*, where *a* is the radius of the cylinder and *k* is the wave-number of the incident wave (*ak* is thus 2π times the ratio of the pile radius to the wavelength), and the direction,

θ of the reflected wave. The relative amplitude is defined by $\frac{a^r}{a^i}$ where a^r is the amplitude of the reflected wave and a^i is the amplitude of the incident wave. The amplitude of the reflected wave is a function of position, defined in polar co-ordinates (r, θ) with respect to the centre of the cylinder.

Results of the analytical solution

Three example wave periods of 5, 7 and 10s have been selected, with a water depth of 5m as being representative of the sandbank sites in the first round. An upper limit on mono-pile diameter would be about 5m, giving a radius of 2.5m. For a gravity base the diameter might be 15m, giving a radius of 7.5m, which for simplicity is assumed to be surface piercing.

Table G.2 gives the values of the wavelengths for the three wave periods, and of the controlling parameter *ak* for the example mono-pile and gravity base.

Table G.2 Wave length data for example waves
(Depth is 5m. Mono-pile radius is 2.5m, Gravity base radius is 7.5m)

Wave period (s)	**wave number, *k* (m^{-1})**	**wave length, λ (m)**	***ak* for mono-pile**	***ak* for gravity base**
5	0.2073	30.3	0.5183	1.5547
7	0.1376	45.7	0.3440	1.0320
10	0.0928	67.7	0.2320	0.6960

Table G.3 gives relative amplitudes of the scattered waves directly in front of the mono-pile. The values in the tables are given with respect to unit amplitude incident waves (that is, the incident waves have a crest to trough height of 2).

Table G.3 Relative amplitudes, mono-pile case, a = 2.5 m, reflection from front of mono-pile (θ = 180°)

ak	***r* = 2.5 m (At mono-pile surface)**	***r* = 25 m**	***r* = 125 m**	***r* = 250 m**	***r* = 500 m**
0.5183	0.6552	0.1850	0.0830	0.0587	0.0415
0.3440	0.4429	0.1133	0.0505	0.0358	0.0253
0.2320	0.2850	0.0657	0.0291	0.0206	0.0145

Table G.4 gives the same information as Table G.3, but for the gravity base.

Table G.4 Relative amplitudes, gravity base case, a = 7.5 m, reflection from front of base (θ = 180°)

ak	*r* = 7.5 m (At base surface)	*r* = 25 m	*r* = 125 m	*r* = 250 m	*r* = 500 m
1.5547	0.8494	0.3533	0.1409	0.0981	0.0688
1.0320	0.7503	0.3899	0.1759	0.1247	0.0883
0.6960	0.7717	0.4011	0.1808	0.1282	0.0908

Tables G.5 and G.6 repeat the information given in Tables G.3 and G.4 respectively, but for a scattering angle of 45°, ie behind and to the side of the turbine base.

Table G.5 Relative amplitudes, mono-pile case, a=2.5m, reflection behind and to the side (θ=45°)

ak	r=2.5m (At mono-pile surface)	r=25m	r=125m	r=250m	r=500m
0.5183	0.3241	0.0484	0.0197	0.0138	0.0097
0.3440	0.2170	0.0253	0.0098	0.0069	0.0048
0.2320	0.1472	0.0139	0.0050	0.0035	0.0025

Table G.6 Relative amplitudes, gravity base case, a=7.5m, reflection behind and to the side (θ=45°)

ak	r=7.5m (At base surface)	r=25m	r=125m	r=250m	r=500m
1.5547	0.4716	0.2623	0.1220	0.0867	0.0614
1.0320	0.4960	0.2124	0.0861	0.0601	0.0422
0.6960	0.4143	0.1509	0.0551	0.0379	0.0265

Conclusions arising from the analytical solution

- **Mono-piles**

It is clear that the mono-piles exert only a minor influence upon waves of period greater than 5s and that the disturbance to the wave pattern is small except close to the pile. Here, the relative amplitude of the scattered wave is in the order of 60% for θ = 180° (back reflection) but less for other directions, eg θ = 45°, where scattering is to the side and behind the structure it is around 32%. Over larger distance, eg 500m, the relative amplitude is far smaller at less than 5% for the shorter period waves in front of the pile, and <1% for the case of θ = 45°.

Figures G.1 and G.2 show the predicted total wave field that arises around the mono-piles for wave periods of 5 and 7s. The waves are entering the model from the bottom boundary and proceeding towards the top of the grid; the cylinder is located in the centre of the grid. The largest wave heights develop immediately in front of the structures due to reflection and the effects are less for the longer period wave condition.

- **Gravity bases**

As expected, gravity bases exert a greater influence upon the waves. For waves of period 10s and less there is appreciable scattering within a radius of order 100m. Closer to the gravity base reflected waves (θ=180°) vary between almost 80% at the base surface to of order 40% at a distance of 25m. For other angles, scattering is less, eg for θ=45° relative amplitudes are respectively of order 50% at the pile surface and 20% at a distance of 25m. Secondary interactions, such as those between waves scattered from adjacent structures could be as much as 15% of the incident wave height

Practical effects

The mono-piles seem likely to have rather little effect on waves, except close to the structure and for shorter waves (less than 5 seconds period). For gravity bases there will be somewhat stronger interactions, if it is assumed that the entire structure is surface piercing:

1. Increased spatial variability of the wave height within the wind farm area.
2. An increase in surface 'choppiness' within the wind farm area due to the interference of the incident waves and the scattered waves whose propagation directions would be different.
3. An increase in wave/current interaction effects in the immediate vicinity of the structures.
4. These, in turn, may enhance sediment re-suspension processes.

The Havelock solution is linear and the scattered waves decrease to zero at large values of *r*, so that no effects of scattering will remain at the coast. However, scattering and other interaction effects will result in some energy loss (by wave breaking) in the wind farm area, and the scattered energy will be lost from the incident wave field as it propagates towards the coast. The numerical modelling study gives an indication of how much energy will be lost by these processes. Note that 'blocking' (that is, the complete absence of waves in the lee of the structure) only occurs for values of *ak* considerably greater than those which would occur in a wind farm situation and is not therefore an important physical effect.

Comparison between the results of the 2D grid model and those from the analytical solution

The two-dimensional grid model is a Boussinesq solution, developed by DHI. It is capable of describing the effects of wave refraction, shoaling, diffraction and reflection. It has the advantage over the analytical solution previously described since it is able to model the non-linear interactions between waves scattered by an array of structures, of the type that would be used in a wind farm development. The model was first tested and compared against the analytical solution for a single cylinder, including a sensitivity test using model grid sizes of 5 and 1m.

Figure G.3 shows a comparison between wave heights predicted by the analytical solution and the two-dimensional grid model, along a line directly in front of the 5m-diameter structure for a wave period of 5s. The length of the incoming wave is approximately 30m and the peaks of the scattered wave height distribution should therefore occur at one-half the wavelength, or 15m, directly in front of the structure. The analytical solution follows this pattern exactly. The results from the 1m grid model also closely adhere to this result and agree well with the analytical solution with respect to maximum values, in the vicinity of the cylinder. The results from the 5m-grid model display more of a spatial shift with respect to the analytical model. However, this coarser grid solution still generates maximum wave height predictions that are close to the analytical solution near to the structure.

Figure G.4 shows a comparison between the analytical solution and the 5m grid model for a wave with a period of 7s. Again, whilst the grid model displays a spatial shift with respect to the analytical results, the maximum wave height values near to the cylinder agree well with the analytical predictions. The purpose of the present analysis is to provide estimates of the maximum wave heights that could develop close to the structures, since this is the prime consideration with respect to evaluating the potential for removal of foundation material by scour.

Figures G.5 and G.6 show the 5m-grid model predictions of total wave height distributions around the 5m-diameter cylinder for wave periods of 5 and 7s, respectively. The two figures have the same configuration as that for Figures G.1 and G.2. Figures G.5 and G.6 display an overall satisfactory comparison with the results of the analytical solution shown in Figures G.1 and G.2, both with respect to numerical distribution and the scattering pattern. The local wave model results show a slight tendency towards greater dissipation of the contours with remote distances from the structure, although the differences are considered to be negligible. In conclusion, the detailed local wave model reproduces the same general pattern of wave scattering as that determined from the analytical solution. The local wave model has therefore subsequently been adopted in sensitivity tests to predict the wave disturbance effects created in the near-field that may develop close to structures in a full wind farm array.

References (G3)

Longuet-Higgins & Cartwright (1957). The amplitude of waves reflected from a vertical circular cylinder. NIO Internal Report No A9.

Sarpkaya, T. and Isaacson, M. 1981. Mechanics of Wave Forces on Offshore Structures. Published by Van Nostrand Rheinhold.

Weigel, R.L. 1964. Oceanographical Engineering. Published by Prentice-Hall / Englewood Cliffs.

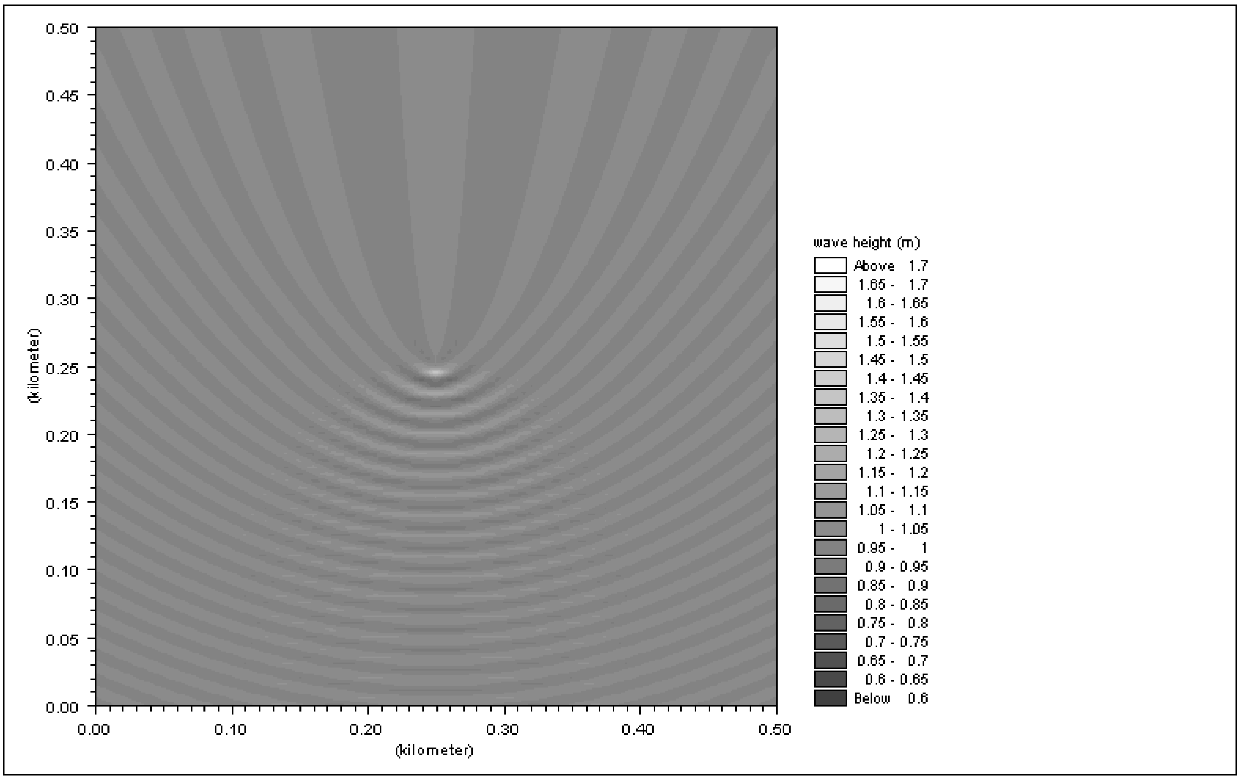

Figure G.1 – Wave height distribution predicted by the analytical solution for waves of 1m height and 5s period scattered by a 5m diameter cylinder in a water depth of 5m. Th e waves enter the model from the bottom boundary and the cylinder is in the centre of the domain.

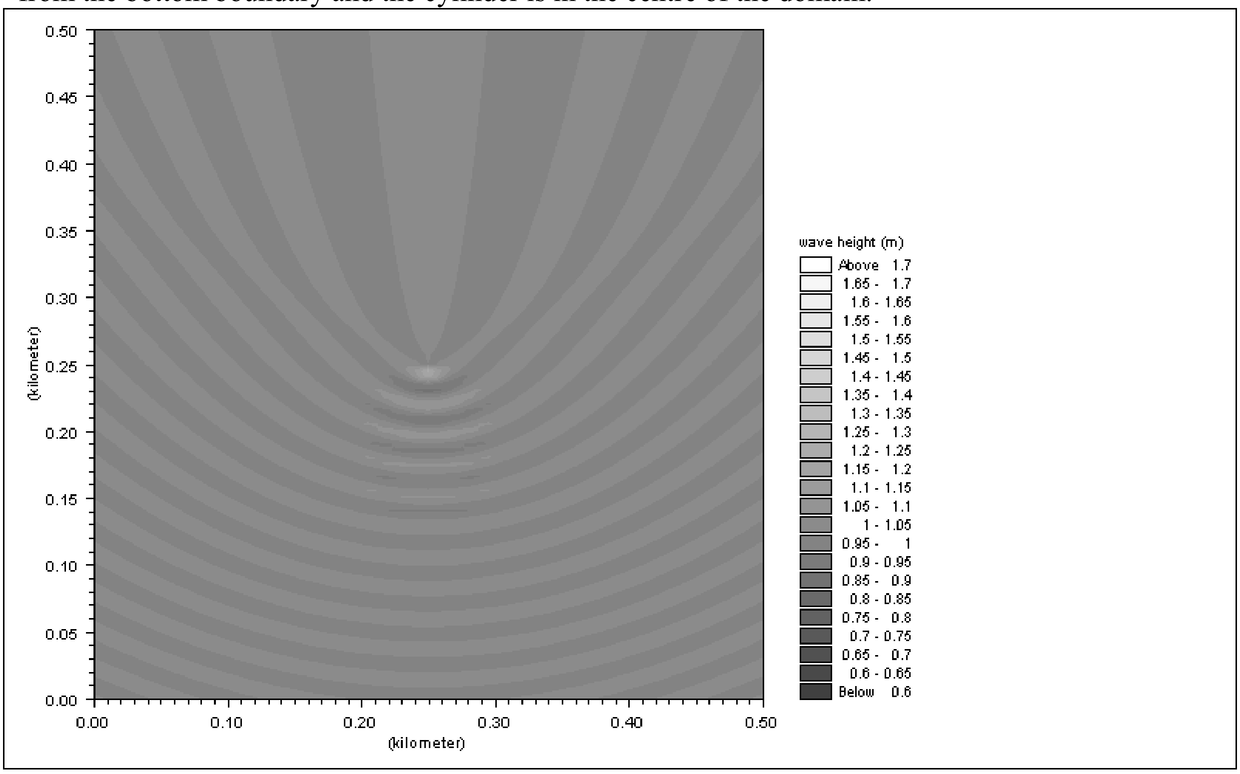

Figure G.2 – Wave height distribution predicted by the analytical solution for waves of 1m height and 7s period scattered by a 5m diameter cylinder in a water depth of 5m. The waves enter the model from the bottom boundary and the cylinder is in the centre of the domain.

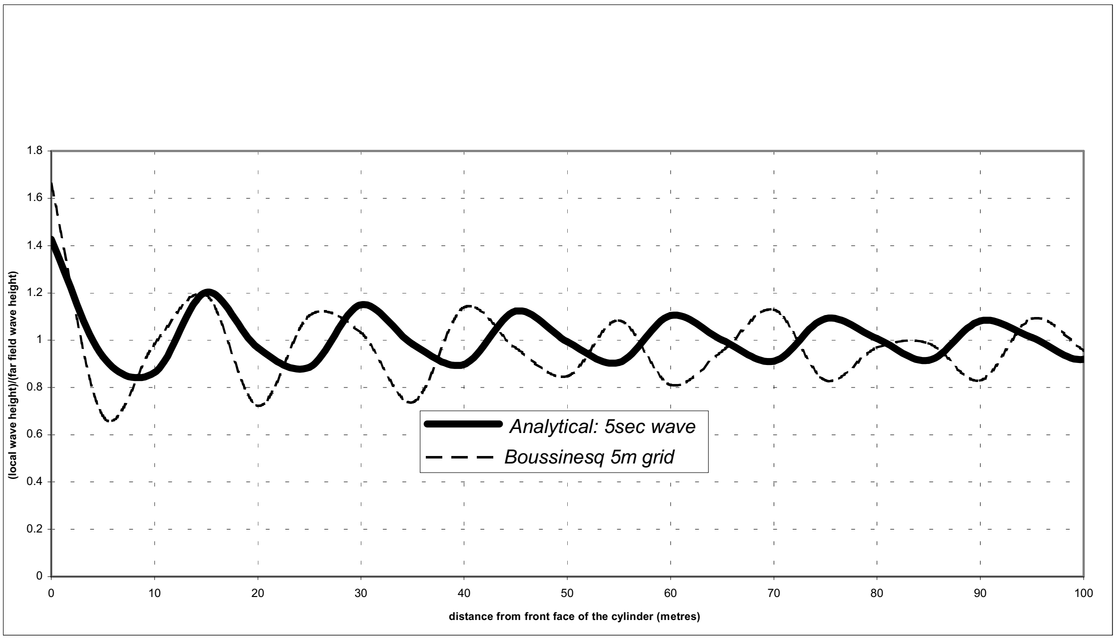

Figure G.3 - Comparison between analytical results and those obtained using a 2D grid model for scattering of waves with 5s period by a 5m diameter cylinder in a water depth of 5m

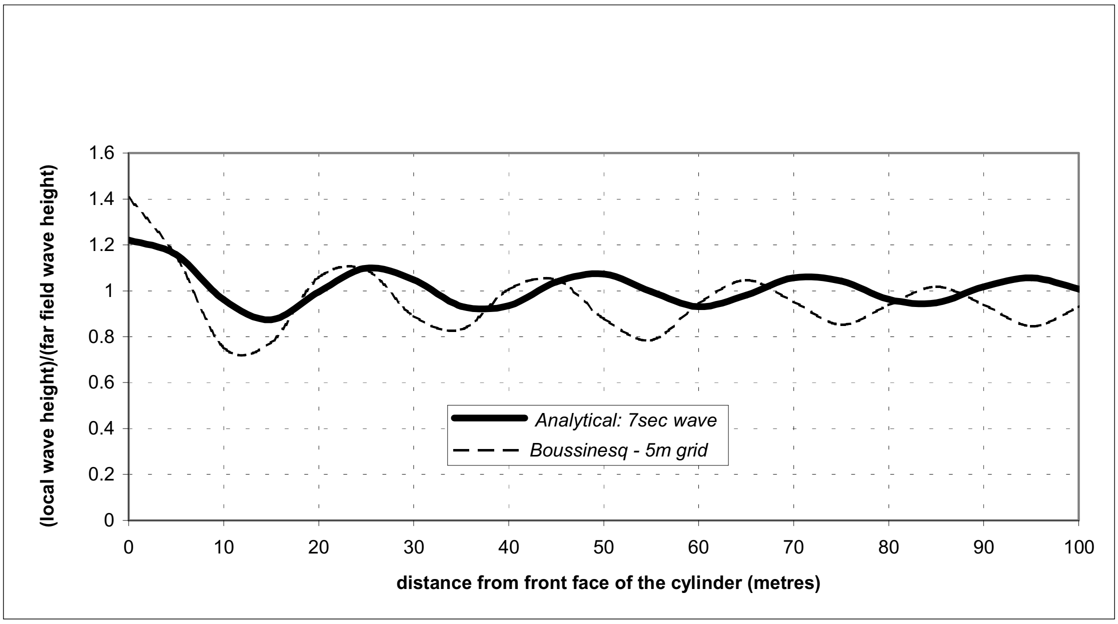

Figure G.4 - Comparison between analytical results and those obtained using a 2D grid model for scattering of waves with 7s period by a 5m diameter cylinder in a water depth of 5m

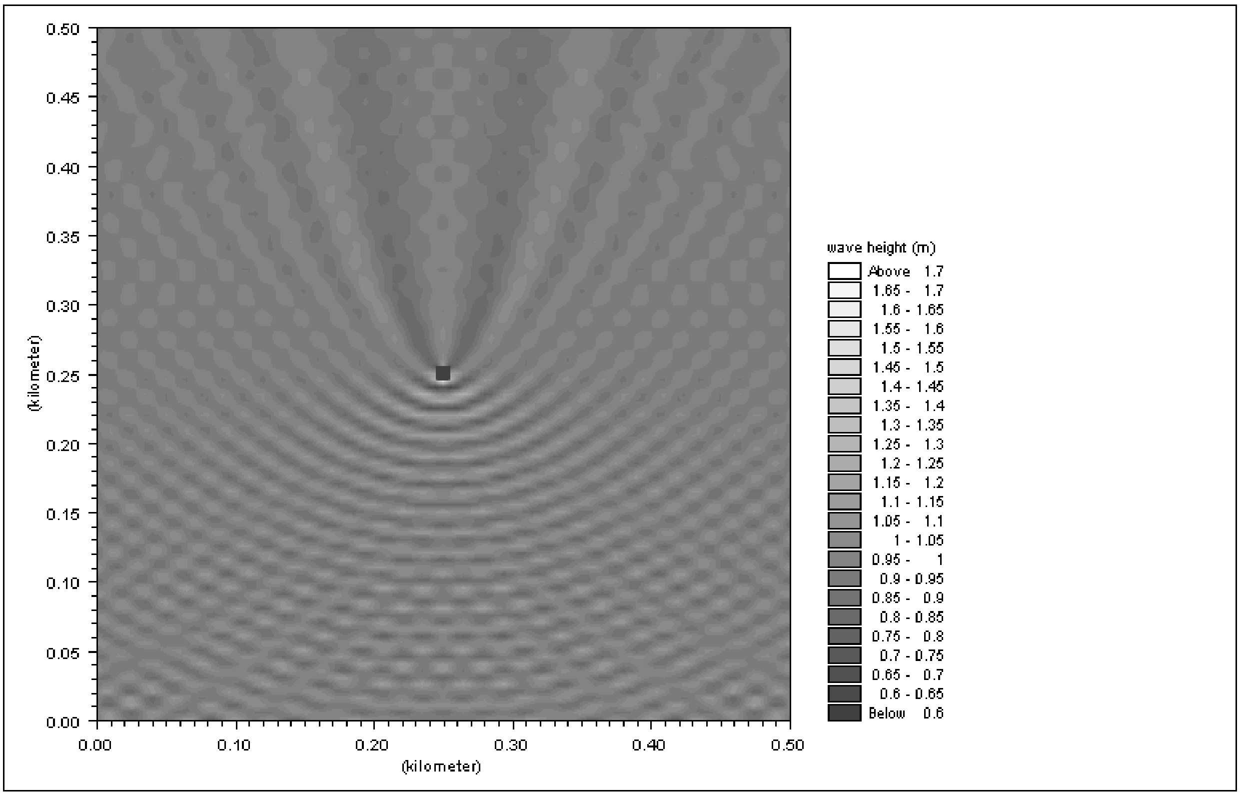

Figure G.5 – Wave height distribution predicted by the 2D numerical solution for waves of 1m height and 5s period scattered by a 5m diameter cylinder in a water depth of 5m.

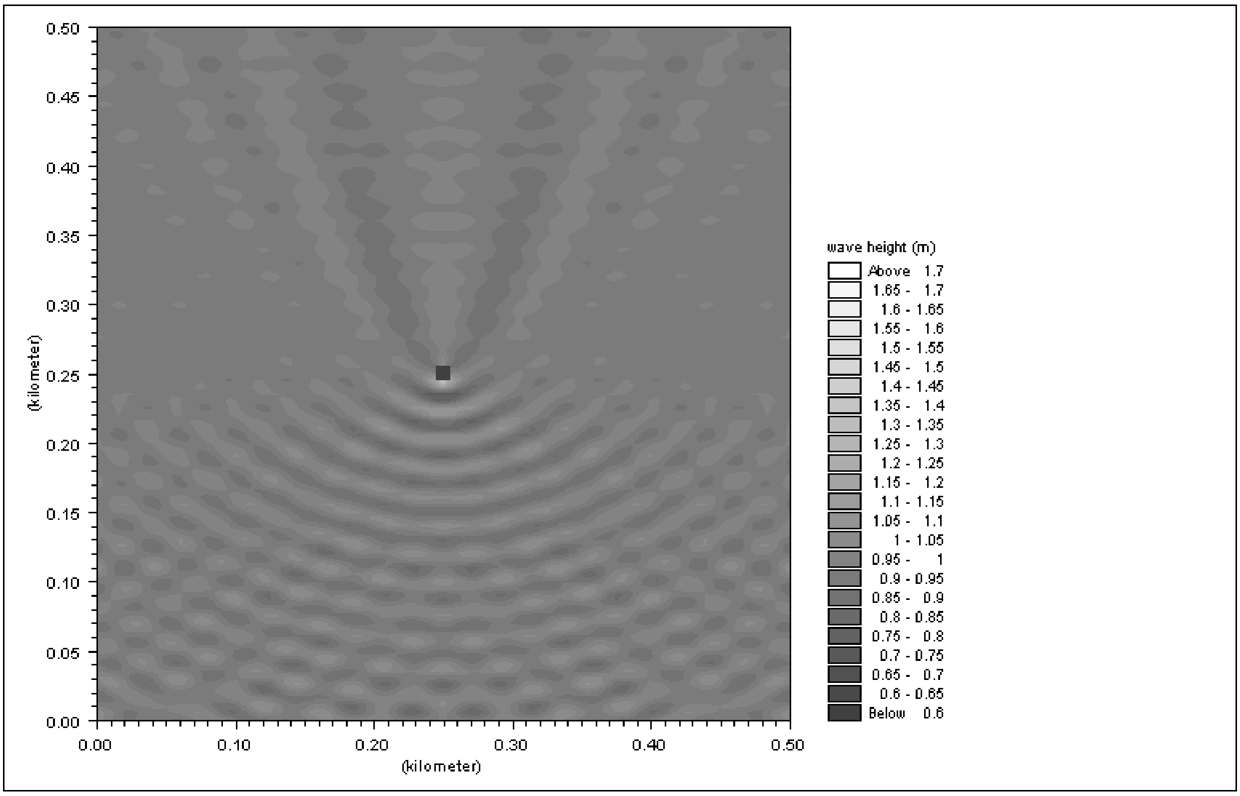

Figure G.6 – Wave height distribution predicted by the 2D numerical solution for waves of 1m height and 7s period scattered by a 5m diameter cylinder in a water depth of 5m.

G.4 Scour

State of present knowledge

To date the majority of research into scour around structures has focused on the effects due to steady unidirectional flows around bridge piers, and through the use of scaled experiments in laboratory flumes. Research into scour around vertical piles resulting from waves and tidal flows and supported by experiments in the field is far more limited. It is frequently the case that an empirical relationship between pile diameter (D) and equilibrium scour depth (S) is determined from this research. However, due to the lack of appropriate dynamic and kinematic scaling laws, these relationships may not be fully applicable at prototype scale, with a further problem being the absence of reliable field data to validate such formula. Furthermore, most relationships have developed for single piles, with very few studies considering effects of multiple pile arrangements.

It is also generally regarded that there are still no proven means to formulate mathematical models that can accurately represent the scour process and the geometry of the scour hole that develops under the influence of waves and currents (Bayram, 2000). As such, the coastal area models used in this study do not present a direct means of predicting scour, but can regarded as suitable to providing input parameters to applicable empirical relationships.

A comprehensive evaluation of scour in the coastal environment was recently provided by SCARCOAST (Scour Around Coastal Structures). This was a 4 year (1997 to 2000) research project funded by the European Union within the framework of the MAST (Marine Science and Technology) programme. The objective of the project was:

'To study the potential risk for scour in the vicinity of coastal structures, and to prepare and disseminate practical guidelines, to be developed from the research programme and also taking into account all "state-of-the-art" knowledge.'

The recent publications by Whitehouse (1998) and Sumer & Fredsoe (2002) were both supported by SCARCOAST, and provide general references for scour issues in the marine environment. A further valuable reference for scour assessments is presented by Hoffmans & Verheij (1997).

Scour Development

Structures that are placed in the marine environment have the potential to modify local coastal processes. A locally modified flow regime created by an obstruction can lead to changes in the local sediment transport capacity which then readjusts to achieve a new equilibrium through the development of scour. Scour at a structure can be considered to be *general scour* and *local scour*, each acting over different spatial and temporal scales. General scour typically occurs over longer time scales and acts over larger areas, such as the overall adjustment of the sea bed in response to a storm event. Local scour results from the immediate impact that a structure makes on the local coastal processes, such as a mono-pile structure modifying local flow patterns. The design process for placing such a structure on the sea bed has to consider the net effect of the combined scour process and provide appropriate scour protection measures to minimise the risk of failure in the foundation.

The development of scour in the marine environment is influenced by:

Geometry of the structure:	dimensions, shape and spacing of piles
Sediment characteristics:	sediment size, specific density
Coastal processes:	tidal flows, water depth and wave regime

The following general relationships for scour can be expected:

- Scour development is usually very rapid in the initial phase after construction, with a rate which decreases until an equilibrium scour depth is reached.
- Scour development due to current and wave action is faster than scour due a current alone.
- Scour effects created by waves only are small.
- Shallow water waves with currents may even reduce scour depths, whereas breaking waves (and long period swell) with currents tend to increase scour.
- The typical shape of scour development is an inverted cone.
- Scouring has to be considered in the design process, even if there is no sediment transport in the ambient condition.

Evaluation of methods

For single slender piles, the maximum scour depth resulting from combined waves and currents has been estimated to be 1.2D (Breusers, 1972), 1.4D (Palmer, 1969) to 1.5D (Bijker & de Bruyn, 1988) – the latter case determined this increases further with breaking waves, up to 1.9D. For the diameter of the scour hole a value of 6D has also been estimated from field observations (Palmer, 1969). Hence, for cylindrical piles with a spacing of <6D, interaction between piles is likely and the scouring depth will be greater than for an isolated pile, with *group* scour occurring.

The validity of the relationships developed for slender piles is limited to cases where wave diffraction effects can be ignored (ie the diffraction parameter, $D/L < 0.2$). With large diameter piles or large diameter gravity bases, where $D/L > 0.2$, the incident wave interacts with the large vertical cylinder to partly reflect a wave outward from the cylinder and partly diffract a wave around to the lee side. For very large diameter piles ($D/L \geq 1$), then wave reflection becomes dominant. Research on scour around large cylinders is again quite limited with one particular notable study by Rance (1980). With waves and currents from the same direction, Rance determined a scour relationship for scour depth of 0.064D, and extent of 0.75D, when the diffraction parameter, D/L > 0.1. It is also important to note that the scour depth for large cylinders does not scale directly with the pile diameter in the same fashion as that which develops for the slender pile case.

It is also generally accepted that the maximum scour depth around a pile due to the combined effects of waves and currents is larger than that which could be expected due to waves acting alone and by about 10% (Eadie & Herbich, 1986).

A 'universal' equation for relative scour depth was proposed by Eadie & Herbich (1986):

$$S/h = (1.941 \cdot 10^{-4}) \cdot (R \cdot M)^{0.654}$$

$S =$ scour depth

$h =$ water depth

$R = U_t D / \nu =$ Reynolds' Number in combined wave - current flow

$M = U_t/[g d_{50}(\rho_s - \rho)/\rho]^{0.5} =$ Sediment Number

$U_t =$ combined wave - current velocity

$D =$ diameter of the cylindrical pile

$\nu =$ kinematic viscosity

$\rho_s =$ sediment density

$\rho =$ water density

$d_{50} =$ median seabed sediment diameter

The difficulty with using this expression arises from the Reynolds' number (Re) that applied to their experiments and the extrapolation of this equation to values of Re at full scale. The largest Re value in their experiments was approximately 25000 – well inside the *sub-critical* flow regime. However, the Reynolds' numbers applying at full scale are likely to be up to 3 orders of magnitude larger. The full scale condition is situated well inside the *super-critical* flow regime and consequently the behaviour of water flow around the turbine support structures is likely to be very different from that which occurred in the small-scale experiments.

Consequently, the present study has used an alternative solution proposed by Johnston & Erasito (1994), which is believed to provide an upper-bound estimate for scour depths in combined waves and currents:

$$S = (2.2 \cdot 10^{-4}) \cdot (U_t D / \nu)^{0.619} \text{ , with } S \text{ expressed in metres}$$

<u>Example calculations for case study</u>

Whilst it is clear that scour will occur around the turbine support structures under a design storm scenario, the purpose here is to develop a picture of the magnitude of stable scour which could occur under average long-term wave and current conditions.

The maximum wave height predicted due to reflection in front of the support structures located on the inshore sandbank was used to provide a sample prediction of possible equilibrium scour depths. According to the output from the near-field wave model, this wave height was 1.86 times the offshore wave height when the peak period of the spectrum was 6.5s. If the 50% annual exceedence wave height of 1m were applied, then the height immediately in front of any support structure would be 1.86m, under this sea condition. With these conditions a wavelength (L) of 42m is obtained, hence the diffraction parameter (D/L) = 0.12 (ie < 0.2).

Taking a representative tidal current velocity of 1.2m/s provides a combined wave-current velocity of 2.2m/s in 5m water depth.

Using the solution proposed by Johnston & Erasito, a scour depth of 5.1m is obtained for a 5m diameter pile (around 1D) for this set of input parameters. This may increase to a scour depth of 5.5m (around 1.1D), using the 10% annual exceedence wave height, and potentially larger scour depths for design wave conditions.

<u>Further examples</u>

Further estimates of scour from comparable marine installations is provided by the following examples:

a. Westhinder Platform, Belgium

Off the Belgian Coast, the design considerations for installing the Westhinder Platform (MOW7) estimated a potential scour depth of between 4 to 6.3m without scour protection, based on methods proposed by Herbich (1984), Wang (1983) and Breusers (1972). Scour protection has been installed at the base of the mono-pile to a diameter of 22m, consisting of filter layers of gravels and sands, and the site is monitored regularly. The platform itself is supported by a 2m diameter mono-pile erected on Westhinder Bank, a large and relatively stable sandbank formed of medium sands and a cover of sand waves. The following design criteria were taken into consideration (De Wolf, et al. 1994):

Pile diameter:	2m mono-pile
Service life:	30 years
Probability of exceedence	10% during service life
Wave regime:	depth-limited with breaking waves
Design wave height (Hmax)	11.30m
Wave period:	7 seconds
Design water level:	8.90m (MSL)
Current velocity:	1.29m/s
Sediment cover:	Medium sands (D_{50} = 0.280mm)
Scour:	4 to 6.3m depth, 22m extent (estimated)

b. Horns Rev, Denmark

For the Horns Rev wind farm the issue of scour has been considered in relation to both mono-pile and gravity type foundations. Across the development site water depths are between 5.8 to 17.5m. The site is associated with a mobile sediment cover of medium-fine sand which has reduced in level by 1.5m from 1876 to the present day. Along with this reduction, sand waves with a typical crest height of 0.5m cover the area. An overall allowance of ±2m is provided for the natural morphological variation in sea bed levels. The scheme intends to install up to 80 turbines spaced 560m apart, using 4m diameter steel mono-piles driven 25m into the sea bed. Placing a mono-pile foundation in these conditions is expected to lead to local scour soon after installation. The maximum depth for the scour hole is estimated to be around 1.9 times the foundation diameter, at a value of 6.7m. With a bed level that has a natural variation of ±2m, there is a potential for a total loss of 8.7m local to the support structure. Such a loss will have implication to the stability of the mono-pile foundation and recommendations are put forward for a filter protection layer (gravels, pebbles and stones) in the design. In the case of an unprotected gravity support foundation it is estimated that a fully developed scour hole will also develop rapidly and within the first year of installation. This has the potential to undermine the structure unless a scour protection layer is included in the design. In both cases it is considered that scour effects remain local to individual foundations and general scour (sheet scour) is not expected to develop across the development site (DHI, 1999). A monitoring programme for metocean parameters has supported the design calculation for extreme wave and tidal events (Neckelmann, 2001).

Pile diameter:	80 * 4m mono-piles
Service life:	50 years
Probability of exceedence	1:100 years
Wave regime:	depth-limited with breaking waves
Design wave height (Hmax)	8.1m (south-west corner)
Design wave height (Hs)	5.4m
Wave period:	8s
Design water level:	5.8 to 17.5 (-2.7m / +3.7m)
Current velocity:	<1.0m/s (0.92m/s)
Sediment cover	Medium sands (D_{50} = 0.3mm)
Scour:	6.7m estimated

c. Christchurch Bay Tower

The Christchurch Bay Tower was a small offshore structure with a gravity foundation, sited off the South Coast in a water depth of 9m. The location in Christchurch Bay has the longest fetch in the south-west sector facing the Atlantic. From this direction long period waves up to 15m in height and 12 second period can enter the bay. However, the shallow water depth at the tower of 8.7m (MSL) leads to attenuation of larger waves by shoaling. Thus, maximum wave heights at 7 to 8m are limited to high waters and surge events (HSE, 1998). The purpose of the tower was to study wave forces and the behaviour of a gravity foundation, with the last data collected in 1987. The first tower suffered considerable problems with scour causing foundation failures which rapidly destabilised the structure. This was then replaced with a second tower using more conservative design estimates and on a thinner layer of sand, and including a skirt around the base to penetrate the sand layer into an underlying layer of clay. For both towers, scour occurred over a large area all round the structure (between 12 to 20m from the base), forming a shallow saucer-like depression, with more concentrated erosion at the edge of the gravity foundation. In both cases this progressed to the underside of the base with a consequent risk of undermining, and in the case of the first tower failure of the foundation (Bishop 1981).

Pile diameter:	2.8m with a gravity base of 10.5m diameter
Service life:	-
Probability of exceedence	-
Wave regime:	depth-limited waves
Design wave height (Hmax)	-
Design wave height (Hs)	7 to 8m
Wave period:	-
Design water level:	8.7m (MSL)
Current velocity:	0.8m/s
Sediment cover	fine sand, overlying clay
Scour:	0.5 to 1m around periphery of base, extending a distance of between 12 to 20m from the base (observed)

d. Scroby Sands

Scour estimates for the Sarah Jane project referred to empirical methods for waves and currents proposed by Johnston & Erasito (1994). Using this approach an estimated equilibrium scour depth of 8.43m was calculated for a 3m pile in response to extreme (1:50 year) wave conditions. Data from a 1m diameter test pile located on the north-east side of Scroby Sands suggested that a summertime equilibrium scour depth was in the order of 1D (Halcrow, 1996).

Pile diameter:	3m
Service life:	30 years
Probability of exceedence	1:50 years
Wave regime:	depth limited waves
Design wave height (Hmax)	-
Design wave height (Hs)	5.5m
Wave period:	8.6 seconds
Design water level:	2.93m (CD)
Current velocity:	1.65m/s tide plus 1.2m/s wave induced = 2.85m/s
Sediment cover	medium sands, with sandy gravels
Scour:	8.43m equilibrium depth (estimated)

References (G.4)

Bayram A. & Larson M. 2000. Analysis of scour around a gropu of vertical piles in the field. Journal of Waterway, Port, Coastal, and Ocean Engineering. July/August 2000.

Bijker E.W. & de Bruyn C.A. 1988. Erosion around a pile due to current and breaking waves. Proceedings of 21st International Conference on Coastal Engineering. Vol. 2.

Bishop J.R. 1981. Experience with Scour at the Christchurch Bay Tower. National Maritime Institute R.120.

Breusers H.N.C. 1972. Local scour near offshore structures. Delft Hydraulics Laboratory. Publication No. 168.

Danish Hydraulic Institute. 1999. *Horns Rev Wind Power Plant*. Environmental impact assessment of hydrography. Baggrundsrapport nr. 8. Project no. 50396-01.

De Wolf P., Van Den Bergh P., Thomas R., Lanckzwerit M. & Hyde P. 1994. Measuring Platforms in the North Sea in Belgian Territorial Waters and on the Belgian Continental Shelf. Measuring Platforms in Front of the Flemish Coast. PIANC Bulletin No. 82.

Eadie R.W. & Herbich J.B. 1986. Scour about a single, cylindrical pile due to combined random waves and a currents. Proc. 20th international conference on coastal engineering, ASCE, Nov 9-14; pp 1858-1870.

Halcrow. 1996. Wave Climate / Scour Study – Sarah Jane Offshore Wind Monitoring Station. Study Report. Powergen PLC.

Health & Safety Executive. 1998. Christchurch Bay Tower Data Archive – Principal Report. Offshore Technology Report – OTO 97 060.

Herbich J., Schiller R., Watanabe R. & Dunlap W. 1984. Seafloor scour-design guidelines for ocean founded structures.

Hoffmans G.J.C.M. & Verheij H.J. 1997. *Scour Manual*. Published by A.A. Balkema. ISBN 90 5410 673 5.

Johnston A.J. & Erasito S. 1994. Scour in the vicinity of piles. Proc. International Conference on Hydraulics in Civil Engineering. Brisbane, pp 315-320.

Neckelmann S. & Petersen J. 2001. Evaluation of the stand-alone wind and wave measurement systems for the Horns Rev 150MW Offshore Wind Farm in Denmark. ELSAMPROJEKT A/S, Kraftværksvej 53, DK-7000 Fredericia.

Palmer H.D. 1969. Wave-induced scour on the sea floor. Proc. Civ. Eng. in the Ocean II, Miami Beach.

Rance P.C. 1980. The potential for scour around large objects. *Scour prevention techniques around offshore structures.* Society for Underwater Technology, London.

Sumer B.M. & Fredsøe J. 2001. Wave scour around a large vertical circular cylinder. Journal of Waterway, Port, Coastal, and Ocean Engineering. May/June 2001.

Sumer B.M. & Fredsøe J. 2002. The mechanics of scour in the marine environment. Advanced series in Ocean Engineering - Volume 17.

Wang R.K. & Herbich J. 1983. Combined current and wave-produced scour around a single pile. COE Report No. 269 – Dept. of Civil Engineering Texas, Engineering Experiment Station.

Whitehouse R. 1998. Scour at Marine Structures – A Manual for practical applications. Published by Thomas Telford Ltd.