New England Wind

AVANGRID RENEWABLES

Construction and Operations Plan

Lease Area OCS-A 0534

Volume III Appendices

February 2024

Submitted by
Park City Wind LLC

Submitted to
Bureau of Ocean Energy Management
45600 Woodland Rd
Sterling, VA 20166

Prepared by
Epsilon Associates, Inc.

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Submitted to:
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Submitted by:
Park City Wind LLC

Prepared by:
Epsilon Associates Inc.

In Association with:

Baird & Associates
Biodiversity Research Institute
Capitol Air Space Group
Geo SubSea LLC
Geraldine Edens, P.A.
Gray & Pape
JASCO Applied Sciences
Public Archaeology Laboratory, Inc.
RPS
Saratoga Associates
SEARCH, Inc.
Wood Thilsted Partners Ltd

February 2024

On April 29, 2022, modifications were made to the project design Envelope that involved changing the maximum wind turbine generator (WTG) and electrical service platform (ESP) topside parameters for Phase 1 (Park City Wind) to match those of Phase 2 (Commonwealth Wind) (see Table 1). As a result of this change, the potential minimum footprint of Phase 1 decreased, and correspondingly the potential maximum footprint of Phase 2 increased (see Table 2). Additionally, the maximum capacity in megawatts for both phases was eliminated to accommodate the rapid advancement in commercially available wind turbine generator size and technology.

Table 1 Modifications to the Phase 1 WTG and ESP Parameters[1]

Maximum WTG Parameters	Previous Dimension	New Dimension[2]
Tip Height	319 m (1,047 ft)	357 (1,171 ft)
Top of the Nacelle Height	199 m (653 ft)	221 m (725 ft)
Hub Height	192 m (630 ft)	214 m (702 ft)
Rotor Diameter	255 m (837 ft)	285 m (935 ft)
Minimum Tip Clearance[3]	27 m (89 ft)	27 m (89 ft)
Blade Chord	8 m (26 ft)	9 m (30 ft)
Tower Diameter	9 m (30 ft)	10 m (33 ft)[4]
Maximum ESP Parameters	**Previous Dimension**	**New Dimension[2]**
Width	45 m (148 ft)	60 m (197 ft)
Length	70 m (230 ft)	100 m (328 ft)
Height	38 m (125 ft)	No change
Height of Topside (above MLLW[5])	70 m (230 ft)	No change

1. Maximum WTG dimensions are included in Table 3.2-1 and maximum ESP dimensions are included in Table 3.2-3 of COP Volume I
2. The new Phase 1 WTG and ESP maximum parameters were revised to match those of Phase 2
3. All parameters are maximum values except tip clearance, where the minimum tip clearance represents the maximum potential impact
4. To accommodate the slight increase in tower diameter, the maximum transition piece diameter/width for Phase 1 monopile foundations was also increased from 9 m (30 ft) to 10 m (33 ft) (see Table 3.2-2 of COP Volume I)
5. MLLW: Mean Lower Low Water

To accommodate the larger Phase 1 WTG dimensions and greater capacity range, the minimum footprint of Phase 1 decreased and the maximum footprint of Phase 2 increased, thus also adjusting the potential number of WTG/ESP positions within each Phase (see Table 2).

Table 2 Modifications to the Phase 1 and Phase 2 Layout and Size

		Previous Layout and Size	New Layout and Size
Phase 1	Number of WTGs	50-62	41-62
	Area	182-231 km^2 (44,973-57,081 acres)	150-231 km^2 (37,066-57,081 acres)
Phase 2	Number of WTGs	64-79	64-88
	Area	222-271 km^2 (54,857-66,966 acres)	222–303 km^2 (54,857–74,873 acres)

Additionally, while the Project Design Envelope (PDE) previously included a total of four or five offshore export cables for New England Wind (two offshore export cables for Phase 1 and two or three offshore export cables for Phase 2), the Proponent has confirmed that there will be a total of five offshore export cables (two offshore export cables for Phase 1 and three offshore export cables for Phase 2).

These revisions remain within the maximum design scenario considered for this report and the maximum potential impacts are still representative considering these modifications. Therefore, this report was not updated to reflect these minor modifications, as the findings are not affected.

New England Wind

(Lease Area OCS-A 0534)

Scour Potential Evaluation

Doc. no.: P0073-C1164-011-001D

Revision	Date	Description	Author	Reviewer	Authoriser
A	30/04/2020	Issued for client comment	BDQ	PAG	PAG
B	22/05/2020	Client comments included	BDQ	PAG	PAG
C	06/09/2021	Client requested updates to reflect new site boundary	NRE	JMM	JMM
D	29/11/2021	Client requested updated to reflect change in design envelope	JMM	JMM	PAG

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1. INTRODUCTION

1.1 PROJECT BACKGROUND

New England Wind is the proposal to develop offshore renewable wind energy facilities in Bureau of Ocean Energy Management (BOEM) Lease Area OCS-A 0534 along with associated offshore and onshore cabling, onshore substations, and onshore operations and maintenance (O&M) facilities. Lease Area OCS-A 0534 is within the Massachusetts Wind Energy Area identified by BOEM, following a public process and environmental review, as suitable for wind energy development. Park City Wind LLC, a wholly owned subsidiary of Avangrid Renewables, LLC, is the Proponent and will be responsible for the construction, operation, and decommissioning of New England Wind.

New England Wind will be developed in two Phases with a maximum of 130 wind turbine generator (WTG) and electrical service platform (ESP) positions. New England Wind will occupy all of Lease Area OCS-A 0534 and potentially a portion of Lease Area OCS-A 0501 in the event that Vineyard Wind 1 does not develop “spare” or extra positions included in Lease Area OCS-A 0501 and Vineyard Wind 1 assigns those positions to Lease Area OCS-A 0534. The Southern Wind Development Area (SWDA) is defined as all of Lease Area OCS-A 0534 and the southwest portion of Lease Area OCS-A 0501. Four or five offshore export cables—two cables for Phase 1 and two or three cables for Phase 2, also known as Commonwealth Wind, will transmit electricity from the SWDA to shore. Figure 1-1 provides an overview of New England Wind.

The SWDA may be 411-453 square kilometers (km^2) (101,590 -111,939 acres) in size depending upon the final footprint of the Vineyard Wind 1 Project (also known as 501 North). At this time, the Proponent does not intend to develop the two positions in the separate aliquots located along the northeastern boundary of Lease Area OCS-A 0501 as part of New England Wind. The SWDA (excluding the two separate aliquots that are closer to shore) is just over 32 kilometers (km) (20 miles [mi]) from the southwest corner of Martha's Vineyard and approximately 38 km (24 mi) from Nantucket. Within the SWDA, the closest WTG is approximately 34 km (21 mi) from Martha's Vineyard and 40 km (25 mi) from Nantucket. The WTGs and ESP(s), in the SWDA will be oriented in fixed east-to-west rows and north-to-south columns with one nautical mile (1.85 km) spacing between positions.

Each Phase of New England Wind will be developed and permitted using a Project Design Envelope (PDE). This allows the Proponent to properly define and bracket the characteristics of each Phase for the purposes of environmental review while maintaining a reasonable degree of flexibility with respect to the selection of key components, such as the WTGs, foundations, submarine cables, and offshore substations. To assess potential impacts and benefits to various resources, a “maximum design scenario” is established for each resource based on the characteristics described within the PDE that have the potential to cause the greatest effect. In some cases, this may introduce conservatism into the environmental review process as the maximum design scenario may not be employed.

This assessment utilizes the maximum size of each foundation type included in the PDE for each Phase of New England Wind.

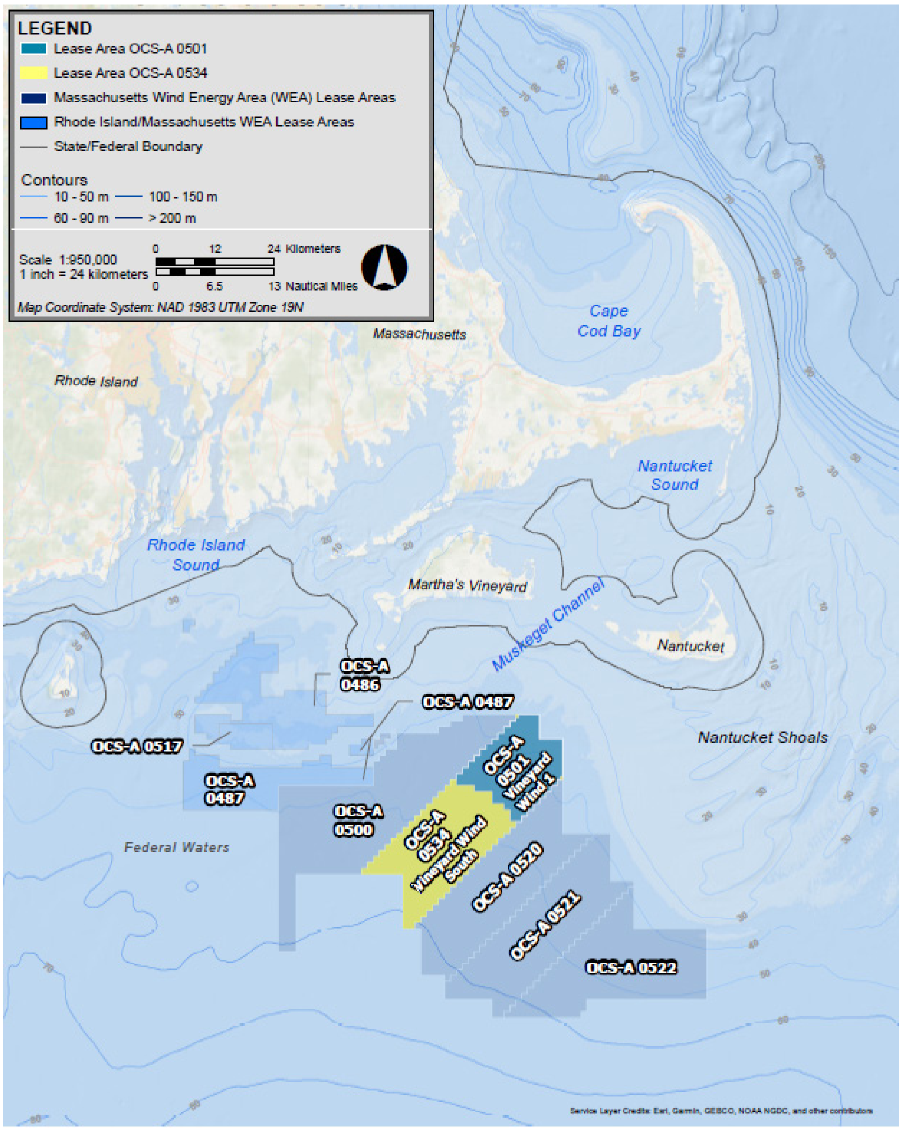


Figure 1-1: Overview of Lease Area Location

The geographic size and number of WTG/ESP positions in the SWDA depends on the final footprint of Vineyard Wind 1. Due to the varying size of Vineyard Wind 1, the SWDA includes an overlap area that was analyzed as part of the Vineyard Wind 1 COP. Figure 1-2 shows the different development areas and potential extents.

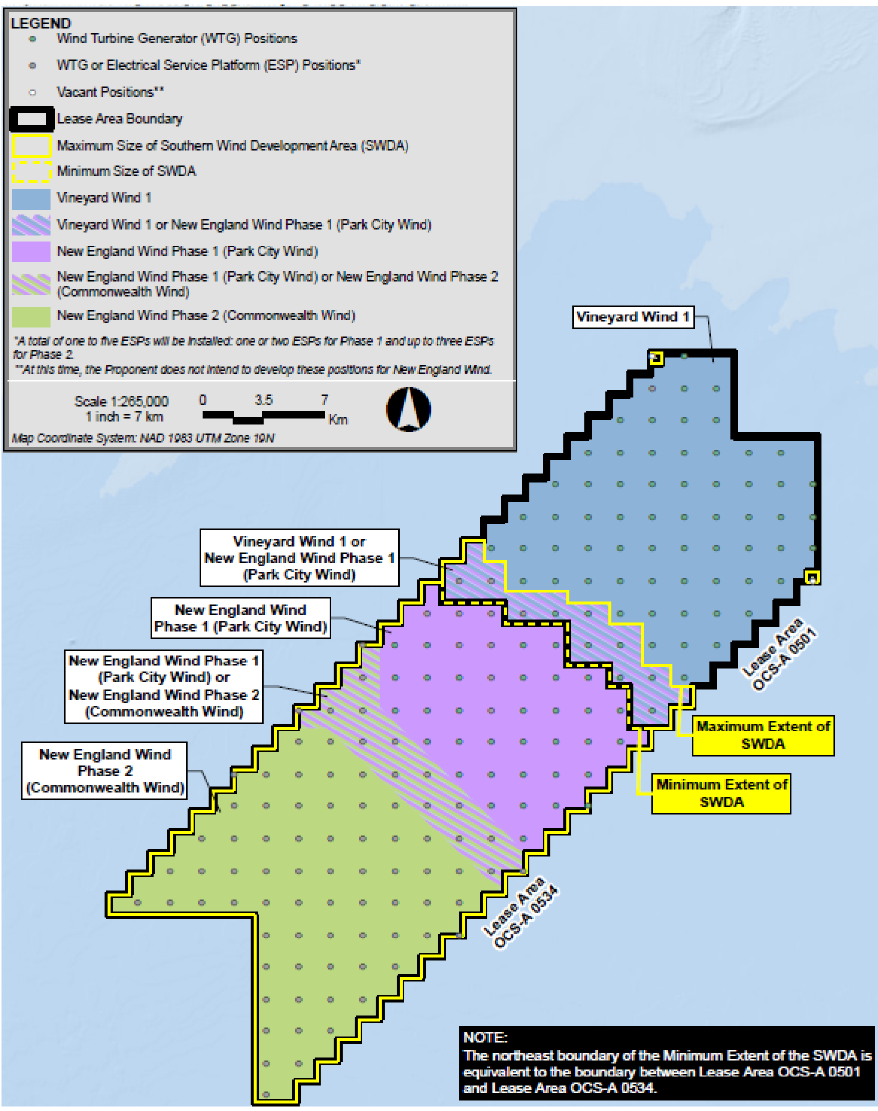


Figure 1-2: SWDA with Potential WTG, ESP Positions in 1NM x 1NM Layout

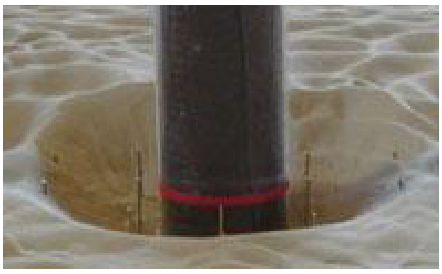
Wood Thilsted Partners Ltd have been appointed by Vineyard Wind to provide technical assistance and expertise in support of the development of the foundations and cable route for New England Wind. This document has been compiled to provide an assessment of the potential scour development resulting from the construction of the planned offshore wind farm. A number of substructure and foundation types are still being considered including, but not limited to, monopiles, piled jackets, suction bucket jackets, and bottom-frame foundations such as TetraBases.

1.2 SCOUR IN OFFSHORE WINDFARMS

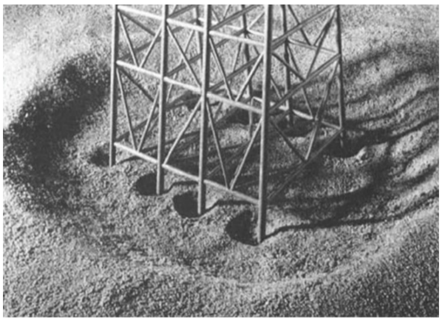
Scour is defined as the removal of sediment from around the base of an object due to the interaction of wave and current-induced flows with a structure and substrate. Any structures constructed as part of marine renewable development such as monopile foundations, jackets for WTGs as well as the cabling necessary for in-field transmission and power export are potential sources for scour at an offshore site. Scour will influence the design of the structure and alter the submerged terrain surrounding the site. Engineers developing such structures therefore have to choose between designing to allow scour to develop and using scour protection to minimize erosion.

Scour can be divided into four different categories as listed below:

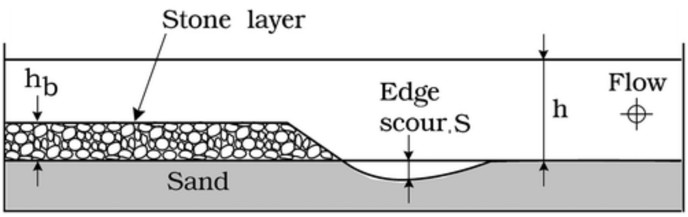
Local scour: The erosion of seabed material in proximity to a single foundation, e.g. a monopile or a single leg of a jacket foundation (Breusers H. N., 1977)



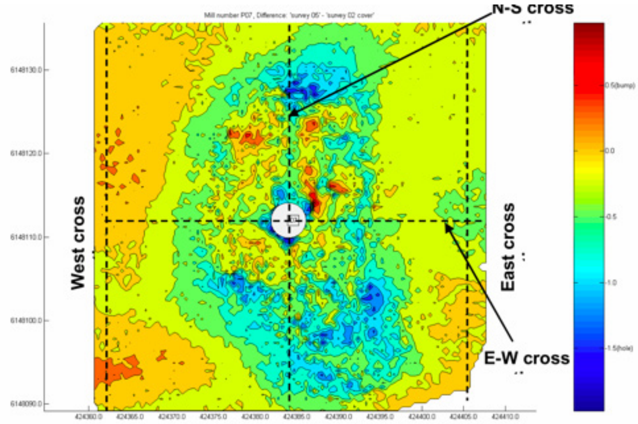
Global scour: The wider erosion around a structure consisting of multiple foundations, e.g. a jacket foundation with 3-4 legs (Judd & Hovland, 2007)



Edge scour (secondary scour): Erosion occurring in proximity to scour protection (Petersen, Mutlu, & David, 2014)



Far field scour: Erosion and deposition occurring at larger distances from the structure and wind farm including overall sea bed movements (Whitehouse R. , 1998)



1.3 THE CAUSES OF SCOUR

Local erosion (scour) can occur at the seafloor around structures due to the interaction between free flow field (combined action of waves and currents) and immersed body (structure). When the free flow field is disturbed by the structure, contraction of the streamlines occurs increasing the flow velocity on the sides of structure. Besides flow contraction, the effect of hydrostatic pressure and wave pressure creates a turbulent flow field around the structure giving rise to horseshoe vortex and lee-wake vortices. The disturbed flow field results in increased flow velocities and turbulent vortices leading to an increase of the bed shear stresses around the structure. Consequently, the sediment transport around the structure increases and local scour may develop. Therefore, the potential development of scour is a function of the seafloor material, hydrodynamic conditions (wave and current speeds) and the MP geometry. Figure 1-3 illustrates the mechanisms of scour development.

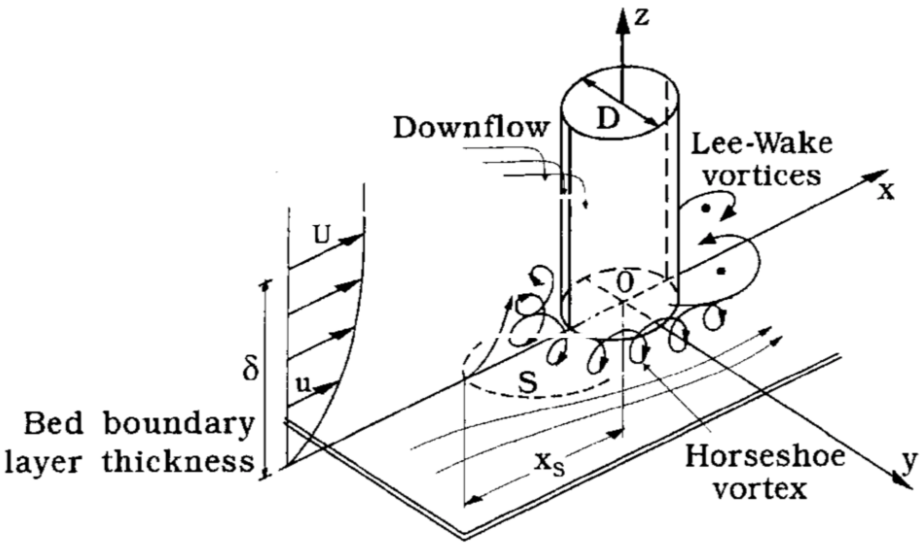


Figure 1-3: Schematic of scour mechanism around a MP.

The key drivers for scour potential of an offshore site include:

- Disturbance of free flow field – any stationary object placed on or with the water column will disturb the free velocity profile of the water flow. The larger the object, the greater this disturbance. This includes permanent foundation structures and temporary jack-up vessels used for installation
- Current / waves – currents and waves are disturbed by the restriction in flow. In combined wave and current scenarios, generally sediment is picked up by waves and transported by currents.
- Mobile seabed surface – Higher scour potential occur when the seabed is mobile. The presence of a rock or stiff clay at or close to the surface will reduce the magnitude of scour as these materials are less susceptible to sediment transport.

1.4 PREDICTING SCOUR

Extensive research has been carried out on predicting and quantifying scour for offshore wind farms. This includes practical considerations based on extensive experience on laboratory experiments and existing windfarm installations (Whitehouse, Sutherland, & Brien, 2006) (Harris, Whitehouse, & Sutherland, 2010) as well as theoretical modelling using computational fluid dynamics (CFD) (Qi & Gao, 2014). Both approaches utilize knowledge of the prevailing conditions of the site as well as the type and dimensions of the offshore structures. However, the accuracy of all such approaches remain modest.

Whitehouse et al. has provided comprehensive summaries on scour forecasting and assessment and prefaces that "*prediction of scour at offshore windfarm foundations in areas with mobile seabeds is a challenging topic*". This is particularly true for areas with shallow water, strong currents and wave action (Whitehouse, Sutherland, & Brien, 2006). Actual field testing is also lacking, meaning that "*scour research has been hampered by a dearth of prototype scour observations and much of the existing knowledge is derived from physical and numerical work which has had very little validation with field data*" (Melling, 2014).

The state-of-the-art of scour assessment and protection design in practice therefore necessitates a holistic and practical approach taking into consideration site conditions in comparison to previous projects, use of general guidelines and application of preventative or remedial measures for cases of uncertainty of scour formation (Zaaijer & Tempel, 2004). This is the approach applied for Vineyard Offshore Wind in this document.

2. THE POTENTIAL FOR SCOUR PROCESSES AT THE SOUTHERN WIND DEVELOPMENT AREA

2.1 SITE CONDITIONS

Mobile seabed and strong hydrodynamics increase the potential for scour. This section assesses both these causes of scour in reference to the proposed development at the SWDA.

Sediment is considered mobile when the bed shear stress (τ) exceeds a threshold shear stress (τ_{cr}). A common method to determine the threshold of motion is through the Shields parameter (Shields, 1936):

$$\theta = \frac{\tau}{g(\rho_s - \rho_w)d_{50}}$$

Where:
θ: Shields parameter [-]
τ: combined wave and current shear stress [N/m^2]
g: gravitational acceleration [m/s^2]
ρ_s: sediment density [kg/m^3]
ρ_w: water density [kg/m^3]
d_{50}: sediment median grain diameter [m]

Sediment will be mobilized when the critical Shields parameter (Soulsby & Whitehouse, Threshold of sediment motion in coastal environments, 1997) value is exceeded:

$$\theta_{cr} = \frac{0.24}{D_*} + 0.055[1 - e^{-0.02D*}]$$

Where D_* is the dimensionless grain size. More information on the calculations of the Shields parameter can be found at Appendix A.

Assuming a hydraulic load amplification factor of 2 to account for the additional turbulence and vortices around the structure, the sediment will become mobile, and thus, scour will potentially develop, when the relative mobility ($^{\theta}/_{\theta_{cr}}$) is higher than 0.5 (Deltares, 2017).

The metocean input parameters necessary for the threshold of motion calculations such as water depth (h), wave height (Hs), wave period (Tp) and flow velocity (U) were obtained from the FLiDAR buoy SAP-1 data. The measurements of wave height, wave period and current from 23rd May 2018 to 23rd April 2020 are presented from Figure 2-1 to Figure 2-2.

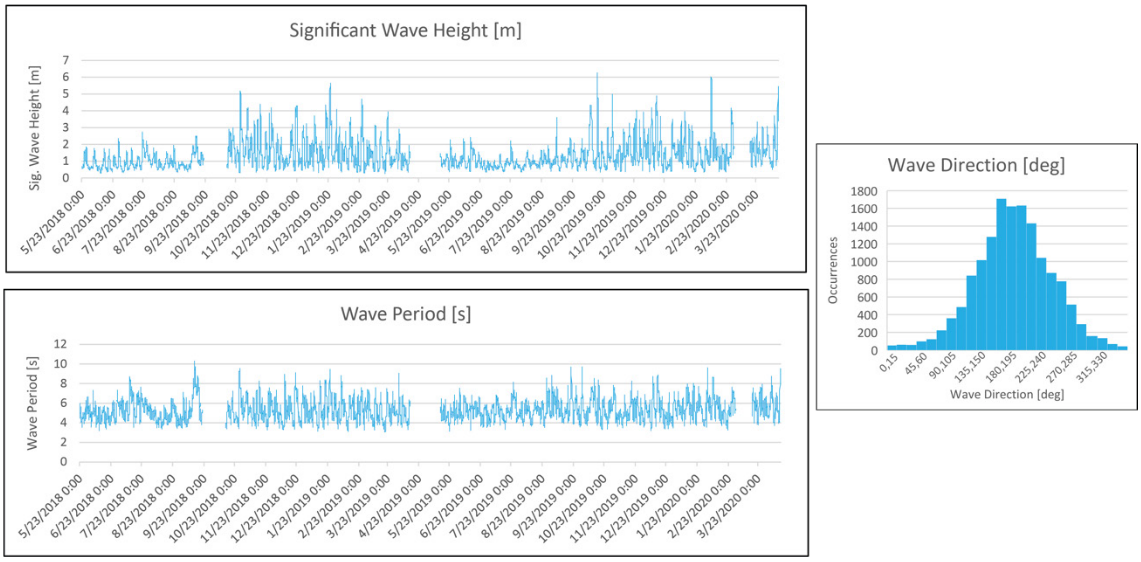


Figure 2-1: Significant wave height and wave period time series and associated wave direction histogram spanning the data observation period for buoy SAP-1.

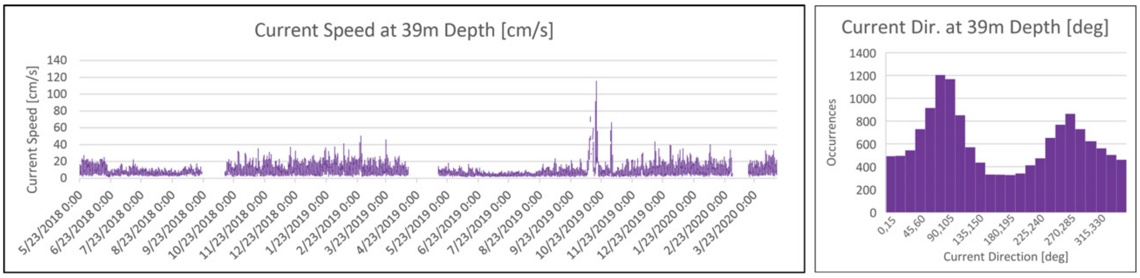


Figure 2-2: Current speed time series and associated current direction histogram spanning the data observation period for buoy SAP-1.

The sediment parameters were obtained from the Geotechnical Interpretation Report (GIR) (Wood Thilsted, 2020) for the focus depth of 20 m below seabed as scour is likely to occur within this range of depth (see Section 2.2.1).The soil up to 20 m bellow seabed is dominated by coarse-grained non-cohesive sediments.

The input parameters adopted in this initial mobility of the seabed for scour potential and the resulting Shields parameters and relative mobility are listed in Table 2-1.

The estimation of relative mobility under normal metocean conditions at the SWDA indicates that under average conditions (current speed of 0.1 m/s) there is no potential for scour development (as $\theta/\theta_{cr} < 0.5$). However, further analysis indicates that there is potential for scour development when above average and extreme conditions with current speeds higher than 0.2 m/s occur.

Table 2-1: Input parameters and outcome Shields and relative mobility parameters of an initial assessment on the potential scour development at the SWDA.

Condition	h [m]	Hs [m]	Tp [s]	U [m/s]	d50 [mm]	ρ_s [kg/m³]	θ	θ_{cr}	θ/θ_{cr}
Normal	40	2	6	0.1	0.226	2650	0.007	0.056	0.13
Above average	40	2	6	0.21	0.226	2650	0.030	0.056	0.53
Extreme	42	2	6	1.2	0.226	2650	1.580	0.056	28.37

Additionally, natural features on the seabed such as ripples, megaripples, sandwaves and obstacles marks (scour and deposition around debris on seabed) can be an indication of scour potential at a basic level (Whitehouse R. J., Harris, Mundon, & Sutherland, 2010).

The ground model developed by WT and Geo SubSea LLC (GSS) indicates the presence of east-west oriented seabed ripples through a central band of the SWDA (Figure 2-3). The presence of ripples confirms the indication of low scour potential at the SWDA given that ripples are rather small bedforms and are an indication of low seabed mobility.

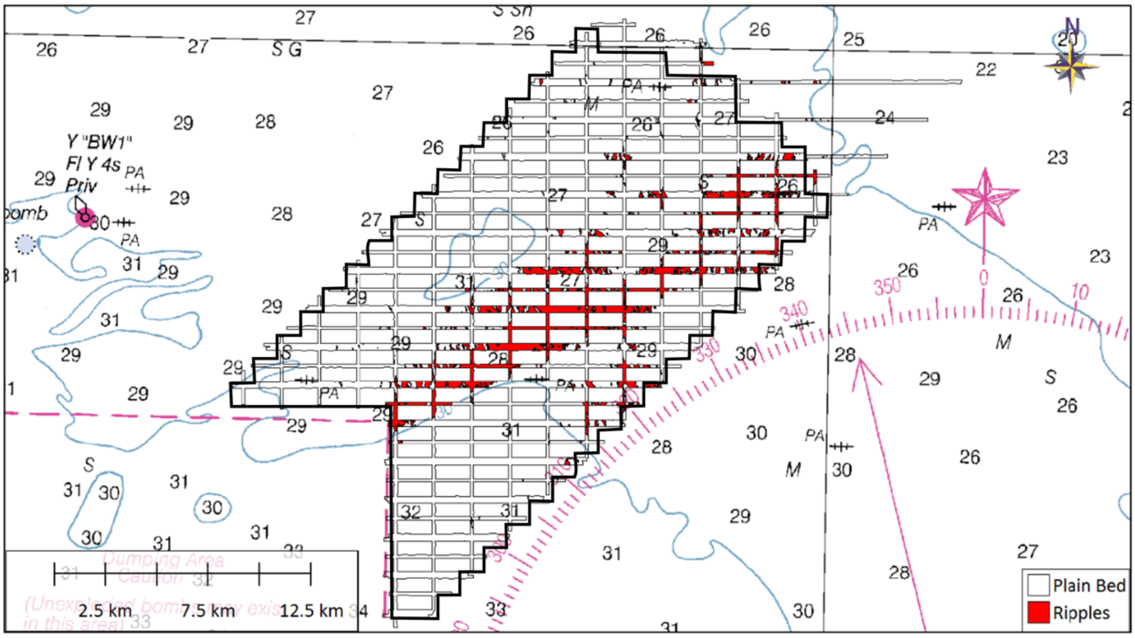


Figure 2-3: Ripples and plain seabed within the SWDA (minimum extent boundary shown in black)

Water depths in the SWDA (excluding the two separate aliquots) generally range from approximately 43 -62 m (141 -203 ft). Figure 2-4 shows the bathymetry across the SWDA. The influence of the waves on sediment transport is less on deeper water depths and the scour potential can be considered to be predominantly controlled by currents (Whitehouse, Harris, Mundon, & Sutherland, 2010).

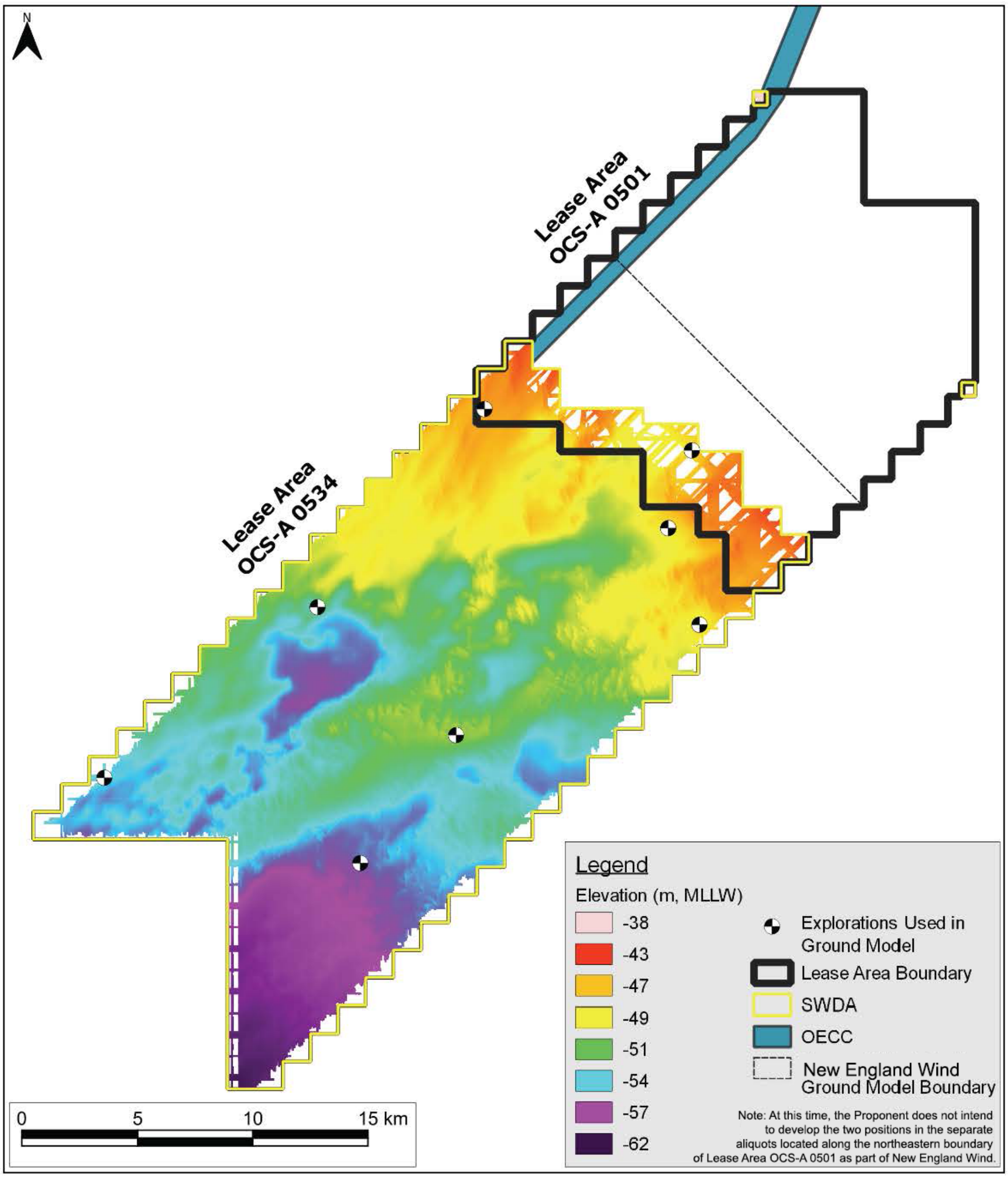


Figure 2-4: SWDA bathymetry (Geo SubSea , 2020)

The current speeds at the SWDA are very low (average of 0.1 m/s), especially if compared to the hydrodynamics of European wind farms such as those in the Thames Estuary off the UK (eg. London Array and Thanet) which have average current speeds of 0.8–1+ m/s (Melling, 2014). This points to a low scour potential at SWDA.

On the other hand, the SWDA site up to 20 m below seabed is predominantly composed of sand with a very low percentage of fines content (Wood Thilsted, 2020) which is favorable for potential scour development within the SWDA. The soil composition below surface is an important factor for scour potential as layers of cohesive soils and hard substrate would decrease the scour potential development.

In general, the SWDA is characterized by a relatively low energy environment. However, the disturbance on the free flow field generated by the structures will increase the seabed mobility locally, potentially developing scour during above average conditions. In addition, the sandy composition of the soil increases the potential for scour development.

2.2 IMPACT ASSESSMENT OF MONOPILES

2.2.1 Local scour

Scour can occur due to both tidal currents and wave induced currents, or a combination of both. In general terms, steady (tidal) currents have more influence on the development of scour than wave-induced currents.

A rule of thumb for scour depth for monopiles is:

$$S = 1.3D$$

Where:
S: Scour depth [m]
D: Monopile diameter [m]

The rule of thumb indicates scour depth of 15.6 m and 16.9 m for monopile (MP) diameters around 12 m and 13 m, respectively. This is a very conservative rule and does not consider the presence of cohesive soils in the SWDA.

Additionally, an initial estimate of the potential scour depth using Sheppard & Miller Jr (2006) and Raaijmakers & Rudolph (2008) formulas of scour depth in coarse-grained non-cohesive soils and extreme metocean conditions observed during the period 16-19 October 2019 (see Figure 2-1 - Figure 2-2 and Table 2-1) obtained from the FLiDAR buoy SAP-1 indicates scour depths of approximately 9 m for MP diameters ranging from 12-13 m. More details on the formulas can be found at Appendix B.

A plot of the relationship between water depth and scour is shown in Figure 2-5– with local scour data from three windfarms situated in the Thames Estuary off the UK. These windfarms are all situated in an area where there are much higher currents than predicted at Vineyard, where the currents in the Thames Estuary are 0.8-1 m/s and the currents in the SWDA are on average 0.1 m/s. Monopiles at Vineyard are expected to have a diameter of up to 12m (for Phase 1) and up to 13m (for Phase 2) and this combined with the water depth ranges gives h/D ratios (water depth h over pile diameter D) of 3 to 5 for this project. This is indicated on Figure 2-5 in blue. Figure 2-5 indicates a scour depth (S) of around 1.3 times the pile diameter. For a 13 m diameter monopile, this equates to a scour depth of 16.9 m. This is a quite conservative value, similar to the rule of thumb.

Figure 2-6 illustrates the dimensions of the scour when allowed to freely develop assuming Sheppard & Miller Jr (2006) and Raaijmakers & Rudolph (2008) formulas which results in a scour depth of 9m considering a MP diameter of 13m. The scour pit would be developed through a process where a) strong vortex currents in close proximity the pile will initially erode the sediment close to the pile and then b) the scour pit would gradually be filled by sand farther from the pile which would then be removed by vortex currents and then the process would repeat. The slope of the freely developing scour is assumed to be equal to the angle of repose of the soil which in the SWDA is predominately composed by sand, thus, the angle of repose is assumed to be 30°.

To prevent the scour pit process from developing, the radial extent of the scour protection must be sufficient to block the strong vortex currents in close proximity to the pile. This will prevent the scour process from developing. Therefore, the radial extent of the scour protection is significantly less than the radius of a freely developing scour pit.

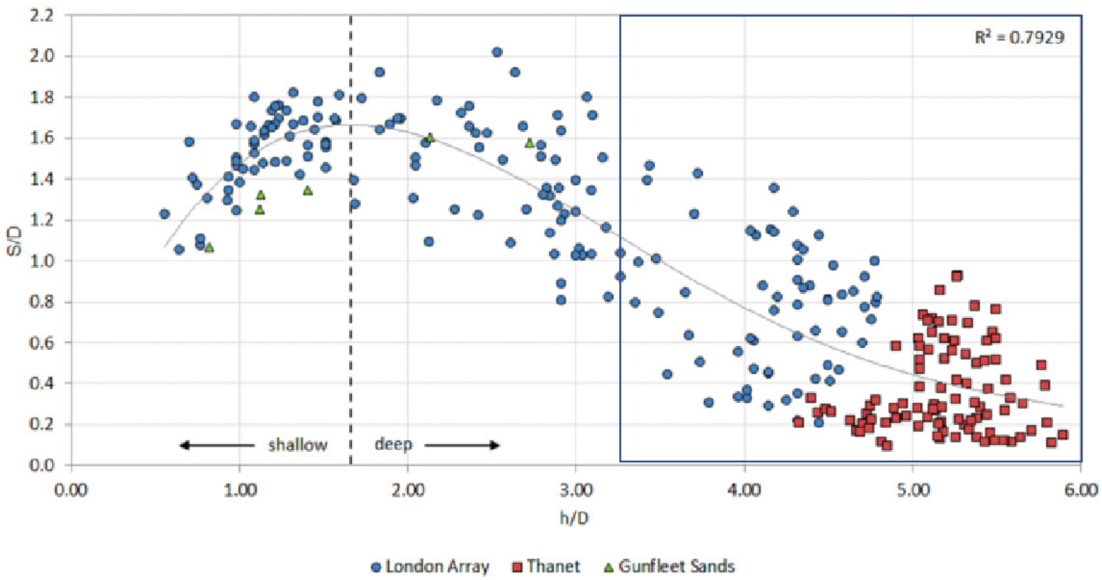


Figure 2-5 A plot of the scour vs monopile diameter (S/D) and normalized water depth (h/D) for three windfarm locations located in the UK. The blue area is representative for 13 m monopile foundations at the SWDA.

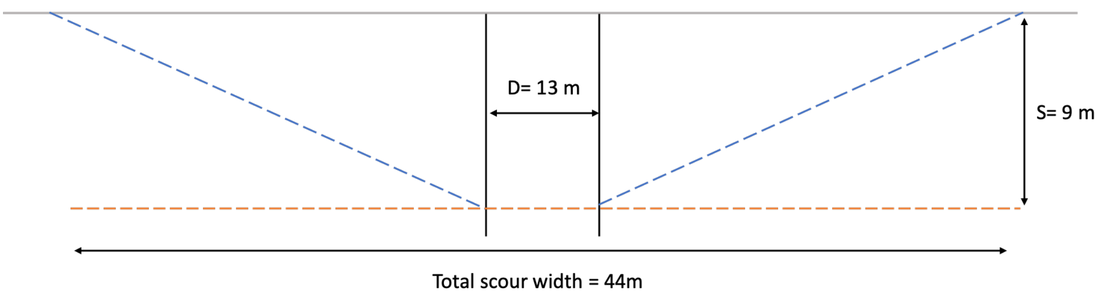


Figure 2-6: Sketch of the potential freely developed scour at the SWDA for a 13 m monopile.

2.2.2 Scour protection

Whilst it would be possible to design for such scour depths as shown in Figure 2-6, in practice they are unwanted as it results in an uneconomical design, large scour holes and technical challenges due to large shifts in the natural frequency of the structure (van der Tempel, Zaaijer, & Subroto, 2004). Therefore, it is considered prudent to allow for placement of scour protection around all monopile foundations.

Typically, the scour protection around monopiles is composed by an armour layer and a filter layer, named double grading. The filter layer prevents the loss of underlying sediment while the armour prevents the erosion of filter material by securing it against movement due to hydrodynamic loading induced by waves and currents. Alternatively, a single grading scour protection system comprises of an armour layer-only that also functions as a filter layer by increasing its thickness. For the purpose of this document, a double grading scour protection system is considered.

The lateral extent of the armour layer is based on the hydrodynamic load of the horseshoe and lee-wake vortices generated by the interaction of the flow field with the structure. Horseshoe and lee-wake vortices have a maximum shear stress on the armour layer of about half the diameter from the MP wall. Generally, the armour top diameter is 3 to 4 times the MP diameter with a side slope of 2:1.

To allow edge scour, a falling apron allowance is provided by extending the filter layer beyond the armour layer extent. The underlying filter layer is extended 0.5 x D beyond the armour layer to provide reserve material to form a falling apron against edge scour advancement.

For Project Envelope purposes it is assumed that the maximum extent of the scour protection around MPs is 6 x D which includes some conservatism to account for ultimate design engineer personal preferences, contractor inaccuracies, migration of material and replenishment and etc.

The thickness of the scour protection system (SPS) depends on the type of SPS (e.g. double-grading or single-grading) and its primary function is to avoid the loss of seabed sediment through the voids between the rocks, known as winnowing failure. Scour protection thickness up to 1-2 m are frequently seen in Europe and these thicknesses have therefore been considered the upper bound envelope for the scour protection sizing.

Table 2-2 and Table 2-3 list the scour protection dimensions considering a double grading SPS with maximum extent of 6 x D for Phase 1 and Phase 2, respectively. The rock weight estimates assume rock density of 2.6ton/m^3 and porosity of n=0.4 and n=0.6 for armor and filter layer, respectively.

Table 2-2: Dimensions and volumes estimates for the Phase 1 Envelope of scour protection for MP at the SWDA.

Layer	Dimensions	Values
	MP Diameter [m]	12
Armor	Top radius [m]	28

	Bottom radius [m]	30
	Thickness [m]	1
	Volume [m^3]	2,530
	Rock Weight [ton]	3,947
Filter	Radius [m]	36
	Thickness [m]	1
	Volume [m^3]	3,958
	Rock Weight [ton]	4,177
	Total Seabed Area [m^2]	**4,072**

Table 2-3: Dimensions and volumes estimates for the Phase 2 Envelope of scour protection for MP at the SWDA.

Layer	Dimensions	Values
	MP Diameter [m]	13
Armor	Top radius [m]	30.5
	Bottom radius [m]	32.5
	Thickness [m]	1
	Volume [m^3]	2,986
	Rock Weight [ton]	4,657
Filter	Radius [m]	39
	Thickness [m]	1
	Volume [m^3]	4,646
	Rock Weight [ton]	4,831
	Total Seabed Area [m^2]	**4,778**

2.2.3 Global scour

Global scour is not applicable for monopiles, as there is only one pile disturbing the water column. In design, some global scour allowance may be appropriate to account for the survey tolerances / natural variation in water depths.

2.2.4 Edge scour

The presence of a structure and scour protection cause flow contraction and turbulence increasing the sediment transport capacity. Because part of the seabed around the MP is protected by the SPS against sediment pickup, a large gradient in sediment transport occurs at the edge of the scour protection and the sediment pickup capacity just outside the scour protection is increased. This leads to the potential development of edge scour depending on sediment transport characteristics.

Edge scour depths also termed secondary scour around protection structures are recorded to be less than 3m for a range of different conditions and scour protection structure types in an extensive study carried out by Petersen et al. (Ugelvig Petersen et al., 2014). Whitehouse et. al. studies (2011) analysed data of edge scour at offshore wind turbine foundations in several locations and reports edge scour predominantly in the range of 0.2-1.2 the MP diameter.

When designing the SPS flexibility is taken into consideration to account for edge scour. Usually, the filter layer is extended by 0.5 times the MP diameter to provide reserve material to form a falling apron against edge scour advancement.

2.2.5 Far field scour

Far-field scour has less clear guidelines available to determine its extent given that ddefining the extent of scour by identifying a position in which there is no change to depth is impractical, as distinguishing between "*significant changes in bed elevation and bed morphology*" and areas where "*little change happened*" is unfeasible (Melling, 2014). Melling et al. concludes that there is no current answer for "*how the extent of a scour hole is defined and what objective criteria can be used to delimit its boundary*".

However, field measurements provide useful information for the estimation of far-field scour. Full-scale measurements of local scours for an entire offshore wind farm is provided by Hansen et al. for Horns Rev 1 (Hansen, Nielsen, Høgedal, Simonsen, & Pedersen, 2006). Erosion here was evaluated for all the monopiles with scour protection and at relatively large distances from unprotected monopiles. Here, only edge scour is relevant and far-field analysis indicates that at large distances, re-deposition of soil is as prevalent as erosion. The graph in Figure 2-7 from Hansen et al. taken shows seabed profiles at distances of up to 25m from the MP in which the black line is the depth from the pre-installation survey (2002) and the cyan line is from the post-installation survey (2005). The sand bed east of turbine number was approximately 0.3 m eroded, whereas backfilling in the order of 0.2 m has occurred on the western side.

Figure 2-7. Measured seabed changes from 2002 to 2005, Horns Rev 1 Offshore Wind Farm

Additionally, a programme of research and monitoring was undertaken at the Scroby Sands OWF by (Cefas, 2006), to observe, measure and quantify potential impacts of OWFs on coastal processes that may lead to disturbance of sedimentary environments or sediment transport. It was concluded that in the range 0-100 m from foundations, the seabed impact was scour holes as predicted. In the range 100-1000 m from foundation, the seabed impact was scour wakes, but was not significant with respect to the total bank volume change. Above 1000m there were no evidence of any impact. Much less overall seabed impact is expected at the SWDA as the scouring potential is significantly less than at Scroby sands due to deeper water and lower current velocities.

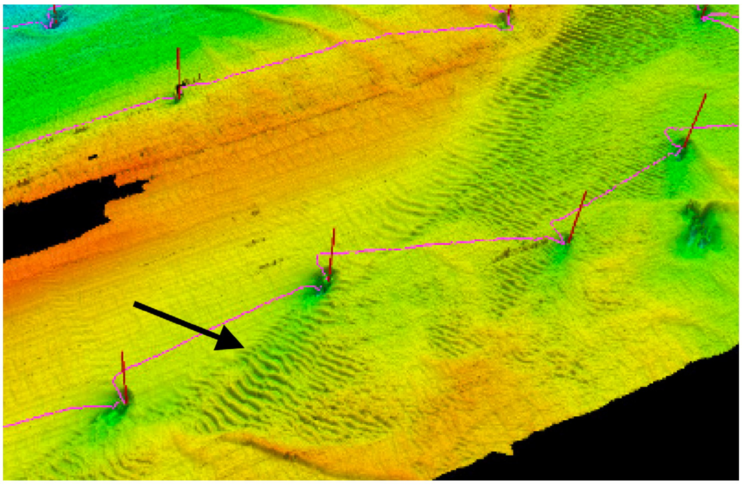
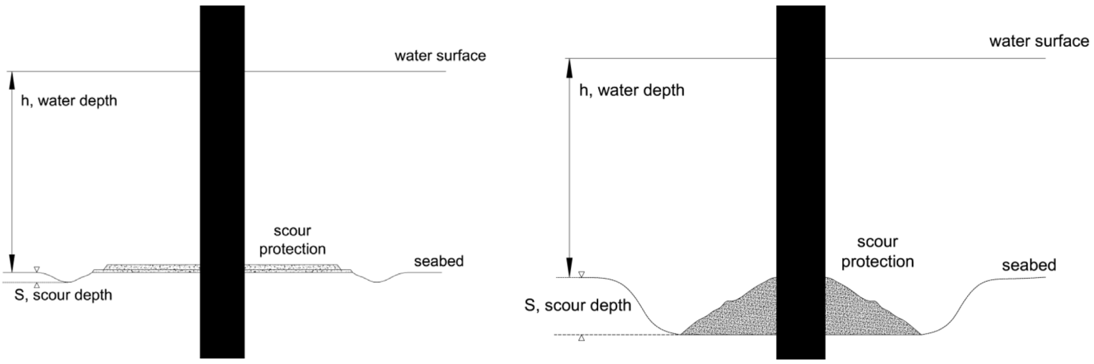


Figure 2-8. Results from the swath bathymetry survey of Scroby Sands OWF. The red cylinders are the MP and the intra-array cable route is shown as magenta. The black arrow shows a far field scour wake extending towards southeast. (Cefas, 2006).

It is important to note that at Scroby Sands OWF, the scour protection was only placed *after* the scour hole had developed as illustrated in Figure 2-9. This resulted in larger edge scour which lead to the occurrence of locally extensive scour wakes with larger amplitude seabed features than the surrounding seabed. There are no reports available of similar significant far-field seabed disturbance for conventional scour protection systems being installed prior to pile installation, as planned for New England Wind. In addition, at some extent the flexibility of the SPS through a falling apron accounts for far-field scour.



Conventional scour protection - the most common practice and the planned scour protection for New England Wind: The scour protection is placed *before* scour hole is allowed to develop. Secondary scour depth approximately 0.12 times diameter (Whitehouse R. J., Harris, Sutherland, & Rees, 2011)

Unconventional scour protection solution applied at Scroby Sands OWF and Arklow Banks: Placement of scour protection *after* the scour hole had developed. Secondary scour depth up to 1.6 times diameter (Whitehouse R. J., Harris, Sutherland, & Rees, 2011)

Figure 2-9. Placement of scour protection before (left) or after (right) development of scour hole

2.3 IMPACT ASSESSMENT JACKETS

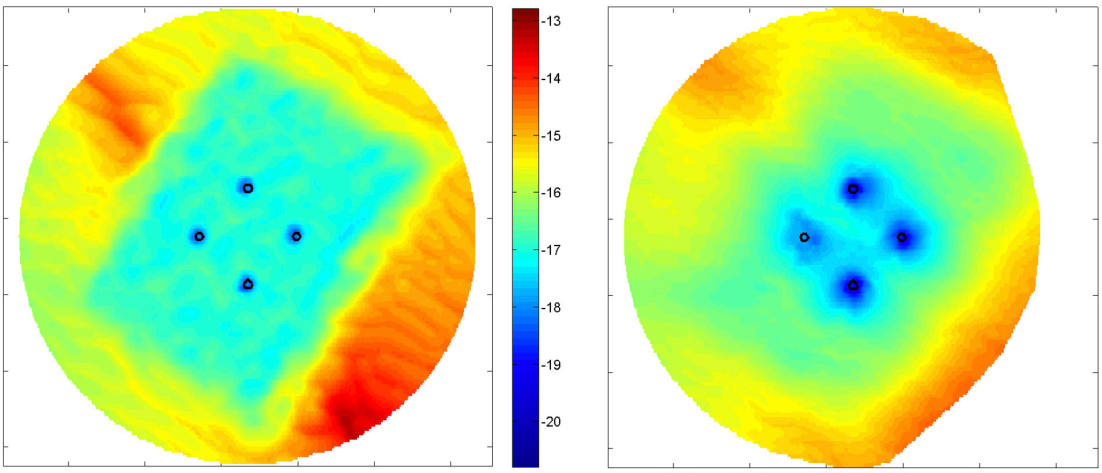
2.3.1 Introduction

This section includes the scour potential evaluation for piled jackets and suction bucket jackets (SBJ).

2.3.2 Local scour

Local scour is considered the scour that occur locally around each of the jacket piles. The depth of the local scour holes is expected to be approximately 1.3 times the pile diameter according to (DNVGL-ST-0126, 2016), however, in extreme cases, a depth of 2.0 times pile diameter may occur (Sumer & Fredsøe, 2002).

Experience from scour development around 4-legged jacket foundation a Thornton Bank Phase 2-3 Offshore Wind Farm in Belgium is illustrated in Figure 2-10 (note that seabed dredging in this case was carried out prior to jacket installation). Four months after the jacket installation, the average scour depths ranged between 1.4 and 1.9 m (0.7 – 0.95 times the MP diameter) and the largest scour depth at each location (maximum of the four piles) ranged between 1.7 and 2.7m (0.85 – 1.35 times the MP diameter).



02.08.2011 shortly after dredging and pile installation

14.01.2012 shortly after a storm event

Figure 2-10. Monitoring of scour development around a four-legged jacket at Thornton Bank Phase 2-3 Offshore Wind Farm in Belgium. (Bolle, De Wintee, Goossens, Haerens, & Dewaele, 2012)

The scouring potential at the SWDA is less severe than at Thornton Bank, and therefore the guidance 1.3 times the pile diameter (DNVGL-ST-0126, 2016) is considered well suited for scour estimation. For piles with maximum diameters of around 4 m for Phase 1 or Phase 2, this equates to a local scour depth of approximately 5 m around the jacket piles. With a slope angle of 30 degrees (typical sand repose angle), this equates to total width of around 22m per pile. Figure 2-11 illustrates the dimensions of the local scour around the jacket piles.

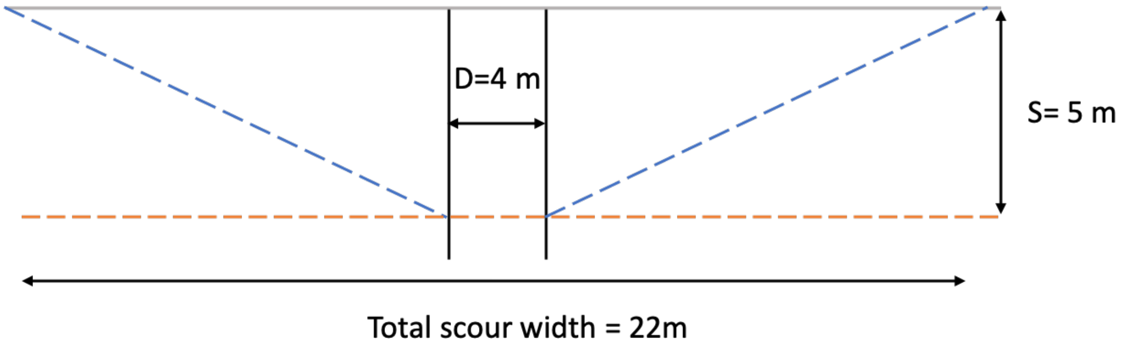


Figure 2-11: Sketch of the potential local scour at the SWDA jacket piles with diameter of 4m.

For suction bucket jackets (SBJ), the development of scour is dependent on the following characteristic of the SBJ (Deltares, 2017):

- The vertical stick-up height of the buckets after installation;
- Additional pipping and anodes attached to the roof of the suction buckets;
- The connection between the buckets and jackets legs and its obstruction to the flow field (transparency and smoothness);
- Diameter and distance from the seabed of the jacket tubes;
- Orientation of the structure with respect to the main flow direction – contracted flow or shed vortices from the upstream leg(s) can increase the scour potential at the downstream leg(s) resulting in asymmetric scour patterns.

Considering the bucket diameter of 15m, vertical stick-up height of the buckets of 5 m, total SBJ height within the water column of 60 m, upper piles diameter of 4 m, this leads to a local scour of approximately 7m assuming 1.3 x D.

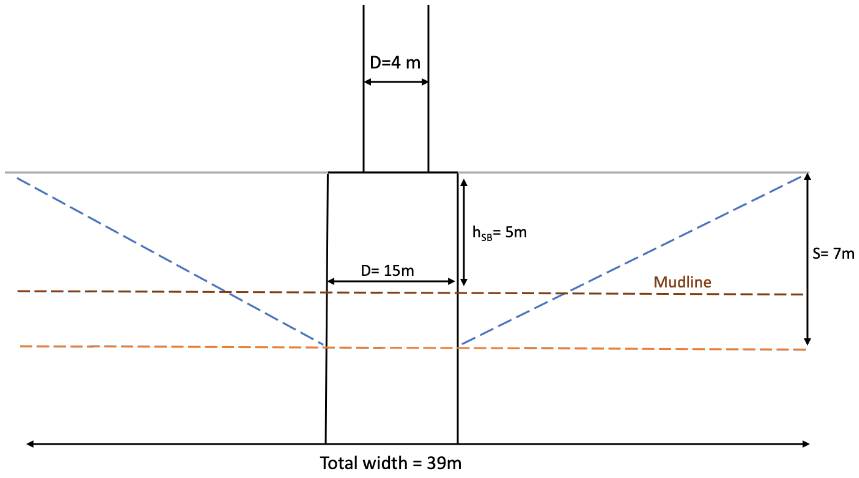


Figure 2-12: Sketch of the potential local scour at the SWDA SBJ with bucket diameter of 15m, vertical stick-up height of the buckets of 1.1 m, and jacket pile diameter of 2 m.

2.3.3 Global scour

When piles are grouped together with transverse or diagonal braces connections such as in jackets structures, variations in the fluid velocity between the piles such as streamline compression, turbulence and shielding effects can occur generating global scour. The total scour refers to the scour due to both local and global scour. The gap-to-diameter ratio and the transparency of the structure close to the seabed affect the flow field and turbulence thus, influencing on the scour depth and extent around a jacket structure (Welzel , Schendel, Hildebrandt , & Schlurmann, 2019).

Field observations of scour depths around a 4-legged OWF jacket structure at Thornton Bank pointed out that the maximum total scour depth observed was around 1.6 the pile diameter (D) and the contribution of the jacket superstructure to the global scour was around 0.4D, based on scour observations before and after placement of the jacket structure onto the pile group (Baelus, Bolle, & Szengel, 2019) and (Bolle, De Wintee, Goossens, Haerens, & Dewaele, 2012) (Figure 2-13).

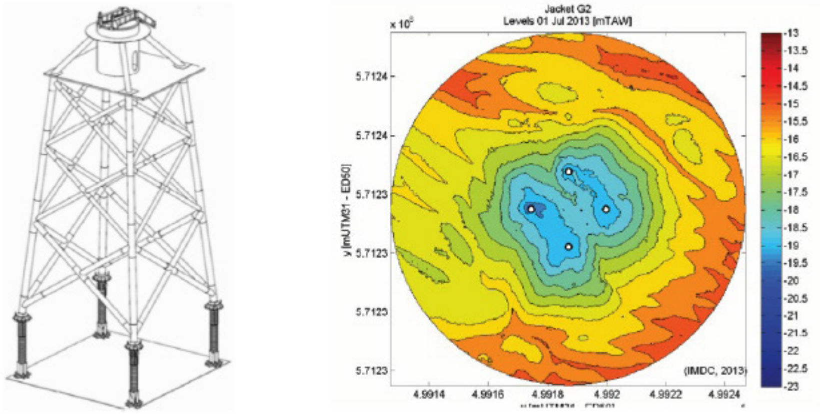


Figure 2-13: Jacket structure at Thornton Bank (left) and scour depth (right) from field measurements (Baelus, Bolle, & Szengel, 2019)

Figure 2-14 presents the total scour depth in relation to the scour depth around piles against the gap-to-diameter ratio (G/D) obtained from literature on current generated scour for groups of 2-3 piles and for a 4-legged jacket (Welzel , Schendel, Hildebrandt , & Schlurmann, 2019) (Li, Qi, & Gao, 2016) (Sumer & Fredsøe, 2002). According to Figure 2-14 it is possible to estimate that the global scour depth for a 4-legged jacket structure is approximately 25% of the local scour depth resulting in a global scour of approximately 1.25 m.

For the 3-legged jackets and the suction bucket jacket (3 buckets) included in the Envelope for New England Wind with G/D ~ 9 a global scour of 10% of the local scour depth can be estimated from Figure 2-14. This results in a global scour depth of approximately 0.5-0.7 m.

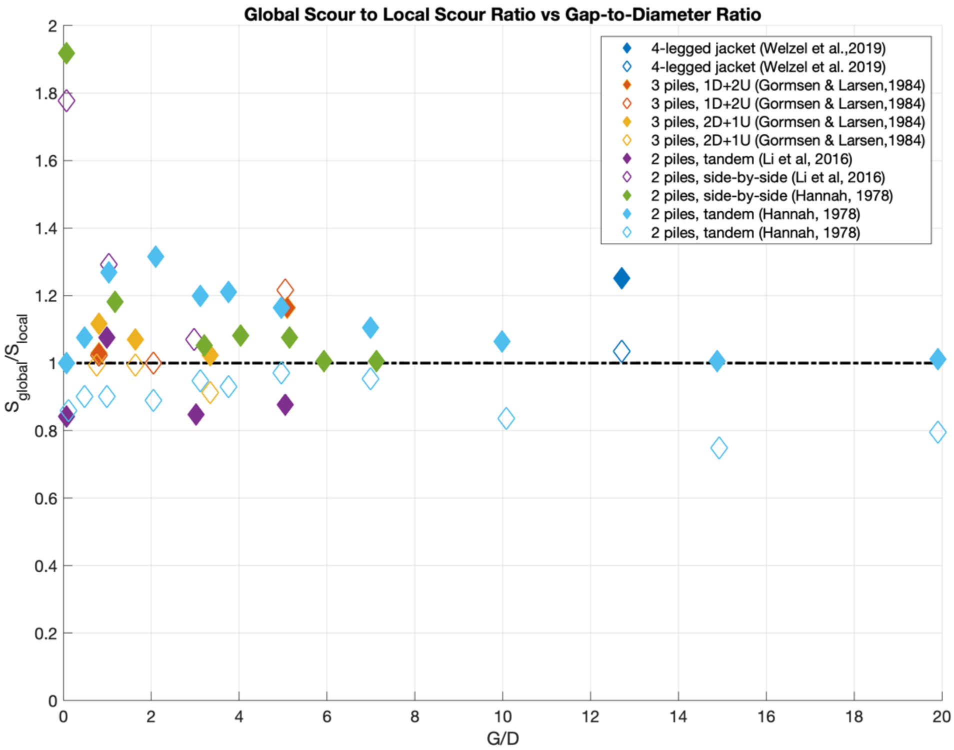


Figure 2-14: Scour depth relative to scour depth of MP against G/D ratio from experimental studies of scour around group of piles and jacket structure. Filled symbols are upstream pile and open symbols are downstream pile. For the 3-pile group, “2D+1U” means 2 upstream and 1 downstream and “1D+2U” means the opposite.

2.3.4 Scour protection

Typically, the scour protection around jackets is composed by a filter layer that extends up to 4 x D (piled jackets) or 1 x D (suction bucket jackets) around the jacket structure footprint and an armor layer around the piles of the jacket. For Project Envelope purposes it is conservatively assumed that the filter layer extends 6 x D (piled jackets) or 1.8 x D (suction bucket jackets), the armor radial extent is estimated to be 2 x D (piled jackets) or 1-1.5 x D (suction bucket jackets) and the height is on average 1 m for the filter layer and armor, resulting in a total height of 2m. Figure 2-15 illustrates a typical scour protection around SBJ.

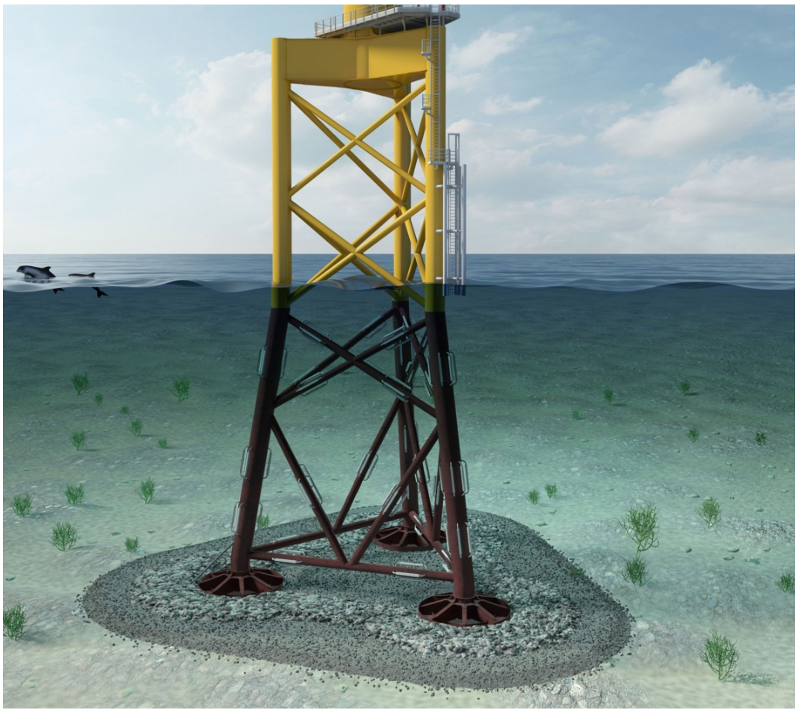


Figure 2-15: Typical scour protection around SBJ.

Table 2-4 to **Table 2-7** list the estimated scour volumes and rock weight for the different WTG and ESP jacket foundation types considered in Phase 1 and Phase 2 Project envelope for New England Wind respectively. A rock density of 2.6 ton/m^3 and porosity (n) of 0.4 for armor and 0.6 for filter are assumed.

Table 2-4: Estimates of Phase 1 scour protection dimensions for the WTG jacket foundations considered for New England Wind.

Characteristics	Jacket (3 Piles)	Jackets (4 Piles)
External dist. between legs [m]	40	40
Pile/Bucket Diameter [m]	4	4
Number of legs [-]	3	4
Extension Filter Layer [m]	12	12
Area of Filter Layer [m^2]	2,585	4,624
Height of Filter Layer [m]	1	1
Volume Filter Layer [m^3]	2,585	4,624
Rock Weight Filter Layer [ton]	2,689	4,809
Radial Extent Armor Layer [m]	8	8
Area (incl. pile/bucket area) per leg -Armor Layer	804	804

Area (excl. pile/bucket area) per leg - Armor Layer	792	792
Height of Amour Layer [m]	1	1
Volume per leg - Armor Layer [m³]	792	792
Rock Weight per leg - Armor Layer [ton]	1,235	1,235
Volume per jacket structure - Armor Layer [m³]	2,375	3,167
Rock Weight per jacket structure - Armor Layer [ton]	3,705	4,940
Total Seabed Area [m²]	**2,585**	**4,624**

Table 2-5: Estimates of Phase 2 scour protection dimensions for the different jacket structures considered for New England Wind.

Characteristics	Jacket (3 Piles)	Jackets (4 Piles)	Suction Bucket Jacket
External dist. between legs [m]	40	40	40
Pile/Bucket Diameter [m]	4	4	15
Number of legs [-]	3	4	3
Extent Filter Layer [m]	12	12	27.5
Area of Filter Layer [m²]	2,585	4,624	6,369
Height of Filter Layer [m]	1	1	1
Volume Filter Layer [m³]	2,585	4,624	6,369
Rock Weight Filter Layer [ton]	2,689	4,809	6,623
Radial Extent Armor Layer [m]	8	8	20
Area (incl. pile/bucket area) per leg -Armor Layer	804	804	2,376
Area (excl. pile/bucket area) per leg - Armor Layer	792	792	2,199
Height of Amour Layer [m]	1	1	1
Volume per leg - Armor Layer [m³]	792	792	2,199
Rock Weight per leg - Armor Layer [ton]	1,235	1,235	3,431
Volume per jacket structure - Armor Layer [m³]	2,375	3,167	6,597
Rock Weight per jacket structure - Armor Layer [ton]	3,705	4,940	10,292
Total Seabed Area [m²]	**2,585**	**4,624**	**6,369**

Table 2-6: Estimates of Phase 1 scour protection dimensions for the ESP jacket foundations considered for New England Wind.

Characteristics	Piled Jacket (3-12 Piles)
Length [m]	70
Width [m]	45
Pile/Bucket Diameter [m]	4
Extent Filter Layer [m]	12
Area of Filter Layer [m^2]	6,023
Height of Filter Layer [m]	1
Volume Filter Layer [m^3]	6,023
Rock Weight Filter Layer [ton]	6,264
Radial Extent Armor Layer [m]	8
Area (incl. pile area) per leg - Armor Layer	201
Area (excl. pile area) per leg -Armor Layer	188
Height of Amour Layer [m]	1
Volume per leg - Armor Layer [m^3]	188
Rock Weight per leg - Armor Layer [ton]	294
Total Seabed Area [m^2]	**6,023**

Table 2-7: Estimates of Phase 2 scour protection dimensions for the ESP jacket foundations considered for New England Wind.

Characteristics	Piled Jacket (3-12 Piles)	Suction Bucket Jackets (3 Buckets)
Length [m]	100	100
Width [m]	60	60
Pile/Bucket Diameter [m]	4	15
Extent Filter Layer [m]	12	27.5
Area of Filter Layer [m^2]	9,953	17,176
Height of Filter Layer [m]	1	1
Volume Filter Layer [m^3]	9,953	17,176
Rock Weight Filter Layer [ton]	10,351	17,863
Radial Extent Armor Layer [m]	8	15
Area (incl. pile area) per leg - Armor Layer	201	707
Area (excl. pile area) per leg -Armor Layer	188	530
Height of Amour Layer [m]	1	1
Volume per leg - Armor Layer [m^3]	188	530
Rock Weight per leg - Armor Layer [ton]	294	827
Total Seabed Area [m^2]	**9,953**	**17,176**

2.3.5 Edge scour

Please refer to 2.2.4.

2.3.6 Far field scour

Please refer to 2.2.5.

2.4 IMPACT ASSESSMENT BOTTOM-FRAME FOUNDATIONS

2.4.1 Local scour

Bottom-Frame foundations (BF) are a relatively new foundation concept and to WT's knowledge there are no available data on scour around this type of structure. Nevertheless, it is expected that the scour mechanism would be similar to the jacket structures (Section 2.3). Local scour could potentially develop around the 3 radial tubes (in a similar way to cables and pipelines laid on the seabed), the center column and the 3 external piles or suction buckets. The development of local and to a certain extend global scour on TB foundations is dependent on the following characteristics of the TB structure:

- The diameter of the radial tubes and center column
- Transparency and smoothness in relation to the obstruction of the flow field by the structure through its center column, diagonal and radial tubes (diameter and distance between them)
- The vertical stick-up height of the buckets after installation, when suction bucket BF
- The diameter and height of the foundation piles, when piled BF
- The height of the radial and center column from the seabed
- The orientation of the structure with respect to the main flow direction – contracted flow or shed vortices from the upstream leg(s) can increase the scour potential at the downstream leg(s) resulting in asymmetric scour patterns.

It is expected that BF with suction buckets will have a similar scour potential of piled BF because even though the suction buckets are larger structures than the TB piles, they only block partially the water column. For the purpose of the present document it is considered that local scour depths of 5-7m can potentially occur at BF foundations at the SWDA similar to the local scour depth of jacket structures (Section 2.3.2).

2.4.2 Global scour

The global scour around BF foundations are expected to be minimal given that the mechanism is similar to jacket structures and the distance between adjacent legs is much higher than the SBJ. This means that there will be relatively more smoothness and transparency to the flow with TB foundations.

Therefore, it is considered reasonable to assume a potential global scour depth of approximately 1 m at BF foundations in the SWDA given that global scour depth around jacket foundations for the SWDA is estimated to be around 0.5-1.25 m.

2.4.3 Scour protection

The BF is a new foundation concept and to WT's knowledge there is no available information on scour protection around BF foundations. Nevertheless, the dimensions of the scour protection are estimated in a similar way of the jacket foundations.

A scour protection composed of a filter layer extending 1-1.5 x D (suction bucket) or 6 x D (piled) and an armor layer extending 1.2 x D or 4 x D or around the buckets and piles, respectively is considered. More details on the dimensions considered for the scour protection estimate is presented in **Table 2-8** and **Table 2-9** for Phase 1 and Phase 2 WTG BF foundations respectively. A rock density of 2.6 ton/m^3 and porosity (n) of 0.4 for armor and 0.6 for filter are assumed.

The site conditions and the structure itself indicate a low scour potential around BF. Moreover, the lack of knowledge about scour and scour protection around these upcoming foundations points to the possibility of applying the strategy of scour monitoring and installation of scour protection as a remediation measure, if necessary.

Table 2-8: Estimates of Phase 1 scour protection dimensions for the WTG BF foundations considered for New England Wind.

Characteristics	Piled BF	Suction Bucket BF
Side length [m]	78	78
Pile/Bucket Diameter [m]	4	15
Extent - Filter Layer [m]	12	20
Area of Filter Layer [m^2]	5,895	8,571
Height of Filter Layer [m]	1	1
Volume – Filter layer [m^3]	5,895	8,571
Rock Weight – Filter Layer [ton]	6,131	8,914
Radial Extent Armor Layer [m]	8	17.5
Area (incl. structure) Pile/Bucket - Armor Layer [m^2]	201	1,963
Area (excl. structure) Pile/Bucket - Armor Layer [m^2]	188	1,787
Height of Armor Layer	1	1
Volume per Pile/Bucket - Armor Layer [m^3]	188	1,787
Rock Weight per Pile/Bucket - Armor Layer [ton]	294	2,787
Volume per BT foundation – Armor Layer [m^3]	565	5,360
Rock Weight per BT foundation – Armor Layer [m^3]	882	8,362
Total Seabed Area [m^2]	**5,895**	**8,571**

Table 2-9: Estimates of Phase 2 scour protection dimensions for the WTG BF foundations considered for New England Wind.

Characteristics	Suction Bucket BF	Piled BF
Side length [m]	87	87
Pile/Bucket Diameter [m]	15	4
Extent - Filter Layer [m]	20	12
Area of Filter Layer [m^2]	9,754	6,862
Height of Filter Layer [m]	1	1
Volume – Filter layer [m^3]	9,754	6,862
Rock Weight – Filter Layer [ton]	10,144	7,136
Radial Extent Armor Layer [m]	17.5	12
Area (incl. structure) Pile/Bucket - Armor Layer [m^2]	1,963	616
Area (excl. structure) Pile/Bucket - Armor Layer [m^2]	1,787	603
Height of Armor Layer	1	1
Volume per Pile/Bucket - Armor Layer [m^3]	1,787	603
Rock Weight per Pile/Bucket - Armor Layer [ton]	2,787	941
Volume per BT foundation – Armor Layer [m^3]	5,360	1,810
Rock Weight per BT foundation – Armor Layer [m^3]	8,362	2,823
Total Seabed Area [m^2]	**9,754**	**6,862**

2.4.4 Edge scour

Please refer to 2.2.4.

2.4.5 Far field scour

Please refer to 2.2.5.

2.5 IMPACT ASSESSMENT – INSTALLATION VESSELS

Any scour processes arising from installation vessels operating alongside monopile, jacket and Electrical Service Platform (ESP) foundations are temporary and very minor and thus to be considered negligible. This is confirmed by the findings from existing offshore wind farms reported by (Cefas, 2006) and Scroby sands and (Fugro Marine GeoServices, 2017).

3. THE POTENTIAL FOR SCOUR PROCESSES ALONG THE CABLE ROUTE

3.1 OVERVIEW OF THE ROUTE

Four or five offshore export cables—two cables for Phase 1 and two or three cables for Phase 2—will transmit electricity from the SWDA to shore. The offshore export cables for both Phases will be installed within a shared Offshore Export Cable Corridor (OECC). The New England Wind OECC is largely the same OECC proposed in the Vineyard Wind 1 Construction and Operations Plan. The New England Wind OECC will travel from the northwestern corner of the SWDA along the northwestern edge of Lease Area OCS-A 0501 (through Vineyard Wind 1) and then head northward along the eastern side of Muskeget Channel toward landfall sites in the Town of Barnstable. At approximately 2 -3 km (1 -2 mi) from shore, the OECC for each Phase will diverge to reach separate landfall sites in Barnstable.

While the Proponent plans to install all New England Wind offshore export cables within the OECC, New England Wind has identified two variations of the Phase 2 OECC: the Western Muskeget Variant and the South Coast Variant. These variations are necessary to provide New England Wind with commercial flexibility should technical, logistical, grid interconnection, or other unforeseen issues arise during the review and engineering processes that preclude one or more Phase 2 offshore export cables from being installed within all or a portion of the OECC.

3.1.1 Limitations

It is noted that at the time of this evaluation (May 2020), the two OECC variations of the Phase 2 OECC: the Western Muskeget Variant and the South Coast Variant, were not considered as part of the base case OECC route.

3.2 SITE SURVEY RESULTS

In 2019 geophysical, geotechnical, and remote sensing surveys were conducted along the potential offshore export cable corridors (with the exception of the Western Muskeget Variant and the South Coast Variant). These survey results show that the cable corridors traverse over a significant variation in geology, with strong influence by tidal currents controlling local seabed morphology and grain size. The scour processes are more prevalent in shallower water along portions of the export cable routes due to increased tidal current flow. Constrictions in the seabed and shoreline geomorphology help funnel water masses through narrow passages which the tidal currents now maintain (e.g. Muskeget Channel).

Sand waves and sand ripples occur regularly throughout the export cable corridor (Figure 3-1). In general, sand waves and ripples indicate active reworking of surficial sediments. Sediment transport occurs daily along the flanks of these features leading to the migration of these bedforms. These features are typical of coastal marine environments where sand is a dominant constituent of the seafloor with active tidal currents on the water column.

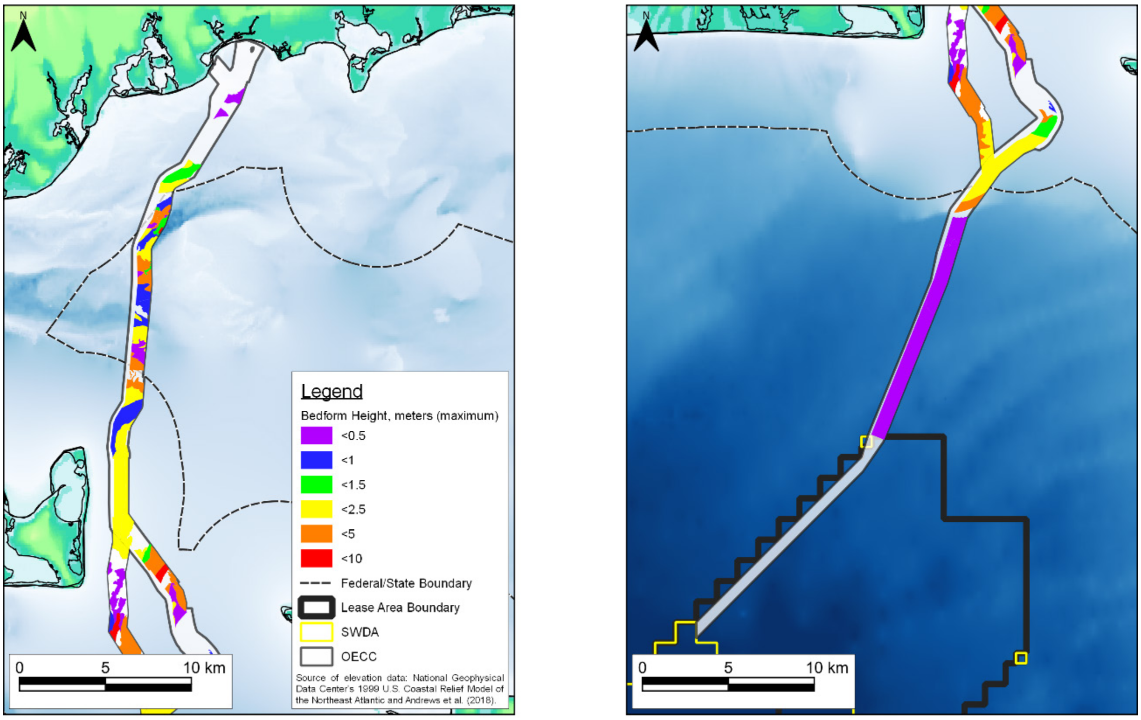


Figure 3-1: Sand waves locations and heights along the Offshore Export Cable Corridor

3.3 SCOUR POTENTIAL EVALUATION

The scour potential around an in-place cable is dependent on several parameters such as seabed mobility, occurrence of coarse material, occurrence of shallow water (for New England Wind considered to be water depth < 10 m) and complex seafloor (e.g. identification of sub surface objects, gravel and boulders).

3.3.1 Areas of mobile sediments

In areas where mobile sediments are present, the surficial sand layer can range from being fairly stable (as indicated by the presence of sand ripples at seabed) to highly mobile (as indicated by the presence of large sand waves at seabed). The sand waves migration could expose a submarine cable when the sand wave trough crosses the buried cable causing a lowering of the seabed and possibly the exposure of the buried cable, if the cable is not buried deep enough. The cable burial depth (depth of lowering) is planned to be below the mobile seabed into the underlying stable seabed. As the cable burial depth is determined in such a way to avoid the cable being expose by migration of bedforms, no scour effects around the cable are expected in areas of mobile sediments.

3.3.2 Potential cable protection

Within certain locations of the OECC, the required cable burial depth might be not achieved, and cable protection could be required. The most common and appropriate methods of cable protection for New England Wind are concrete mattresses, rock placement, and gabion rock bags, as illustrated in Figure 3-2.

Figure 3-2. Examples of potential cable protection systems: (left) placement of prefabricated concrete mattresses; (b) deposition of rocks along the cable route.

Typical mattress sizes are 2.4 x 4.8 m or 3m x 6m, mattress thicknesses are often 150mm, but can be up to 300mm. Typical rock dumping is designed with stone sizes in the order 10-30cm to cover a width of approximately 3.5-5.0 m and an average height of 0.6 m. However, the cable protection could reach 7-9 m in width locally and so a maximum width of 9 m is assumed.

4. CONCLUSIONS

A holistic approach has been used to evaluate the scour potential for New England Wind and cable route site taking into account the current development of scour research and the methodologies presented previously, and the foundation types and diameters expected. On the site, significant scour is not expected due to the low currents. The type of foundation has influence on the scour depth, therefore scour protection might be necessary to reduce any effect of scour at the site depending on the type of foundation. The envelope of the scour protection dimensions is presented in COP Volume I, which includes some conservatism to account for uncertainties such as ultimate design engineering, contractor and installation inaccuracies and migration and replenishment of material during and/or after installation.

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APPENDIX A – CALCULATIONS OF MOBILITY CRITERIA FOR SCOUR DEVELOPMENT

Sediment is considered mobile when the bed shear stress (τ) exceeds a threshold shear stress (τ_{cr}). A common method to determine the threshold of motion is the Shields curve (Shields, 1936) which defines the Shields parameter, θ, defining the ratio of driving and stabilizing forces as outlined in:

$$\theta = \frac{\tau}{g(\rho_s - \rho_w)d_{50}}$$

Where:
θ: Shields parameter [-]
τ: combined wave and current shear stress [N/m^2]
g: gravitational acceleration [m/s^2]
ρ_s: sediment density [kg/m^3]
ρ_w: water density [kg/m^3]
d_{50}: sediment median grain diameter

The combined wave and currents action can produce a maximum shear stress greater than the individual components. The combined wave and current shear stress oscillate around a mean value, τ_m and has a peak value of τ_{max} (Soulsby, 1997):

$$\tau_m = \tau_c \left[1 + 1.2\left(\frac{\tau_w}{\tau_c + \tau_w}\right)^{3.2}\right]$$

$$\tau_{max} = \sqrt{(\tau_m + \tau_w \cos\varphi)^2 + (\tau_w \sin\varphi)^2}$$

Where:
τ_w: wave shear stress [N/m^2]
τ_c: current shear stress [N/m^2]
φ: angles between current and wave direction [°]

When currents and waves are propagating in the same direction, τ_{max} results in:

$$\tau_{max} = \sqrt{{\tau_m}^2 + {\tau_w}^2}$$

The wave-induced bed shear stress (τ_w) is defined as:

$$\tau_w = 0.5\rho_w f_w U_w^2$$

Where:
f_w: dimensionless wave friction coefficient [-]
U_w: near bed wave orbital velocity amplitude [m/s]

The wave friction coefficient (f_w) is a function of the amplitude of the near bed wave particle amplitude (A) and bed roughness and it can be defined by several expressions.
A common one is after Soulsby (1997):

$$f_w = 1.39\left(\frac{A}{d_{50}/12}\right)^{-0.52}$$

$$A = \frac{U_w T_p}{2\pi}$$

The near bed wave orbital velocity amplitude (U_w) in an irregular sea state can be approximated to be:

$$U_w = \frac{H_{rms}\pi}{T_{m-1}} \frac{1}{\sinh(k_{m-1}h)}$$

Where:
H_{rms}: root-mean-square wave height $H_{rms} = H_{m0}/(2\sqrt{2})$ [m]
T_{m-1}: -1^{th} spectral wave period, approximately $T_{m-1} = T_p/1.1$ [s]
k_{m-1}: wave number based on the wavelength of T_{m-1}, $k_{m-1} = 2\pi/L_{m-1}$ [m^{-1}]

The current-induced bed shear stress (τ_c) is defined as:

$$\tau_c = 0.5\rho_w f_c U_c^2$$

Where:
f_c: dimensionless friction coefficient [-]
U_c: flow velocity [m/s]

The dimensionless friction coefficient is given by:

$$f_c = \frac{2g}{C^2}$$

Where:
g: gravitational acceleration [m/s^2]
C: Chézy coefficient, assumed C=65 for sandy seabed

According to (Soulsby & Whitehouse, 1997), sediment will be mobilized when the critical Shields parameter is exceeded:

$$\theta_{cr} = \frac{0.24}{D_*} + 0.055[1 - e^{-0.02D*}]$$

Where D_* is the dimensionless grain size:

$$D_* = \left[\frac{g(s-1)}{\nu^2}\right] d_{50}$$

and:

ν: kinematic viscosity of water [m^2/s];
s: ρ_s/ρ_w [-]

For $D_* < 10$ (fine sand) an alternative expression for the critical Shields parameter is proposed (Soulsby & Whitehouse, 1997):

$$\theta_{cr} = \frac{0.30}{1 + 1.2D_*} + 0.055[1 - e^{-0.02D*}]$$

Assuming a hydraulic load amplification factor of 2 to account for the additional turbulent and vortices around the structure, the sediment will become mobile, and thus, scour will potentially develop, when the relative mobility ($^{\theta}/_{\theta_{cr}}$) is higher than 0.5 (Deltares, 2017).

APPENDIX B – SCOUR CALCULATIONS

The equilibrium scour depth in coarse-grained non-cohesive soils is calculated considering wave-dominated and current-dominated conditions. When considering wave-dominated conditions, the Raaijmakers's formula (Raaijmakers & Rudolph (2008) is used:

$$Seq = 1.5\tanh\left(\frac{h}{D}\right) K_w\, K_h$$

$$K_h = \left(\frac{h_p}{h}\right)^{0.67}$$

$$K_w = 1 - \exp\,(-A)$$

$$A = 0.012KC + 0.57KC^{1.77}U_{rel}^{3.76}$$

Where:
Seq: equilibrium scour depth [m]
D: MP diameter [m]
h: water depth [m]
K_h: correction factor for pile height, for MPs $K_h = 1$
h_p: pile height, for MPs $h_p = h$
K_w: correction factor to account for wave action:
KC: Keulegan-Carpenter number in function of wave height, $KC = UwTp/D$
U_{rel}: relative velocity, $U_{rel} = U_c/(U_c + U_w)$ where U_c: flow velocity and U_w: near bed wave orbital velocity amplitude

When considering a current-dominated condition, the Sheppard's formula is used (Sheppard & Miller Jr, 2006):

In the clear-water scour range ($0.47 \leq \frac{U}{U_{cr}} \leq 1$):

$$S_{eq} = 2.5tanh\left(\frac{h}{D}\right)^{0.4}\left(1 - 1.75\left(ln\left({}^{U}/_{U_{cr}}\right)\right)^2\right)\left(\frac{D/d_{50}}{0.4\,(D/d_{50})^2 + 10.6(D/d_{50})^{-0.13}}\right)D$$

In the live-bed scour range up to the live-peak $\left(1 < \frac{U}{U_{cr}} \leq \frac{U_{lp}}{U_{cr}}\right)$:

$$S_{eq} = 2.5tanh\left(\frac{h}{D}\right)^{0.4}\left(2.2\left(\frac{{}^{U}/_{U_c} - 1}{{}^{U_{lp}}/_{U_c} - 1}\right) + 2.5\left(\frac{D/d_{50}}{0.4\,(D/d_{50})^2 + 10.6(D/d_{50})^{-0.13}}\right)\left(\frac{{}^{U_{lp}}/_{U_c} - {}^{U}/_{U_c}}{{}^{U_{lp}}/_{U_c} - 1}\right)\right)D$$

And in the lived-bed scour range above the live-bed peak $\left(\frac{U}{U_{cr}} > \frac{U_{lp}}{U_{cr}}\right)$:

$$S_{eq} = 2.2tanh\left(\frac{h}{D}\right)^{0.4} D$$

Where:
S_{eq}: equilibrium scour depth [m]
D: monopile diameter [m]
d_{50}: median sediment grain diameter [m]
U: current velocity [m/s]
U_{cr}: velocity at threshold conditions for sediment motion (sediment critical velocity) [m/s]
U_{lp}: lived-bed peak scour velocity (velocity where the bed planes out) [m/s]

The sediment critical velocity (U_{cr}) is calculated according to van Rijn (1993) taken from Florida Department of Transportation (2005):

$$U_{cr} = 2.5u_* ln\left(\frac{h}{2.72z_0}\right)$$

$$u_* = \sqrt{\tau_{cr}/\rho_w}$$

$$z_0 = \begin{cases} \frac{\nu}{9u_*}, 0 < Re_* \le 5 \\ k_s 10^{-3}\left(-6 + 2.8Re_* - 0.58Re_* \ln(Re_*) + 0.002{Re_*}^2 + 111/Re_*\right), 5 < Re_* \le 70 \\ \frac{k_s}{30}, Re_* > 70 \end{cases}$$

$$Re_* = u_* k_s / \nu$$

$$k_s = \begin{cases} 2.5d_{50}, & for\ d_{50} \ge 0.6\ mm \\ 5d_{50}, & for\ d_{50} < 0.6\ mm \end{cases}$$

Where:
u_*: critical friction velocity [m/s]

τ_{cr}: critical bed shear stress, when $\theta = \theta_{cr}$
ρ_w: water density [kg/m^3]
z_0: roughness length [m]
Re_*: Reynold number associated with u_* [-]
ν: kinematic viscosity of water [m^2/s]
k_s: Roughness coefficient [m]

The live-bed scour peak velocity (U_{lp}) is calculated following van Rijn (1993):

$$U_{lp} = \begin{cases} 0.8\sqrt{gh}, & if\ 0.8\sqrt{gh} \geq 29.31u_*\log\left(4h/d_{90}\right) \\ 29.31u_*\log\left(\frac{4h}{d_{90}}\right), & if\ 0.8\sqrt{gh} < 29.31u_*\log\left(4h/d_{90}\right) \end{cases}$$

Where:
g: gravitational acceleration [m/s^2]
h: water depth [m]
d_{90}: sediment grain size exceeded by 10%

Both environmental conditions are calculated, and the most conservative estimate is taken as the estimate of scour depth in coarse-grained non-cohesive soils.