Appendix E4 – Foundation / Substructure Assessments

Seagreen Wind Energy

July 2012

9V5268

CONTENTS

Tables

Figures - Part I

Figures - Part II

Part I - Scour Assessment

1 INTRODUCTION

The presence of any foundation / substructure in the marine environment will cause changes locally to both the flow regime and the passage of waves. These changes have the potential to result in scour of the sea bed locally around the base of the foundations / substructures under certain threshold conditions and in particular types of sea bed strata.

The formation of a scour hole around a foundation / substructure can potentially cause instability in the structure and therefore engineering design is undertaken to ensure that either scour protection is provided or suitable foundation / substructure types are selected to remain structurally sound under the range of tidal and wave loading conditions and sea bed changes that may be anticipated during the life of the OWF development.

However, any residual scour that does occur can have two potential impacts in respect of environmental issues, namely:

- a scour hole or pit will be created which will occupy a proportion of the sea bed as a ‘scour footprint’; and
- any material scoured from the sea bed is likely to become entrained within a sediment plume, subsequently transported further afield by tidal currents, and ultimately deposited back on the sea bed.

To assess these issues the following approach has been adopted:

- Existing literature, including papers published in scientific and professional journals, technical design guides and OWF Environmental Statements, has been extensively reviewed to identify scour processes around different foundation / substructure types of monopiles, piled jackets, piled tripods, caissons, rectangular / square (a.k.a. cross beam or flat bed) gravity base structures (GBS) and conical GBS.

  This has enabled assessment of:

  (i) formulae used to predict scour formation in the marine environment under conditions imposed by tidal currents alone, waves alone and combined currents and waves;
  (ii) scour assessments applied in other OWF Environmental Statements for different foundation / substructure types;
  (iii) physical laboratory scale tests of scour around conical GBS, rectangular / square GBS, piled tripods and piled jackets; and
  (iv) field evidence of scour formation around different foundation / substructure types, particularly drawing from post-installation monitoring of scour around monopiles at Scroby Sands, conical GBS at Thornton Bank in Belgium, and piled tripods at Alpha Ventus in Germany.

- Scour prediction methods have been formulated, using first-principle developments from the existing published methods, for foundation /substructure types of piled jackets, piled tripods, rectangular / square GBS and conical GBS.
- The scour prediction methods have been verified against published field observations from Thornton Bank (for conical GBS), Alpha Ventus (for piled tripods), 1/36 scale physical laboratory tests (for piled jackets) and a 1m x 1m physical model (for rectangular / square GBS).

- The scour prediction methods have been used to determine the predicted scour depths, scour footprint areas and scour volumes around different foundation / substructure types using typical 'worst case' dimensions expected to be considered for use at the Firth of Forth OWF for Phase 1 Projects Alpha and Bravo. These structural dimensions were defined by the Concept Engineering Study (CES) (GL-Garrad Hassan, July 2011) and the Construction Methods Statement (CMS) (Seagreen, February 2012). Physical parameters used to define the tidal currents, water depths, wave height and period, and sea bed sediment were defined by datasets arising from the project-specific metocean, geophysical and benthic (grab sample) surveys.

  [**Note**: Since production of the CES and CMS further development of foundation / substructure options occurred and a Rochdale envelope was produced. The worst case foundation / substructure types for assessment of effects on the physical environment have been included in these scour assessments, namely conical GBS for 50m water depth/average soils and conical GBS for 60m water depth/weak soils for Wind Turbine Generators (WTG) and meteorological masts (met masts) and rectangular / square GBS for Offshore Platforms (OSP). Some other foundation / substructure types assessed in this Appendix were later superseded, but remain included here for purposes of completeness and general comparison between foundation / substructure types.]

- Based on the results from these scour prediction methods, expert-based assessments have then been made within the ES of: (i) the significance of the scour volumes; (ii) the use of scour protection to mitigate scour impacts; (iii) the presence of sensitive receptors; and (iv) the fate of scour material based on knowledge of tidal flow patterns and tidal excursion distances. In accordance with Appendix D1 containing the agreed Position Paper on Coastal and Sea Bed Impact Assessment (Seagreen, 2010) and its Further Evidence Base (Seagreen 2011), the assessment has been stopped at that point based on findings of no significant adverse effect, without the need for numerical modelling of the fate of scour material or the deposition of sediments on the sea bed.

[<u>Important Note</u>: The scour assessments use conservative approaches and therefore probably over-estimate the likely actual scour that would be developed. This is a suitable approach for purposes of EIA, but these assessments are not intended for use in structural design.]

2 SCOUR DEPTHS AND VOLUMES

2.1 Background

The scour depths and volumes have been calculated under the action of currents alone, waves alone and combined waves and currents for each foundation / substructure type considered. The parameters that lead to the greatest scour depths and volumes have then been used as the worst case value in the subsequent assessments. To adopt a 'worst case' scenario in the EIA it has been assumed that no scour protection will be provided under any of the foundation / substructure types considered. A 'realistic worse case' scenario is that scour protection actually will be provided in accordance with the CMS. It is especially the case that it is highly likely that scour protection will be provided for both types of GBS.

The foundation / substructure types and dimensions considered in these assessments are presented in Table 1.

Table 1 - Summary of principle structural dimensions considered in the scour assessment

Structure Type	Structural dimensions
Conical GBS for WTG and met mast **Rochdale worst case** for 60m water depth / weak soils	Base of Cone 35.4m diameter
	Baseplate 72m diameter (octagonal) x 2.8m high at the outer edge, 4.2m high at the cone intersection plane
Conical GBS for WTG and met mast **Rochdale worst case** for 50m water depth / average soils	Base of Cone 28.4m diameter
	Base Plate 52m diameter (octagonal) x 2m high at the outer edge, 3.0m high at the cone intersection plane
Rectangular GBS **Rochdale worst case** for OSP at up to 1 location in Project Alpha	100m x 75m rectangular base plate 7.5m thick, with six square columns up to 15m x 15m
Square GBS **Rochdale worst case** for OSP at up to 2 locations in each of Project Alpha and Project Bravo	40m x 40m square base plate 7.5m thick, with four square columns up to 7.5m x 7.5m
Piled Jacket Structure 50m water depth	Primary piles 2.1m diameter, 4 off at 32m apart
	Bracing 0.66m diameter at 2.5m above bed
Piled Steel Tripod 50m water depth	Lower bracing members 3m (assumed)
	Pile diameters 3m (assumed)
	Base of tower 3m above seabed (assumed)

Assessment of scour hold development has been undertaken for both the 1 in 1 year and the 1 in 50 year event conditions. The GL-Garrad Hassan report on the Structural Basis of Design provides estimates of wave and current parameters under these conditions (summarised in Table 2).

Table 2– Summary of typical return period input data of wave and current – water depth 50 metres

Return Period (years)	H_s (m)	T_p (s)	U_c (m/s)	U_w (m/s)
1	6.7	11.0	1.21	0.68
50	8.7	13.0	1.38	1.17

Nomenclature: H_s significant wave height; T_p peak wave period; U_c current velocity; U_w peak value of wave-induced water particle velocity at the seabed

Notes: H_s and U_c are taken directly from the GL-Garrad Hassan report tables and T_p has been estimated from the range of wave periods quoted in their tables.

2.2 Piled Jacket

The dimensions of the jacket structure in 50m water depth have been taken from drawing file number *108694-BD-011-RevA-Jacket 50m 6MW* as presented in the CES. The assumed configuration is as follows:

- The principal components from the scour perspective are the four corner piles of the jacket, which are of 2.1m diameter, situated a distance of 32m apart. The distance between the four piles is quite considerable, making it unlikely that the scour holes attached to individual piles will be able to interact.
- The secondary structural members that could invoke scour are the horizontal bracing elements, which are of 0.66m diameter and are located at a distance of around 2.5m above seabed level. Sumer and Fredsϕe (2002) provided an empirical method for predicting the scour depth beneath horizontal cylinders located at an arbitrary elevation above the seabed
- To undertake predictions of the equilibrium scour depths around the piles, the methods proposed by Harris et al (2010) have been applied. For the purpose of the 'worst case' assessment in the EIA, it is assumed that no scour protection has been provided to the piles.
- The CMS states that the operator will assess the need for scour protection on the basis of individual on-site experience as the installation of the wind farm proceeds. It is acknowledged that some scour may occur during the elapsed time between the installation of the structure and the provision of scour protection. Scour under waves tends to progress on a shorter time scale to equilibrium than that attributable to currents. If allowed to progress, scour around the main piles can affect the natural frequency of the jacket structure, leading to a possible increased fatigue risk over a long period of time, unless such scour is allowed for in the design from the outset.

The working procedure appears to be not to place scour protection for jackets, for practical access reasons. The Concept Engineering Study states in Table 8-1 that no scour protection costs were allowed for in the cost input, because it is not feasible to post-install scour protection to jackets.

2.3 Piled Tripod

It is believed that the German offshore wind farm Alpha Ventus, located in the North Sea, is the only project to date where the tripod structural solution has been adopted. The water depth at Alpha Ventus is reported to be 30m; however, the tripod structure is intended for use in water of intermediate depths and therefore presumably it could be appropriate to the Firth of Forth. The diameter of the main vertical column of the Alpha Ventus tripod structure is 6m; Figure 1 shows a photograph of the tripods under construction.

Figure 1 – Tripod support structures for Alpha Ventus under construction
Source: Kaiser and Snyder (2012)

For the purposes of the present scour assessment, it has been assumed that the three corner piles of the tripod are of 3m diameter. It has been assumed that the diameter of the three supporting lower cross members is also 3m. These dimensions have been adopted for consideration of scour predictions and are likely to be subject to change in a detailed design.

Due to the limited application of the tripod design to date, it can be regarded as an innovative structural approach. On the other hand, the design and construction of steel jackets is well understood and is standard practice, with presumably, a lower level of project risk. As far as scour observations are concerned, Stahlmann and Schlurmann (2010) reported on the results of a series of physical model tests of the tripods for Alpha Ventus, at scales of 1:40 and 1:12. The scour hole that developed under wave loading was considerable – of the order of 1.1 cylinder diameters in depth and with a considerable extent of footprint, according to the photographs provided by Stahlmann and Schlurmann (2010). It is again believed to be impractical to place scour protection around a tripod support structure and Table 8-1 of the CES states the same viewpoint.

2.4 Rectangular / Square GBS

The rectangular / square GBS has a deep hollow base, which is ballasted to assist stability and has the advantage over the conical GBS in that the wave loading is less. On the other hand, the design of this GBS to resist the applied bending moments due to wave and wind loading could be onerous, since they transmit shear forces at the interface with the main base slab, in order to carry the flexure.

- The dimensions required for undertaking a scour assessment of this structure are the diameter and height of the base and these have been presented as worst case dimensions of 100m x 75m rectangular baseplate (7.5m thick) in the Rochdale Envelope for OSP at up to 1 location within Project Alpha and 40m x 40m square baseplate (7.5m thick) for OSP at up to 2 locations within each of Project Alpha and Project Bravo. The water depth considered in all cases is 50m.
- The prediction of scour around a rectangular / square GBS is amenable to treatment using the methods developed by Bos et al (2002[a]).

2.5 Conical GBS

For the conical Gravity Base Structure (conical GBS), two realistic scour scenarios are possible:

- The temporary works condition, when either the backfill, or later the filter, have been placed, but the final scour protection blanket is not yet in position; and
- The in-service situation, when the scour protection is fully provided and is in operation.

The duration of the temporary construction condition at Thornton Bank was of the order of 6 to 8 weeks, as discussed by Bolle et al (2009, 2010) and this scenario is a risk situation as far as sediment release is concerned, since the finer materials of the backfill and the filter are likely to be susceptible to scour and possibly transport by waves and currents.

The EIA, however, initially considers a worst case of no scour protection being provided, with scour protection then used as a potential (or in the case of GBS highly likely) mitigation measure.

The details assumed for the conical GBS are as follows:

- Conical GBS in 50m water depth, using dimensions provided in the Rochdale Envelope. The wave action on such a structure is a major design driver, especially in these water depths. The root diameter of the conical flask is 28.4m, whilst that of the octagonal base plate is 52m. Applied wave bending moments at the root of the cone will be transmitted by vertical forces acting around the diameter of the shell, transmitted to the base plate, which will resist these forces by flexure. This approach produces a base plate diameter that is substantially greater than that of the cone base, and this result is probably also beneficial for long-term scour mitigation.

- Conical GBS in 60m water depth, using dimensions provided in the Rochdale Envelope. The root diameter of the conical flask is 35.4m, whilst that of the octagonal base plate is 72m.
- The large diameter of the base plate of the conical GBS may be effective in helping to reduce the scouring influences generated by downward-acting local currents caused by the presence of the conical structure. On the other hand, the thickness of the base plate itself, unless it is buried, could also generate scour. Potentially seabed preparation works involving dredging to up to 5m could be required, enabling the baseplate to be buried.
- For the conical GBS, two approaches to predicting scour have been considered:
    - o Apply the methods proposed by Khalfin (1983, 2007) and Khalfin et al (1988) for scour by waves and currents, as appropriate, using the base diameter of the cone as the characteristic cylinder diameter
    - o For the base plate, use the scour prediction approaches developed by Bos et al (2002[a,b]) for a submerged GBS

3 SCOUR ASSESSMENT METHODS

3.1 Footprint of the scour hole

The shape of the scour hole, at least under the action of a steady current, is elliptical in plan form, and this was shown in photographic evidence by Eadie and Herbich (1986), and was reported again by Sumer and Fredsϕe (2002). Later, Bolle et al (2010) reported on a 1:52 scale physical model test undertaken for scour around the conical GBS of Thornton Bank, and again showed a photograph displaying an elliptical footprint. Harris et al (2010) suggested a form for the elliptical footprint of a scour hole, which seems to match the experimental evidence well:

- On the upstream side, the slope of the scour hole is equal to the angle of repose ϕ
- However, on the *downstream* side, the slope is around *one half* of the angle of repose ϕ, ±2°
- On the side slopes of the scour hole, they recommend an assumed angle of around $5/6\phi$

The angle of friction for the Holocene deposits is 25°-30°, for design applications. Largo Bay (part of Forth Formation) is clays and soft to firm silty clays. St Andrew's Bay (second part of Forth Formation) is fine to coarse sands – angle of friction 25°-30°. Wee Bankie is quoted as sandy-gravelly clay. Marr Bank is fine sand - angle of friction 28°-35°. The present calculations are based upon an angle of friction of 28°.

In respect of wave action around the structures, due to the limited availability of evidence, an elliptical scour footprint has also been assumed, because the asymmetry in the seabed water particle velocities generated by the passage of the wave could preferentially distort the form of the scour hole in a direction that is collinear with the motion of the wave.

At Scarweather, on the upstream side of the structure, the observed slope of the scour hole was ~29° and on the downstream side it was more like 14°; this finding accords with the recommendations of Harris et al (2010). Figure 2 shows the experimental result of equilibrium scour around a monopile reported by Eadie and Herbich (1986), in which the disparate slopes on the upstream and downstream sides are clearly visible.

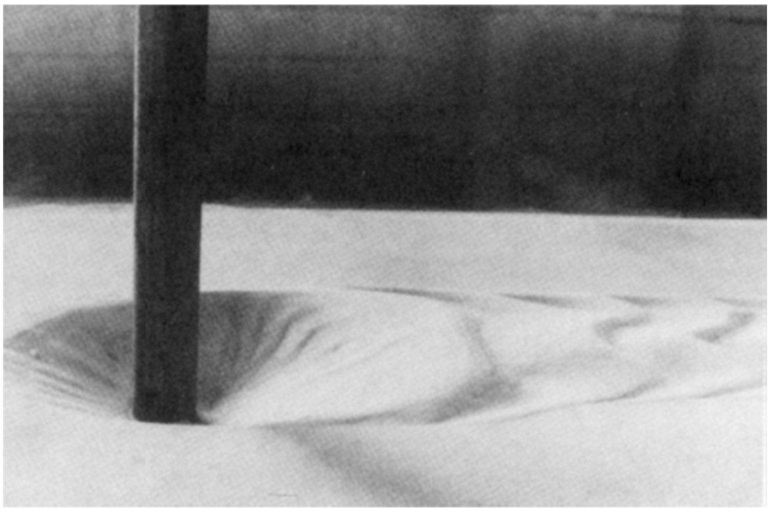


Figure 2 - Scour hole around a monopile – note the steeper gradient on the upstream side
Source – Eadie and Herbich (1986)

3.2 Values of median sediment diameter

Figure 3 shows the cumulative probability distribution of the median sediment diameter D_{50} obtained from the analysis of the 103 samples that were taken at the site. The most probable value of D_{50} is around 365µm, although it is noted that the methods for the prediction of equilibrium scour depth are not greatly sensitive to the value of D_{50} when live bed scour is the dominant process. For the purpose of the study, a D_{50} value of 365µm has been adopted.

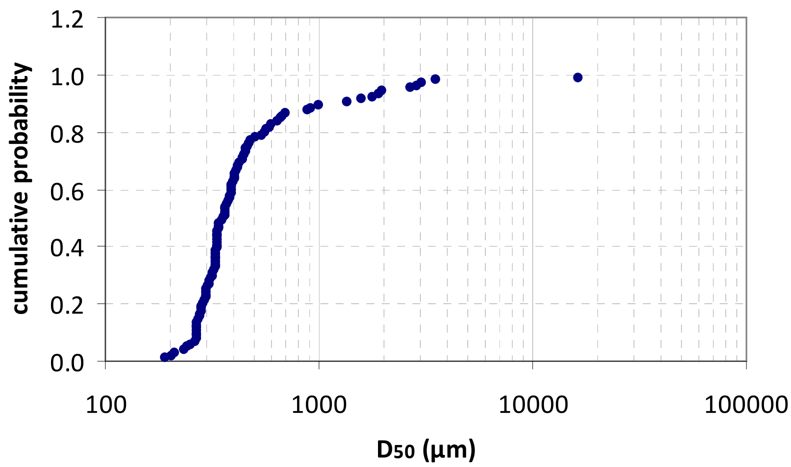


Figure 3 – Cumulative probability distribution of the median sediment diameter D_{50} based upon data reported from the site investigation

3.3 Prediction of scour around the GBS designs

It was found that the method proposed by Khalfin (2007) for the prediction of scour around large diameter cylinders in waves, successfully reproduced the scour reported by Bolle et al (2009, 2010) around the conical GBS of Thornton Bank, during the construction phases when the backfill and later the filter were temporarily exposed to wave and tidal action. Likewise, the solution developed by Khalfin (1983) and subsequently modified by Bos et al (2002[b]), for the prediction of scour around large cylinders in currents, provided results that showed a satisfactory level of agreement against observations made at Thornton Bank.

For predicting the equilibrium scour depth around the conical GBS in waves and currents combined, the method developed by Soulsby and Clarke (2002) was first used to predict the peak value of seabed shear stress under the joint action. This output was then used to modify the value of the applied seabed friction velocity in the Khalfin (2007) solution.

Under certain circumstances, it might be possible to also use the method proposed by Bos et al (2002[a]) for predicting scour around caissons due to the combined action of waves and currents. However, there is a difficulty with establishing the height of the caisson, for a conical GBS. If the cone stops well below the free surface and is topped by a uniform cylinder, then the height of the cone could be used. This approach produced a satisfactory application of the Bos et al (2002[a]) method for Thornton Bank. However, the cone of the conical GBS in the present project design extends to the mean water level, rendering it even more speculative to apply the Bos et al solution. Scour around the base plate alone, under the joint action of currents and waves, is directly amenable to treatment using the method due to Bos et al (2002[a]).

Yeow and Cheng (2003) reported on the results of a series of wave tank scour tests conducted on a model cylinder situated on top of a caisson. Figure 4, taken from their paper, shows a typical configuration. Even when the top of the caisson was level with the seabed, Yeow and Cheng (2003) found that scouring occurred on the up-wave face of the caisson. They explained this phenomenon in terms of the formation and separation of a horseshoe vortex, induced by the upper cylinder. The vortex caused separation to occur around the edge of the caisson, thus inducing scour around the circumference, even when the exposed height of the caisson was zero. The effect was particularly prevalent when the diameter of the upper cylinder was around one half of that of the caisson – as in the case of the conical GBS designs being considered here. The upper cylinder initiates the scouring mechanism, which is then transmitted to the face of the lower caisson, by first flowing along the top of the structure. For this reason, the equilibrium scour depth for the conical GBS was predicted using the base diameter of the cone as the structural diameter. However, it was then assumed that the predicted scour depth was applied around the periphery of the base plate. This approach conforms in principle with the observations made by Yeow and Cheng (2003).

On the other hand, if the top of the caisson is significantly higher, as in the case of the rectangular / square GBS, then Yeow and Cheng (2003) found that the caisson itself controlled the scouring process, and the influence of the upper cylinder was diminished. Consequently, for the prediction of scour around the rectangular / square GBS, where the caisson is high and exposed, the diameter of the caisson was used for predicting the scour depth.

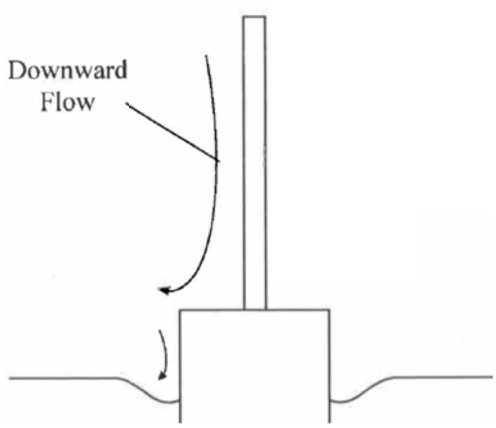


Figure 4 – Sketched visualisation reported by Yeow and Cheng (2003), of flow down the up-wave face of a cylinder situated on top of a caisson

Figure 4, reproduced from Yeow and Cheng (2003), also suggests the concept of a null zone somewhere in front of the structure. The wave is advancing from the left and moving towards the right and in so doing, it will generate a forward mass flux in that direction, which will manifest as a current. On the other hand, the downward-descending jet transmits a current moving against the wave, thus suggesting that a null zone will exist at some location in front of the structure. The existence of a null zone, or stagnation line, was also reported by the DHI Group (2012), in the context of current-induced flow around a monopile. However, the principles are the same as those applying to the tests conducted by Yeow and Cheng (2003); it is only the application that is different. In the case of current-induced motion around a monopile, there is still a downward – descending jet, which generates a local current that opposes the incoming stream, at the base of the pile – hence the existence of the stagnation line.

For predicting the scour around the rectangular / square GBS, the methods proposed by Bos et al (2002[a]) may be applied without modification. This is an empirical approach developed for the prediction of scour around large submerged caissons and appears to provide predictions that are in good agreement with prototype observations.

3.4 Prediction of scour around the Jacket design

For predicting the scour due to currents and a combination of waves and currents, the methods presented by Harris et al (2010) were adopted. Valedictory tests were made by comparing predictions obtained by this method against experimental observations of scour around a 4-legged $^{1}/_{36}$ scale model jacket, reported by Yang et al (2010). The seabed material in their test was made of coal granules.

It was found that the reduction in equilibrium scour depth promoted by the action of the wave, as recommended by Sumer and Fredsϕe (2002) and adopted by Harris et al (2010), tended to produce a lower value than that observed in the tests conducted by Yang et al (2010). It was found that an improvement in the prediction could be effected by taking into account the wave-induced flux, in the assessment of the total applied current.

Figure 5 shows the experimental configuration of the physical model tests by Yang et al (2010); it is noted that there is very little interaction between the four scour holes around the corner piles of the jacket, despite the fact that the model sediment was lighter than sand. It is noted that the ripples on the seabed are smaller in height than would be expected, had the model been undertaken using a sand bed. For this reason at least, it is believed that the tests by Yang et al (2010) probably represent a realistic description of scour around a jacket structure in storm conditions. Tests conducted on a sand bed tend to produce larger ripples, which are often not in keeping with prototype observations and which also radically change the behavioural properties of the seabed boundary layer, compared to the prototype scenario.

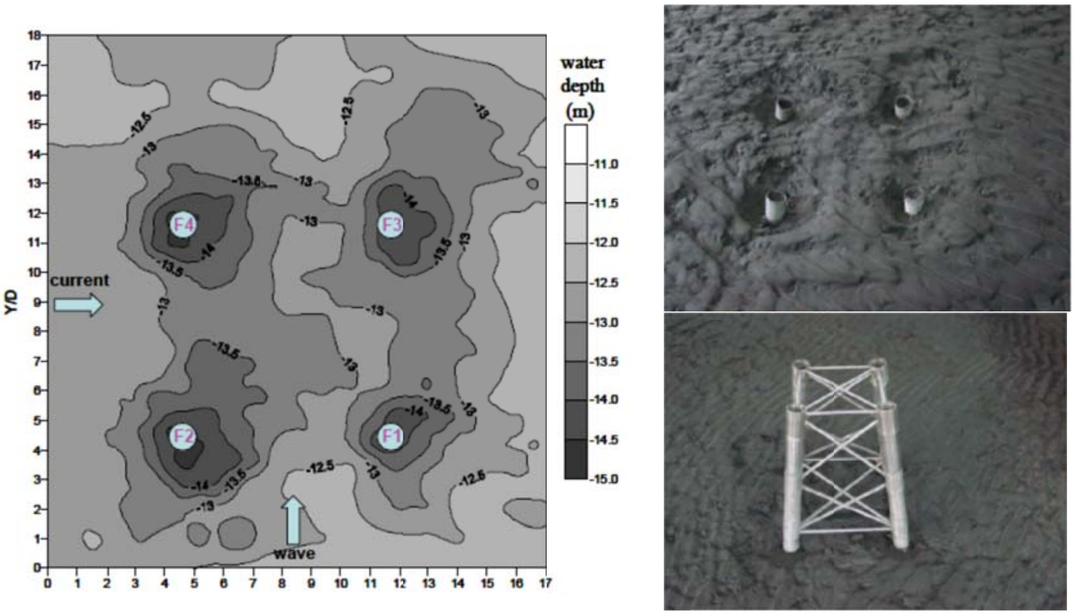


Figure 5– extent of scour observed around the model experiments reported by Yang et al (2010) using coal grains as the seabed material

3.5 Prediction of scour around the Tripod design

For predicting scour around the tripod structure due to waves, a comparison was made between the results of physical model tests on the Alpha Ventus tripods reported by Stahlmann and Schlurmann (2010), and predictions made by applying the methods developed by Khalfin et al (2007). Based on this comparison, the following conclusions were drawn:

1. For predicting scour around the tripod in waves only, use the method proposed by Khalfin (2007)
2. For predicting scour around the tripod in currents only, use the standard method for slender piles, as described by Harris et al (2010)
3. For predicting scour around the tripod in waves and currents combined, use the method by Khalfin (2007), but modify the applied seabed shear velocity by using the method derived by Soulsby and Clarke (2002).
4. For predicting scour around the tripod in waves and currents combined, also check the results by using the method for a slender pile, as described by Harris et al (2010).
5. Take the worst case of (4) and (3) for the final prediction of scour around the tripod in waves and currents combined

4 SCOUR ASSESSMENT RESULTS SUMMARY

Based on the scour assessment methods described above, detailed assessments of scour depths, footprint areas and scour volumes under currents alone, waves alone and combined waves and currents have been performed for 1 in 1 year and 1 in 50 year return period events. Results from these detailed assessments are presented in full in Appendix A.

From these results the worst case scour volume has been taken and a release rate has been calculated from the assessment of timescales for scour development. These results are summarised in Table 3. For all foundation / substructure types both the 1 in 1 year and 1 in 50 year values are presented.

Table 3 – Scour Footprint Areas and Scour Volumes and Material Release Rates used in Worst Case Assessments

Foundation / Substructure Type	Return Period	Scour Footprint Area (m^2)	Scour Volume (m^3)	Comments
Jacket	1 in 1 yr	842	838	For purposes of comparison with GBS values
	1 in 50 yr	842	838	
Tripod	1 in 1 yr	956	1,152	
	1 in 50 yr	956	1,152	
Rectangular GBS (100m x 75m rectangular baseplate)	1 in 1 yr	1,174	2,038	Rochdale Envelope Worst Case for OSP at up to 1 location in Project Alpha
	1 in 50 yr	1,850	4,032	
Square GBS (40m x 40m square baseplate)	1 in 1 yr	137	81	Rochdale Envelope Worst Case for OSP at up to 2 locations in each of Project Alpha and Project Bravo
	1 in 50 yr	518	597	
Conical GBS (52m baseplate diameter)	1 in 1 yr	3,137	1,067	Rochdale Envelope Worst Case for WTG and met masts (50m water / average soils)
	1 in 50 yr	4,283	4,304	
Conical GBS (72m baseplate diameter)	1 in 1 yr	5,150	924	Rochdale Envelope Worst Case for WTG and met masts (60m water / weak soils)
	1 in 50 yr	6,671	4,877	

5 REFERENCES

Bolle, A., Haerens, P., Trouw, K., Smits, J. and Dewaele, G. (2009): *Scour around gravity-based wind turbine foundations – prototype measurements*. In: Coasts, Marine Structures and Breakwaters. Adapting to Change. Ed Allsop, W. 103-118. Thomas Telford Books, London.

Bolle A., Mercelis P., Goossens W. and Haerens P. (2010).*Scour monitoring and scour protection solution for offshore gravity based foundations*. Proceedings of the Fifth International Conference on Scour and Erosion, San Francisco, 7-10 November 2010. Geotechnical Special Publication No. 120, ASCE (ISBN 978-0-7844-1147-6)

Bos, K.J., Veheij, H.J., Kant, G. and Kruisbrink, A.C.H. (Bos et al 2002[a]): *Scour Protection around Gravity Based Structures Using Small Size Rock*. Proceedings . ICSF-1: First International Conference on Scour of Foundations, Texas, Nov 17-20, 2002

Bos, K.J., Chen, A., Verheij, H.J., Onderwater, M. and Visser, M. (Bos et al 2002[b]): *Local Scour and Protection of F3 Offshore GBS Platform*. Proc 21[st]IntnlConf of Offshore Mechanics and Arctic Engrg, June 23-28, Oslo. Paper No OMAE2002-28127

Department of Energy and Climate Change - DECC (2008): *Dynamics of scour pits and scour protection – synthesis report and recommendations* (Milestones 2 and 3) – Final Report

DHI Group (2012)
http://www.dhigroup.com/News/2012/03/02/LongLastingScourProtectionForOffshoreWindFarms.aspx

Eadie, R.W. and Herbich, J.B. (1986): Scour about a single, cylindrical pile due to combined random waves and a current. *ProcConf Coastal Engrg,* pp 1858 – 1870

Haake, G., Rolfes, R., Schaumann, H. Huhn, Schlurmann, L.Lohaus and Achmus, M. (date not known): Research on support structures in the German offshore wind farm Alpha Ventus (provenance unknown)

Harris, J.M., Whitehouse, R.J.S. and Benson, T. (2010): *The time evolution of scour around offshore structures.*ProcInstCivEngrs, Maritime Engineering, Vol 163, pp 3-17

Hoffmans, G.J.C.M. and Verheij, H.J. (1997): *Scour Manual*. Published by A.A. Balkema, Rotterdam. ISBN 90 5410 673 5

Kaiser, M.J. and Snyder, B.F. (2012): *Offshore Wind Energy Cost Modelling, Green Energy and Technology*. DOI:10.1007/978-1-4471-2488-7_2. Springer Verlag

Khalfin, I.S.H. (1983): *Local scour around ice-resistant structures caused by waves and current effect*.P.O.A.C. Symposium 28, Helsinki, Vol. 2, pp. 992-1002.

Khalfin, I.Sh.,Avdeewa, V.I. and Longiniv, V.V. (1988): *Local scour of riprap around ice-resistant cylindrical and conical gravity structures caused by waves.* ProcIntConf on Technology for Polar Areas (POLAR-TECH-88), Trondheim: Tapir Publishers, 1988, Vol I, pp 275-285

Khalfin, I. Sh. (2007): *Modelling and calculation of bed score around large-diameter vertical cylinder under wave action*. Water Resources, Vol 34, No 1, 49-59 (Note – the word ‘score’ is not a typographical error, but is the word used by the author)

Soulsby, R.L. and Clarke, S. (2002): *Bed shear stresses under combined waves and currents on smooth and rough beds*. HR Wallingford Report TR137, Rev 1.0. Produced within Defra project FD1905 (EstProc)

Stahlmann, A. and Schlurmann, T. (2010): *Physical Modelling of Scour Around Tripod Foundation Structures for Offshore Wind Energy Converters*. ProcConf Coastal Engrg

Sumer, M. B. and Fredsϕe, J. (2002): *The Mechanics of Scour in the Marine Environment.*Published by World Scientific. ISBN 981 02 4930 6

Whitehouse, R.J.S. , Harris, J.M., Sutherland, J., and Rees, J. (2011): *The nature of scour development and scour protection at offshore windfarm foundations*. Marine Pollution Bulletin 62, (2011), pp 73-88

Yang, R-Y., Chen, H-H., Hwung, H-H., Jiang, W-P., Wu, N-T. (2010) :*Experimental study on the loading and scour of the jacket-type offshore wind turbine foundation*. ProcIntnlConf Coastal Engrg. 32

Yeow, K. and Cheng, L. (2003): *Local scour around a vertical pile with a caisson foundation*. Proc Cong Asian and Pacific Coasts 2003.

APPENDIX A - SCOUR ASSESSMENT RESULTS

Conical Gravity Base Structure

Predictions of scour around the Conical Gravity Base Structure – 50m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	50	28.4	1.65	601	2872	62x60	Khalfin (1983)/ Bos (2002[b])
1	6.7	11.0	-	50	28.4	1.56	536	2828	61x59	Khalfin (2007)
1	6.7	11.0	1.21	50	28.4	2.18	1067	3137	65x62	Khalfin (2007)/Soulsby (2002)
1	6.7	11.0	1.21	50	*52.0*	0.21	9	2212	53x53	Bos et al (2002[a]): base plate
50	-	-	1.38	50	28.4	2.15	1037	3122	65x62	Khalfin (1983)/ Bos (2002[b])
50	8.7	13.0	-	50	28.4	3.33	2582	3754	72x67	Khalfin (2007)
50	8.7	13.0	1.38	50	28.4	4.24	4304	4283	77x72	Khalfin (2007)/Soulsby (2002)
50	8.7	13.0	1.38	50	*52.0*	0.53	60	2351	55x54	Bos et al (2002[a]): base plate

For scour due to waves plus currents around the base plate (using the method due to Bos et al 2002[a]), the height of the structure has been taken to be the height of the base plate at its intersection with the bottom of the cone. This height is 3.0m.

Predictions of scour around the Conical Gravity Base Structure – 60m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	60	35.4	1.75	924	5150	82 x 80	Khalfin (1983)/ Bos (2002[b])
1	6.7	11.0	-	60	35.4	1.11	366	4740	79 x 77	Khalfin (2007)
1	6.7	11.0	1.21	60	35.4	1.75	924	5150	82 x 80	Khalfin (2007)/Soulsby (2002)
1	6.7	11.0	1.21	60	*72.0*	0.16	7	4165	73 x 73	Bos et al (2002[a]): base plate
50	-	-	1.38	60	35.4	2.27	1574	5496	85 x 83	Khalfin (1983)/ Bos (2002[b])
50	8.7	13.0	-	60	35.4	2.70	2250	5791	88 x 85	Khalfin (2007)
50	8.7	13.0	1.38	60	35.4	3.92	4877	6671	95 x 90	Khalfin (2007)/Soulsby (2002)
50	8.7	13.0	1.38	60	*72.0*	0.50	73	4366	75 x 74	Bos et al (2002[a]): base plate

For scour due to waves plus currents around the base plate (using the method due to Bos et al 2002[a]), the height of the structure has been taken to be the height of the base plate at its intersection with the bottom of the cone. This height is 4.2m.

Nomenclature:

- A_e predicted equilibrium scour footprint area (*including* footprint area of the base plate)
- d water depth (m)
- D cylinder diameter (m)
- Dims gross dimensions of the scour footprint (*including* the base plate diameter)
- H_s significant wave height (m)
- S_e predicted equilibrium scour depth (m)
- T_p peak wave period (s)
- U_c current speed (m/s)
- V_e predicted equilibrium scour volume (m^3)

Notes:

1. Volumes, areas and dimensions of scour are presented rounded up to the nearest integer value. This also applies to the entire set of scour predictions for the other structures.
2. In this table, the cylinder diameter D is the structural size driving the prediction of the equilibrium scour depth. It was assumed that the predicted equilibrium scour depth is achieved around the periphery of the base plate, when calculating the scour

volume. This assumption is probably realistic for scour by currents alone, but may be conservative for scouring by waves.

3. The models predict that scour around the Conical GBS is greater under the action of currents and waves together, than it is when either currents or waves are acting alone. It is believed that the physical explanation for this might be that when the cylinder is of large diameter and surface-piercing, then wave reflection will cause energy-trapping on the up-wave face of the cylinder at the seabed level, in much the same way as occurs in front of a sea wall. However, due to the curvature of the cylinder, the wave-induced scour is likely to diminish with angular position around the structure, for a given individual wave direction. On the other hand, over a long period of time, with waves approaching from many directions, it can be assumed that the wave-induced scour could become relatively uniformly distributed around the structure.
4. As explained earlier, the equilibrium scour depth for the Conical Gravity Base Structure has been predicted using the diameter of the base of the cone as the representative diameter. It was then assumed that this scour depth occurs around the periphery of the base plate, when calculating the corresponding scour volume. The background to this approach is derived in relation to experimental results reported by Yeow and Cheng (2003).

Discussion of results

1 Scour in currents only

The Khalfin (1983) equation, modified by Bos et al (2002[b]), does not provide the maximum predicted depth of scour hole when the caisson is surface-piercing, but when the structure is at an intermediate height, compared to the water depth. The table below summarises the predicted equilibrium scour depth using the Khalfin (1983) solution for the cone diameter of 35.4m, but for a range of structural heights, in a water depth of 60m. The predicted maximum equilibrium scour depth occurs when the structure has a height of around 20m.

Predicted equilibrium scour depths for a caisson of 35.4m diameter in 60m water depth, with a current speed of 1.21m/s, for various heights of the structure.

Height of structure h_s (m)	(h_s/D)^1.43	N	[(0.5αc·U)^2/(gh)]^N	Scour depth S_e (m)
5	0.061	0.427	0.077	1.50
10	0.164	0.540	0.039	2.04
15	0.293	0.620	0.024	2.26
20	0.442	0.684	0.017	2.33
25	0.608	0.737	0.012	2.32
30	0.789	0.785	0.009	2.27
35	0.984	0.827	0.007	2.19
40	1.191	0.865	0.006	2.11
45	1.409	0.901	0.005	2.02
50	1.639	0.933	0.004	1.93
55	1.878	0.964	0.003	1.84
60	2.127	0.993	0.003	1.75

Method: Khalfin (1983) and modified by Bos et al (2002[b]).

2 Scour in waves only and in waves plus currents

The Khalfin (2007) solution for scour around a surface-piercing caisson in waves is a function of two dimensionless terms:

- The Keulegan-Carpenter number *KC*
- The relative mobility of the seabed particles expressed as the ratio of the applied seabed shear velocity to the critical value for the seabed sediment: $U_{fw}/u_{*,crit}$

Khalfin recommended the following empirical fit to his experimental data:
$S_{max}/D = 0.0753(U_{fw}/u_{*cr} - 0.5)^{0.69}KC^{0.68}$ with $S_{max}/D = 0$ at $U_{fw}/u_{*cr} \leq 0.5$

The experimental validity ranges of the solution were as follows:

$0.09 \leq d/L \leq 0.30$; $0.06 \leq D/L \leq 0.80$; $0.50 \leq U_{fw}/u_{*cr} \leq 2.40$; $0.10 \leq KC \leq 3.50$

In the above set of criteria, *L* is the wavelength and *d* the water depth.

In the present study, two water depths have been considered, namely 50m and 60m; the return period wave input parameters of height and period have been retained as the same for both depths. Under those circumstances, the value of U_{fw} – the peak wave-induced shear velocity – will be smaller in the 60m water depth than in the 50m situation. The value of $u_{*,crit}$ on the other hand, is a constant property of the seabed material. Consequently, in the present case, increasing the water depth leads to a reduction in the ratio U_{fw}/u_{*cr}. At the same time, the Keulegan-Carpenter number will also become smaller in 60m of water on two counts: (i) the cylinder diameter is larger in 60m of water than in 50m and (ii) the peak water-particle velocity under the wave will be smaller in 60m water depth than in 50m. Consequently, the right hand side of the Khalfin equation, (reproduced above for convenience, from Annex A), will be significantly smaller in 60m of water than in 50m depth. Balanced against that, the structural diameter has been increased.

Overall, one would expect a decrease in the predicted equilibrium wave-induced scour depth to occur in 60m of water, compared to the 50m depth, since the increase in the cone structural diameter with water depth is not great but there will be a reduction in wave-induced water particle velocities at the seabed and also the *KC* number is reduced by about 40-45%. A comparison between the predicted wave-induced scour depths in 50m and 60m water depth indicates that this expectation has been fulfilled.

On the other hand, the predicted scour due to currents is more strongly driven by the cylinder diameter and therefore since the structure is larger in the 60m water depth design than in the 50m solution, then it can be expected that the scour depth in currents alone would increase. Again a comparison between the two sets of prediction confirms this expectation.

Finally, it follows that because the trends for scour in waves and currents as a function of water depth are, in the present case at least, working in opposite directions, then the scour depths in waves plus currents combined are likely to show only a modest difference, and that is observed to be the case. The scour volumes will be larger in a 60m water depth, however, because they are calculated using the diameter of the base plate as the principal periphery.

It is noted that the 50-year values of U_{fw}/u_{*cr} are beyond the range of the experimental values reported by Khalfin (2007), at 3.42 and 4.03 in water depths of 60m and 50m respectively. However, the Khalfin data for coarse sand suggest that the solution is probably capable of reasonable extrapolation beyond the experimental range of 0.5 to 2.4.

It is noted that in the case of waves-only scour, the increase in base plate diameter in the 60m water depth is insufficient to offset the effects of the decrease in scour depth, with the result that scour volumes are smaller in 60m than in 50m water depth, for that scenario. This argument applies not only to the elliptic scour hole, but also to the symmetrical cone, whose scour volume can be calculated using a closed form solution, whereas that of the elliptic cone is obtained by Gaussian quadrature numerical integration.

Rectangular / Square Gravity Base Structure

Predictions of scour around Rectangular (100m x 75m) Gravity Base Structure – 50m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	50	45	5.21	2038	1174	*2x(31x24)	Khalfin (1983)/ Bos (2002)
1	6.7	11.0	-	50	45	1.81	86	142	*2x(11x8)	Khalfin (2007)
1	6.7	11.0	1.21	50	45	2.53	234	272	*2x(15x12)	Khalfin (2007)/Soulsby (2002)
50	-	-	1.38	50	45	6.54	4032	1850	*2x(39x30)	Khalfin (1983)/ Bos (2002[b])
50	8.7	13.0	-	50	45	3.86	829	644	*2x(23x18)	Khalfin (2007)
50	8.7	13.0	1.38	50	45	4.92	1716	1047	*2x(29x23)	Khalfin (2007)/Soulsby (2002)

Predictions of scour around Square (40m x 40m) Gravity Base Structure – 50m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	50	15	0.97	13	41	*2x(6x4)	Khalfin (1983)/ Bos (2002)
1	6.7	11.0	-	50	15	1.27	30	70	*2x(7x6)	Khalfin (2007)
1	6.7	11.0	1.21	50	15	1.78	81	137	*2x(10x8)	Khalfin (2007)/Soulsby (2002)
50	-	-	1.38	50	15	1.34	35	78	*2x(8x6)	Khalfin (1983)/ Bos (2002[b])
50	8.7	13.0	-	50	15	2.71	287	318	*2x(16x13)	Khalfin (2007)
50	8.7	13.0	1.38	50	15	3.46	597	518	*2x(20x16)	Khalfin (2007)/Soulsby (2002)

Nomenclature:

A_e	predicted equilibrium scour footprint area (*including* footprint area of the base plate)
d	water depth (m)
D	cylinder diameter (m)
Dims	gross dimensions of the scour footprint (*including* the base plate diameter)
H_s	significant wave height (m)
S_e	predicted equilibrium scour depth (m)
T_p	peak wave period (s)
U_c	current speed (m/s)
V_e	predicted equilibrium scour volume (m^3)

Notes:

1. The baseplate is 100m x 75m rectangular
2. The height of the baseplate is 7.5m in all cases
3. It is assumed that the columns act as a single combined surface piercing unit (as a worst case)

Four Legged Jacket Structure

Predictions of scour around Four Legged Jacket Structure – 50m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	50	2.1	2.73	838	842	18 x 15	Harris et al (2010)
1	6.7	11.0	1.21	50	2.1	1.61	216	347	12 x 10	Harris et al (2010)
50	-	-	1.38	50	2.1	2.73	838	842	18 x 15	Harris et al (2010)
50	8.7	13.0	1.38	50	2.1	1.92	346	462	13 x 11	Harris et al (2010)

Nomenclature:

A_e predicted equilibrium scour footprint area (*including* footprint area of the base plate)
d water depth (m)
D cylinder diameter (m)
Dims gross dimensions of the scour footprint (*including* the base plate diameter)
H_s significant wave height (m)
S_e predicted equilibrium scour depth (m)
T_p peak wave period (s)
U_c current speed (m/s)
V_e predicted equilibrium scour volume (m^3)

Notes:

1. V_e denotes the total scour volume arising from all four of the corner piles together plus the contribution from the bracing elements
2. A_e denotes the total scour footprint arising from all four of the corner piles together
3. The dimensions are the gross dimensions of the scour hole around one pile and include the pile itself
4. There was a large standard deviation attached to the principle coefficient of the equilibrium scour depth for currents alone, derived by Sumer and Fredsøe (2002), upon which the method used by Harris et al (2010) is based. The values given in the table are based upon the average, or expected value, of the coefficient, which has a value of 1.3, whereas it possesses a standard deviation of 0.7. As a result, the most probable upper-bound value of equilibrium scour depth is around twice the expected value given in the table.
5. Once the current speed is greater than the critical threshold value for the seabed material, then no further increase in equilibrium scour depth around a single slender pile can occur, according to the method being used here. For that reason, the predicted 1-yr and the 50-yr equilibrium scour depths in currents alone are the same and are controlled entirely by pile diameter. For justification, reference can be made to equation 6 and the supporting definitions, reported by Harris et al (2010), in respect of the bed condition coefficient.
6. The maximum downstream extent of the scour hole, under the expected conditions, is 11m and the upstream extent is 5m. Therefore the gap between two scour holes will be around 15m, with the piles being 2.1m diameter and situated 32m apart. If the most probable upper-bound scour hole were to develop, even then the scour holes would only just be on the point of touching one another. Therefore, it is unlikely that the scour holes will interact in the jacket structure.

Tripod Structure

Predictions of scour around the Tripod Structure – 50m water depth

Return (yrs)	H_s (m)	T_p (s)	U_c (m/s)	d (m)	D (m)	S_e (m)	V_e (m^3)	A_e (m^2)	Dims (m x m)	Methods of predicting equilibrium scour depth
1	-	-	1.21	50	3.0	3.28	1152	956	22 x 18	Harris et al (2010) and S&F (2002)
1	6.7	11.0	-	50	3.0	0.76	79	114	7 x 7	Khalfin (2007)
1	6.7	11.0	1.21	50	3.0	1.06	115	171	9 x 8	Khalfin (2007)/Soulsby& Clarke
1	6.7	11.0	1.21	50	3.0	1.55	216	289	12 x 10	Harris et al (2010) and S&F (2002)
50	-	-	1.38	50	3.0	3.28	1152	956	22 x 18	Harris et al (2010) and S&F (2002)
50	8.7	13.0	-	50	3.0	1.62	275	309	13 x 11	Khalfin (2007)
50	8.7	13.0	1.38	50	3.0	2.07	435	449	15 x 13	Khalfin (2007)/Soulsby& Clarke
50	8.7	13.0	1.38	50	3.0	1.93	378	402	14 x 12	Harris et al (2010) and S&F (2002)

Nomenclature:

A_e predicted equilibrium scour footprint area (*including* footprint area of the base plate)
d water depth (m)
D cylinder diameter (m)
Dims gross dimensions of the scour footprint (*including* the base plate diameter)
H_s significant wave height (m)
S_e predicted equilibrium scour depth (m)
T_p peak wave period (s)
U_c current speed (m/s)
V_e predicted equilibrium scour volume (m^3)

Notes:

1. V_e denotes the total scour volume arising from all three of the corner piles together plus the contribution from the bracing elements
2. A_e denotes the total scour footprint arising from all three of the corner piles together
3. The dimensions are the gross dimensions of the scour hole around one pile and include the pile itself
4. S&F (2002) in the Methods column denotes that a correction has been applied to the predicted equilibrium scour depth, to account for the relative length of the pile, using the method proposed by Sumer and Fredsϕe (2002). This value of this multiplication factor is 0.84, on the assumption that the height of the top of the pile above the seabed is 10 metres.

The distance between the centre lines of two collinear piles in the Alpha Ventus tripods is around 25m and this applies to a water depth of 30m. If the distance between the piles can be scaled by water depth, then in 50m of water, it is likely that the corresponding distance will be around 40m. Likewise, the horizontal distance of each of the three corner piles from the vertical line through the central column is of the order of 15m in the Alpha Ventus structure and again adopting a pro rata distance on relative water depth, this distance would be around 25m if the water depth were 50m.

On the basis of relative proportions, it is assumed that the base of the central column is at a height of 3m above the seabed and that the mean height of the three lower raking brace cylinders is 3m above the seabed. It is predicted that this configuration will, in the 50-year return period condition, generate an additional 88m^3 of scour, to a depth of around 0.5m. Under 1-year conditions, the corresponding volume will be 49 m^3 of additional scour.

For the worst case scour scenario, which is that due to currents alone, the predicted downstream extent of the scour hole footprint is 13m and the upstream edge is located at 6m from the pile edge. The scour hole of an individual pile will therefore possibly interact with the erosion taking place beneath the central column. Interaction between the scour holes formed by any two of the three corner piles is fairly unlikely, although over time, collapses of the seabed material between two adjacent holes could occur. Inspection of Figure 6, which shows the condition of the model seabed at the end of a scour test on the 1:12 model tripod structure for Alpha Ventus, probably confirms this viewpoint.

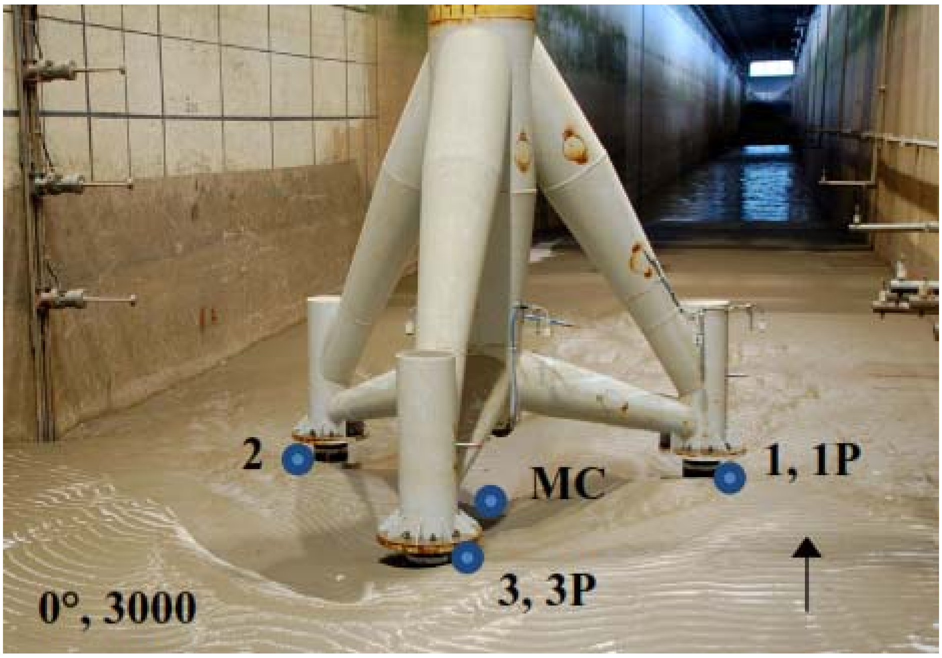


Figure 6 - Condition of the model seabed at the end of a scour test on the 1:12 scale model of the Alpha Ventus tripod structure reported by Stahlmann and Schlurmann (2010)

Scour timescales

The information available regarding the timescale of scour around GBS is rather scanty. Hoffmans and Verheij (1997) provide some guidance, based upon a solution offered by Teramoto (also referenced by Bos, 2002[a]), but it is only suitable for clear-water scour. Beyond that, the available methods are those proposed by Sumer and Fredsϕe (2002) for predicting the timescale of scour around mono-piled structures and which have been applied here throughout. A general consensus is that wave-induced scour reaches equilibrium fairly quickly, within the timescale of a typical storm event, whereas scour due to currents takes a considerably longer period to achieve the final result.

DECC (2008) and Whitehouse et al (2011) provide some field observations on time scales of scouring around single piles and note that in some cases, the timescale seems to be very long and that at one pile in the North Sea (N7), the scour process might still be advancing after a period of 5 years of scour and partial recovery. Bos et al (2002[b]) made similar comments regarding the development of scour around the gravity base structure F3, also located in the North Sea. Their paper shows a number of scour configurations developing at a given cross section over a period of three years, and they quote a time-variation in scour depths ranging from 2.5m to 3.5m.

The table below shows predictions of the time scale of current-induced scour predicted for the four types of structure considered in this study, made by applying the methods recommended by Sumer and Fredsϕe (2002) for piles. For wave-induced scour, the Sumer and Fredsϕe method in all cases predicts a time to equilibrium depth of less than three hours.

Predictions of the time scale of current-induced scour

Structure Type	Diameter (m)	Return Period (years)	Predicted time to equilibrium (days)
GBS	28.4 (base of cone)	1	17
		50	10
Jacket	2.1	1	1.4
		50	0.8
Tripod	3.0	1	1.5
		50	0.8

Field experience of scour around piled structures

The scour depths predicted in this study in respect of piled foundations are of the order of one pile diameter, or in some cases, a little more than that. It is valuable at this stage to compare the overall magnitude of these predictions against the depth of scour holes observed in the field; the table below provides a summary of data on scour holes depths observed around monopiles and it is noted that those founded on a sufficient depth of sand generated scour holes of around 1.0 to 1.2 times the pile diameter. It is therefore believed that the values of scour hole depths predicted in the present study are realistic.

Summary of scour experiences in the field

Location	Sediment type	D_p (m)	max S_e (m, or as a ratio of D_p)	h (m)	U_c (m/s)	Annual H_s (m)
Scroby	Medium sand, some gravel/shell, clay at Depth. D_{50} is 0.2 to 0.4mm.	4.2	0.95 to 1.38 D_p	3-12	1.65	1.0 – 3.5 (depth-limited)
Arklow Bank	Loose to medium dense sand and sandy gravel	5.0	0.8D_p prior to placement of scour protection	2-6	>2.0	5.6 (depth-limited)
N7	Fine medium dense sand	6.0	1.05D_p	5.2	0.75	1.1
Scarweather (met mast)	Medium to fine shelly sand	2.2	0.3 to 0.6D_p	6.0	1.1	2.8
OtzumerBalje inlet	Medium sand	1.5	1.47D_p	11.7	1.4	sheltered
Egmondaan Zee	Medium grade sand	2.9	0.79D_p	20		Not quoted
Barrow	fine sand to muddy sand, some gravels overlying clay; exposed clay. D_{50} is ~0.09mm and d_{84} is ~0.15mm.	4.75	1.21D_p on sandy areas but restricted by resistant sub-grade elsewhere	12- 18	0.8	4.9 (T_p 9.8s)
Kentish Flats	fine sand; infilledpaleochannel with clays and sands; clay near surface or exposed	5.0	0.46D_p restricted by resistant sub-grade and also surficial clay	3-5	0.9	3.3 (depth-limited)
North Hoyle	gravelly medium sand or sandy gravel overlying clay	4.0	0.13D_p restricted by resistant sub-grade	6–12	1.2	4.9
Constable Bank	Not specified	1.89	0.79D_p			
Gravity Base Structure F3	Fine sand, 150μm D_{50}	70x80x16 caisson dims	3.5m over the period 1994 to 1998	42.3	0.72m/s (storm)	4.9 (one-yr return)

(results as reported by Whitehouse et al 2011, and with some support from DECC, 2008; also from Bos et al 2002[b]. With the exception of Gravity Base Structure F3 (reported by Bos et al, 2002[b]), all of the structures are piles)

Nomenclature:

D_p pile diameter
S_e observed equilibrium scour depth
h water depth at LAT
U_c current speed
H_s significant wave height

Part II - Installation Assessment

1 BACKGROUND

Impacts to the marine environment may arise during the construction process, caused by the installation of foundations / substructures.

Prior to the installation of any type of foundation or substructure, a pre-installation survey will be undertaken to confirm that no obstructions are present.

2 PILED TRIPOD OR PILED JACKETS

Piled foundation solutions do not normally require any sea bed preparation and will usually be driven using a hydraulic hammer operating from an installation vessel. It is unlikely that a jack-up barge will be used since there are few capable of working in the water depths found at the Phase 1 site. Depending on the site ground conditions, drilling may be required to supplement driving operations to achieve the required penetration. This would require careful control of drilling fluids, cement grout and disposal of the drilling arisings. Assuming that the operations are undertaken in accordance with industry-standard procedures, there are considered to be insignificant impacts to the physical environment arising from the installation of piles.

3 GRAVITY BASE STRUCTURES (GBS)

GBS usually require sea bed preparation prior to installation. Typically this will be undertaken by a cutter suction dredger to remove superficial sediments, followed by rock and/or gravel dumping to form a level footing.

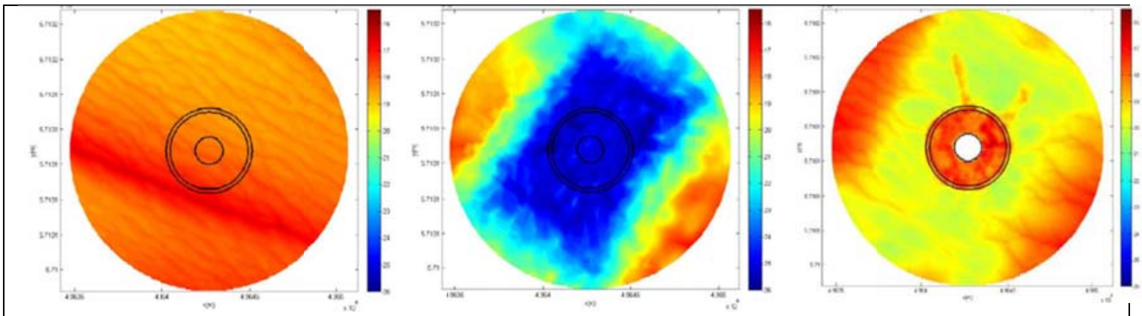
The depth of dredging required for conical GBS in the worst case (areas of weak soils) is likely to be up to a maximum of 5m below the original sea bed over the footprint area, tapering to zero at the boundary of the 'zone of influence'. This is likely to be at a maximum of 8 locations in each of Project Alpha and Project Bravo. At other locations where conical GBS may be used, seabed preparation depths of up to 3m are anticipated as the maximum requirement. For the OSP, sea bed preparation depths of up to 5m are assumed for the rectangular / square GBS.

The arisings are likely to be disposed of *in-situ* through side-casting methods, although it is possible that some could be used as ballast. It is also possible that if cutter suction dredging is used that material may be taken into the hopper of the dredging vessel and release in bulk into the water column, from where the majority will rapidly be transported vertically downwards through the water column to settle on the seabed.

3.1 Field Evidence from Thornton Bank

The most accessible example of the use of gravity base structures (GBS) to date is that of Thornton Bank off the coast of Belgium. The first six WTGs of the C-Power farm were installed in 2008 on Thornton Bank using conical GBS. Considerable seabed preparation (excavation, filter layer, gravel layer and GBS placement and backfilling) was undertaken prior to provision of both a filter layer and armour layer of scour protection materials. Bathymetric measurements were performed by Dredging International using multibeam echosounder for monitoring of erosion pits. For each of the six GBSs, five surveys were executed: (i) prior to works; (ii) after dredging of pits; (iii) after installation of gravel bed; (iv) prior to installation of filer layer; and (v) after completion of the works.

In the survey data shown in Figure 1, the dredged pit was clearly visible during construction, however after the installation of the GBS and the scour protection materials no secondary scour was observed.



Baseline Conditions **Dredged Pit** **Post Completion (no secondary scour)**

Figure 1 – Sea bed conditions at Thornton Bank at different stages of conical GBS installation (source: Van den Eynde *et al*, 2010)

3.2 Firth of Forth – Fate of Seabed Preparation Material

At the Phase 1 site of the Firth of Forth zone there are four types of GBS under consideration;

- Conical GBS suitable for water depths of 50m;
- Conical GBS suitable for water depths of 60m;
- Rectangular GBS (100m x 75m baseplate) suitable for OSP at up to 1 location in Project Alpha; and
- Square GBS (40m x 40m baseplate) suitable for OSP at up to 2 locations in each of Project Alpha and Project Bravo.
-

It has been estimated that seabed preparation to a level of 5m below the seabed may be required at up to 8 locations within each of Project Alpha and Project Bravo for the conical GBS. Each of the GBS for OSP will also require up to 5m sea bed preparation. At all other foundation / substructure locations, seabed preparation will be up to 3m.

The volumes of seabed material yielded from seabed preparation activities is presented in Table 1.

Table 1 – Seabed preparation volumes

Foundation / Substructure	Baseplate	Footprint	Seabed preparation depth	Volume of seabed preparation material
Conical GBS (60m water depth / weak soils)	72m octagonal diameter	4,295m^2	5m	21,475m^3
Conical GBS (50m water depth / average soils)	52m octagonal diameter	2,240m^2	3m	6,720m^3
Rectangular GBS (for OSP at up to 1 location)	100m x 75m rectangular	7,500m^2	5m	37,500m^3
Square GBS (for OSP at up to 4 locations)	40m x 40m square	1,600m^2	5m	8,000m^3

This volume of material will be released into the marine environment in a phased manner as construction progresses across the site. To determine its fate assessments have been made of the sediment grading curves derived from the benthic survey (locations shown in Figure 2) and of the metocean data covering the governing physical process that may initiate mobilisation of these sediments.

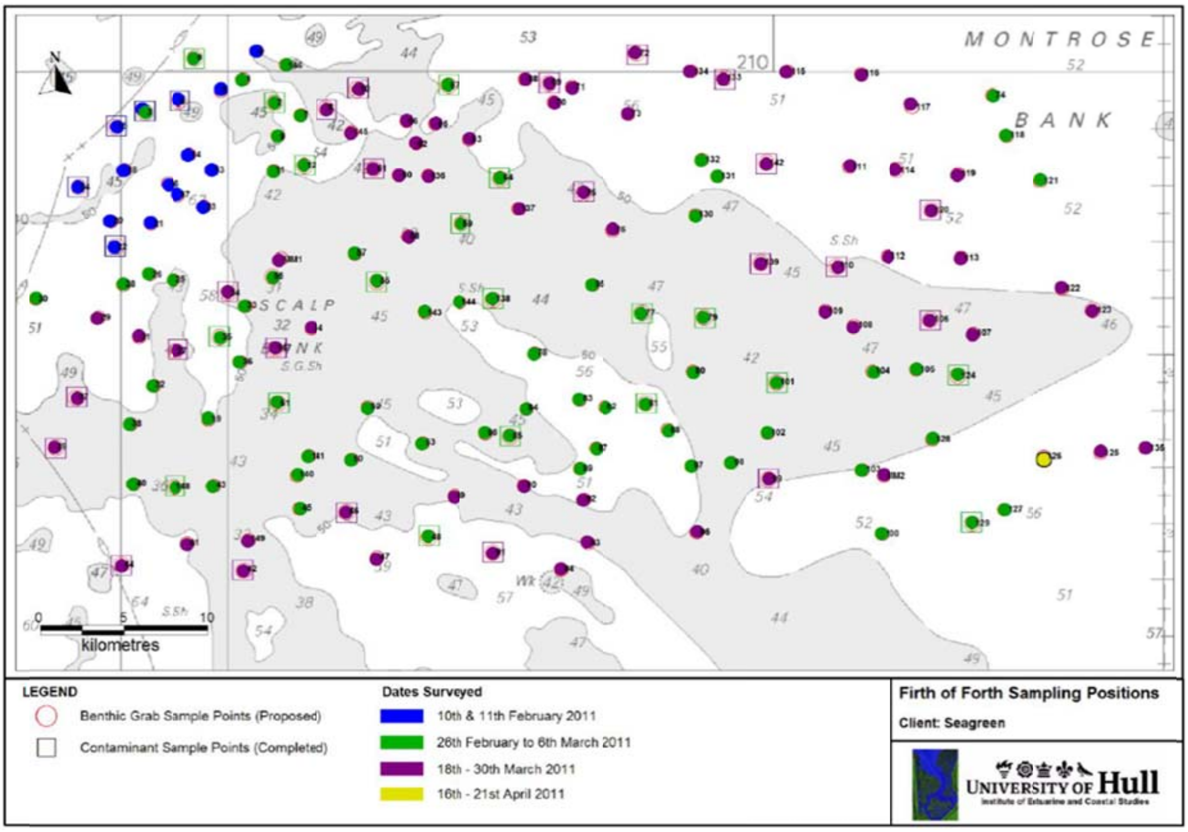


Figure 2– Sea bed sediment grab sampling locations (source: University of Hull, 2011)

Table 2 provides a summary of the tidal current statistics recorded at the various deployment sites and reported in the Fugro metocean data report. The two most northerly sites A and B are located in Projects Alpha and Bravo and are the most relevant ones for the present study.

The Fugro report states that the maximum tidal current speed of 0.91m/s, which occurred at Site A, took place during a period of spring tides that caused the maximum recorded water level at most sites.

Table 2 – Summary of Tidal Current Statistics

Site ID	Current Speed (m/s)	
	Maximum	Minimum
A - AWAC	0.91	0.35
A - ADCP	0.74	0.28
B	0.88	0.32
C	0.72	0.26
D	0.77	0.29
E	0.76	0.29
F	0.68	0.21
G	0.72	0.26
H	0.76	0.23

Source: Fugro Metocean Report Table xvi

By way of comparison and working off the Admiralty Tide Tables for Rosyth and the prediction of the one-year return period tidal current speed, which is 1.21m/s, it is possible to infer an estimate for the peak current speed that could occur on the spring and neap tides. By such an approach, it is estimated that the peak current speed on a typical good spring tide is around 0.95m/s and the corresponding figure for the neap tide is 0.39m/s.

The sediment diameter that is at the point of incipient motion for a current speed of 0.95m/s is around 1.8mm, according to the predictive methods proposed by Soulsby (1997). This result may be independently checked by the application of Van Rijn's solution, for which knowledge of D_{90} is also required. Taking D_{50} of 1.8mm and a D_{90} of 13mm, inferred typically from the grading data for the sediment samples, as shown in Figure 3, the application of Van Rijn's solution provides a threshold of motion of 0.8m/s, which reasonably corroborates the result obtained by applying the Soulsby approach.

Considering the notional neap tide current value of 0.39m/s, the threshold diameter at incipient motion is well below 100μm, according to the Soulsby (1997) solution and may be taken as being outside the range of validity of the method of prediction for Soulsby and also for Van Rijn. It may therefore be concluded that the tidal currents prevalent during a neap tidal cycle would be unlikely to transport the sea bed sediment in any great amount, but that conversely, a spring tide would be able to invoke a significant volume of seabed movement.

Figure 3 shows plots of the grading characteristics of the sediment samples taken from the site. The figure also shows vertical lines representing the sediment diameters corresponding to incipient motion, under the conditions of, from left to right, spring tide and the 1 in 1 year and 1 in 50 year return period tidal currents. The sediment diameter corresponding to the neap tide has not been plotted, as it is believed that the predicted diameter is below the range of classified granular material.

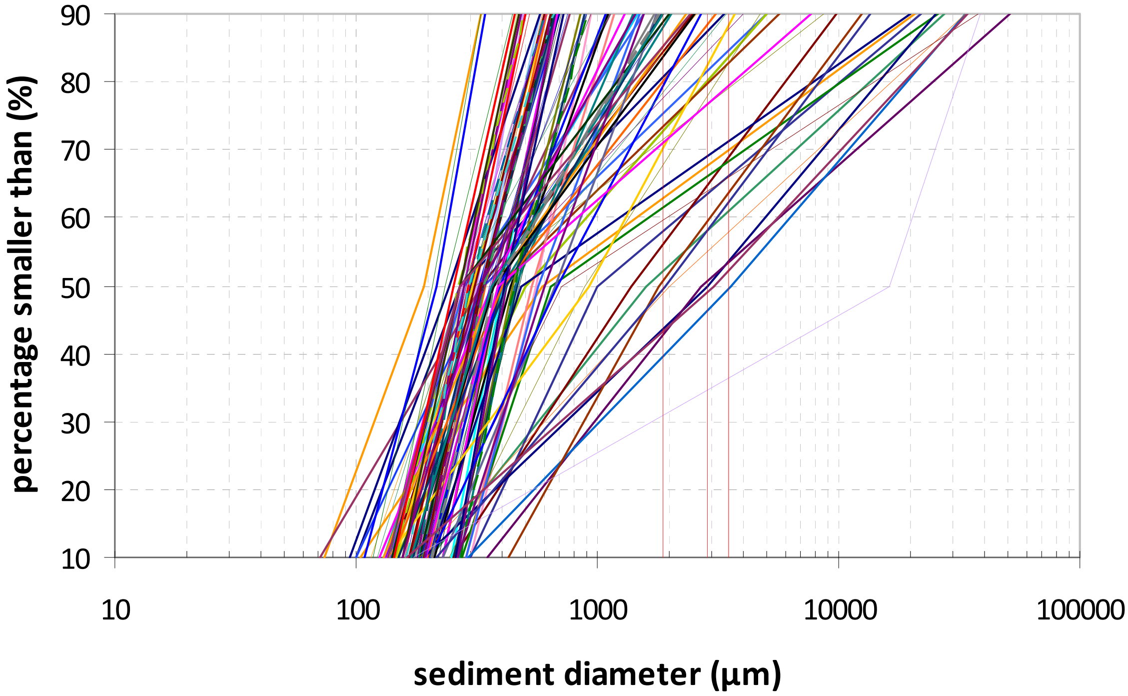


Figure 3 – Critical thresholds of motion for particular sediment grain sizes under, from left to right, mean spring, 1 in 1 year and 1 in 50 year currents.

For the smallest sediment diameter sampled, which is of the order of 200µm at D_{50}, the predicted threshold of motion is around 0.54m/s according to the Soulsby (1997) solution. Using Van Rijn's approach and adopting a typical corresponding D_{90} of 450µm, the predicted threshold of motion is 0.46m/s.

It is therefore believed that the side-cast material, or material released from a dredger hopper back to the seabed, and indeed the sea bed itself, are likely to remain quiescent during neap tide conditions, but under spring tide and storm events, they will experience erosion and transport. Once that process begins, it is clear from Figure 3 that there is insufficient coarse sediment available to provide much in the way of natural armouring and while the material will eventually stabilise during an event, a significant amount of scouring and associated material transport will take place before that occurs.

Summary

Key findings from this assessment are:

- There will be little impact on the physical environment during the installation of piled structures, assuming that industry-standard best practice installation methods are adopted.
- There will potentially be effects associated with the installation of GBS structures, due to seabed preparation process.

- If the dredged material is simply side-cast or returned to the seabed from the dredger hopper (as must be assumed as a 'worst case scenario'), then up to 21,475m^3 of predominantly sands and gravels will arise at each of the 8 no. WTG locations where preparation depth of 5m is needed, and between up to 6,720m^3 at each of the remaining locations where preparation depth of 3m is needed within each of Project Alpha and Bravo. A further 69,500m^3 will cumulatively arise from the rectangular / square GBS that will be used for OSPs at a maximum of 5 locations across the Seagreen Project.

- During neap tides, current velocities are insufficient to mobilise sediment that has been side-cast during these sea bed preparation activities;

- During spring tides (and greater events) sediment that has been side-cast during sea bed preparation activities can readily become mobilised;

- There are very few samples containing substantial amounts of material greater in size than the critical thresholds for particle motion and therefore there will be minimal natural 'armouring' of the sediment that has been side-cast during sea bed preparation activities; and

- The most likely situation is that the material side-cast or returned to the seabed from the dredger hopper will reside in mounds on the sea bed during neap tides and start to become mobilised and dispersed during sprint tides and storm events. Consequently, the entire volume disturbed during sea bed preparation at each foundation / substructure location will not instantaneously be released into the water column.