ACP OCRP-4-202x U.S. Recommended Practices for Geotechnical and Geophysical Investigations and Design

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Standards Committee

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Foreword

The Foreword is included with this document for information purposes only and is not part of the **Offshore Compliance Recommended Practices.**

The regulatory framework for the U.S. offshore wind industry has been under development for well over a decade but the first commercial projects are just making their way through the process now[1]. In 2005, the United States Department of the Interior (DOI) was given authority, under the Energy Policy Act of 2005 (EPAct 2005), to grant leases on the Outer Continental Shelf (OCS) for offshore renewables and to promulgate any necessary regulations needed to ensure safe and orderly deployments. Under this authority DOI delegated this responsibility to the Minerals Management Service (MMS). The MMS was later reorganized (2010 – 2012) to create the current regulatory agencies, the Bureau of Ocean Energy Management (BOEM) and the Bureau of Safety and Environmental Enforcement (BSEE). In 2009, MMS published 30 CFR 585, which are the first federal regulations governing the development of offshore wind facilities. It outlines a process spanning a typical offshore wind project (cradle to grave), from competitive leasing of the OCS and gaining site control, to permitting, commercial operations planning, facility design, commissioning, operations, and inspection, all the way through decommissioning. In the initial version of the 30 CFR regulation, no specific standards are incorporated by reference. The regulation requires "best practices" be used, with the intent that best practices would eventually evolve from industry experience as it matured.

To that end, from 2009 to 2012, the U.S. offshore wind industry, in collaboration with BOEM, the National Renewable Energy Laboratory (NREL) and the U.S. Department of Energy (DOE), and the American Wind Energy Association (AWEA), developed a roadmap from existing standards to facilitate the definition of "best practices", which was titled *AWEA Offshore Compliance Recommended Practice (OCRP) 2012*. Over 50 members of the offshore wind industry participated in the development of *AWEA OCRP 2012* which covers all aspects of fixed-bottom offshore wind facility development, starting with the design phase through to decommissioning. It refers to over 100 standards, guidelines, and technical specifications. After its publication in October 2012, it became the *de facto* reference for offshore wind development in the United States and has been used as an informative framework for regulators, developers, and certified verification agents.

However, for several reasons, *AWEA OCRP 2012* no longer adequately addresses the regulatory requirements for BOEM/BSEE and the offshore wind development community. First, when it was written, the formal process for review and approval by the American National Standards Institute (ANSI) had not yet been adopted by AWEA. This formal approval process is critical for the acceptance of standards by the regulators because U.S. ANSI-approved consensus standards and guidelines have vital procedural safeguards that allow them to be adopted by developers to guide project design and approval, referenced by BOEM in future revisions of 30 CFR 585 or, if appropriate, they can be explicitly quoted by BOEM/BSEE in 30 CFR 585 or other regulations. In addition to this important step, missing but needed to attain credibility in the regulatory process, the scope of *AWEA OCRP 2012* was too narrow and did not cover key aspects of the current U.S. industry. Floating foundation systems are explicitly not covered even though the industry is rapidly moving toward the commercialization of floating wind. Also, the complexity of collecting, processing, validating, and applying metocean data was not addressed. Similarly, requirements for geotechnical and geophysical data collection were not addressed at all, despite the wide range of site conditions across the potential U.S. lease areas and number of substructure variants. In addition, the treatment of subsea high voltage cables was very light in *AWEA OCRP 2012* and did not adequately recognize the unique challenges associated with the use of subsea cables that the industry is currently facing in Europe. Finally, in addition to the noted missing elements in *AWEA OCRP 2012*, the document is over ten years old and does not adequately reflect the experience gained through the installation of over 40 gigawatts of offshore wind globally, and the extensive U.S. project development experience that has occurred since it was written.

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[1] Cape Wind, the first offshore wind project in the United States, received notice of COP approval and notice of no objections to FDR/FIR in September 2014. The project requested a lease suspension in 2015 due to difficulty obtaining project financing. Cape Wind relinquished its lease in 2017.

In December 2016, BOEM requested that AWEA establish a new initiative to update the existing *AWEA OCRP 2012* document to address the above concerns. In September 2017, the AWEA Wind Standards Committee voted to approve the formation of an offshore wind subcommittee to oversee the development of this initiative. This subcommittee was formed under the leadership of Walt Musial, Principal Engineer at NREL, and held its inaugural meeting on October 23, 2017. At that meeting, five working groups were formed to address the *AWEA OCRP 2012* deficiencies. These working groups include:

- OCRP 1 - Working Group 1 - ACP Offshore Compliance Recommended Practices (OCRP) Edition 2 under the leadership of Rain Byars and Graham Cranston.
- OCRP 2 - Working Group 2 - ACP U.S. Floating Wind Systems Recommended Practices under the leadership of Lars Samuelsson and Leif Delp.
- OCRP 3 - Working Group 3 - ACP U.S. Offshore Wind Metocean Conditions Characterization Recommended Practices under the leadership of Mike Drunsic and Lorry Wagner.
- OCRP 4 - Working Group 4 - ACP U.S. Recommended Practices for Geotechnical and Geophysical Investigations and Design under the leadership of Matt Palmer and Mathieu Guinard.
- OCRP 5 - Working Group 5 - ACP Recommended Practices for Submarine Cables under the leadership of Georg Engelmann and Bob Hobson.

These dedicated and qualified industry conveners each assembled a diverse group of subject matter experts in their respective working groups. All told, over 300 members of the U.S. offshore wind industry participated in this initiative.

Initially, the working groups developed a coordinated set of work scopes that were approved through the ANSI process, and each worked independently to develop a recommended practice (RP) document following the ACP/ANSI rules. Each RP provides a roadmap for U.S. offshore wind development in its respective area with a view toward adding transparency and consistency to the regulatory approval process which can provide benefits to developers, regulators, and the general public.

All the working groups collectively assembled face-to-face at semi-annual meetings throughout a three-year period from 2018 through 2020 where issues with harmonization, consistency, potential conflicts, and gaps were identified and resolved. Together, these working groups have developed a comprehensive set of consensus-based RPs to guide the safe and orderly deployment of offshore wind energy in the United States. These nationally focused RP documents account for the unique offshore conditions on the U.S. OCS, but they also apply to potential installations in state waterways (e.g., Great Lakes). They provide reasonable requirements for commercial offshore development covering a range of project development activities including project design, construction and deployment practices, operation, safety, inspection, and decommissioning, while anticipating the new and evolving nature of the offshore wind technology. This suite of offshore RPs will help clarify the requirements for developers beyond what was provided by *AWEA OCRP 2012* and enable BOEM/BSEE to adopt better requirements that reflect industry best practices.

Although these five RP documents were written independently by their respective working groups, a significant effort was made to coordinate the technical interfaces. As such, they are intended to be used as a set. The governing RP was written by Working Group 1 - AWEA Offshore Compliance Recommended Practices (OCRP) Edition 2. This document supersedes the original *AWEA OCRP 2012* document and, in several areas, defers directly to the companion RP documents from Working Groups 2 through 5. Similarly, the companion RP documents refer to the governing ACP OCRP-1-2022 document.

It is the expectation of all who participated in this important standards development process that this comprehensive set of RP documents will clarify the complexities of offshore wind development in the U.S. while providing clarity for all stakeholders and, in doing so, will help lower offshore wind energy costs and increase worker safety for the public good.

1 Scope

This document provides the scope of activities for the ***ACP U.S. Geotechnical and Geophysical Investigations and Design Working Group***. This working group was formed by the Offshore Wind Subcommittee, which is a subcommittee to the American Clean Power Association (ACP) Wind Technical Standards Committee (WTSC). ACP is granted authority to establish wind energy standards by the American National Standards Institute (ANSI). The purpose of this working group is to make recommendations to Offshore Wind Subcommittee on the use of standards and guidelines related to geotechnical and geophysical concerns for offshore wind in the United States.

The working group shall write Recommended Practices (RP) in compliance with the ANSI/ACP Standards Development Procedures. The RP developed under this scope shall apply to the following:

- Offshore wind facilities that may potentially be installed in U.S. state and federal waters in the continental United States, Hawaii, and Alaska, including inland bodies of water such as the Great Lakes
- Fresh and salt water at any water depth
- All wind turbine generating (WTG) substructures and foundations in contact with the seafloor
- All offshore substations, meteorological towers, and other offshore wind components in contact with the seafloor
- Fixed bottom and floating structure associated with offshore wind components
- All phases of project life: planning, designing, constructing, operating, decommissioning, and re-powering

The working group shall consider the following:

- Existing codes, standards, recommended practices, and guidelines relevant to this scope
- Geotechnical and geophysical investigation methods, including vessels
- Correlation and interpretation of geophysical and geotechnical results
- In-situ and laboratory testing
- Requirements for specific WTG substructure and foundation designs including but not limited to monopile, jacket, gravity base, suction bucket, and seafloor connections for floating offshore wind components.
- Requirements for other specific structures related to offshore wind installations including offshore substation substructures and meteorological towers.
- Soil types that could be encountered offshore in state or federal jurisdictions of the US
- Subsurface hazards and areas of concern (e.g., boulders, unexploded ordnance, archeology, hazardous waste)
- Reporting requirements and data management methods
- Geotechnical or geophysical considerations for cable burial
- Design issues related to:
  - Applicability of p-y curves for large diameter WTG monopiles
  - Cyclic degradation
  - Soil damping
  - Scour
  - Seismic
  - Integrated modeling

The development of the RP by the ACP U.S. Geotechnical and Geophysical Investigations and Design Working Group shall include, but is not limited to the following considerations:

- Definition of minimum requirements specific to this scope by reference to existing relevant industry codes, standards, and guidelines.

- Working with other Offshore Wind Subcommittee working groups to manage interfaces among RPs and explore how synergies can be leveraged to create greater efficiencies throughout a project and the industry.
- Developing methods to fill gaps in existing standards specific to this scope.

2 Terms and Definitions

For the purposes of this document, the following terms and definitions apply. Definitions are aligned with IEC 61400-1 and IEC 61400-3-1 wherever possible.

2.1
accreditation
procedure by which an authoritative body gives formal recognition that a body is impartial and technically competent to conduct specific tasks such as certification, tests, specific types of tests, etc.

2.2
array cables
submarine cable that connects the turbines to each other and to the offshore substation

2.3
as low as reasonably practicable (ALARP)
the principle that the quantum of residual risk for an activity has been weighed against the sacrifice of money, time or trouble involved in the measures necessary for averting the risk

2.4
certification body
body that conducts certification of conformity

2.5
certified verification agent (CVA)
an individual or organization, experienced in the design, fabrication, and installation of offshore marine facilities or structures, who will conduct specified third-party reviews, inspections, and verifications in accordance with 30 CFR 285[2]

2.6
design life
The period of time defined in the design basis as the intended minimum useful life of a wind farm component, spanning from the time of installation to the time of decommissioning.

2.7
designer
party or parties responsible for the design of an offshore wind turbine or other assets of an offshore wind farm (e.g., offshore substations, cables)

2.8
developer
party or parties responsible for the permitting, planning, construction, and commissioning of offshore wind facilities

2.9
environmental conditions
characteristics of the physical environment (e.g., wind, waves, sea currents, water level, sea ice, marine growth, scour, and overall seabed movement) that may affect the offshore wind farm

2.10
export cables
submarine cable(s) that connect the onshore and offshore substations, or between an AC offshore substation and a DC converter substation. Cable(s) exporting the generated electrical power from the offshore substation (OSS) (or directly from the wind farm if an OSS is not a part of the project) to onshore substation

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[2] With the promulgation of the rule *Reorganization of Title 30* in the Federal Register on January 31, 2023, many of the rules governing safety, environmental oversight and enforcement were moved from Section 585 to Section 285 under Title 30.

2.11
foundation
part of a support structure that transfers the loads acting on the sub-structure into the seabed

2.12
geophysical
Non-intrusive site investigation and data acquisition

2.13
geotechnical
Intrusive site investigation and data acquisition, as well as design and engineering of soil and soil/structure interaction

2.14

guidance note
Information in the RP is given either to increase the understanding of the statements or to provide informative references (i.e., acceptable, but not mandatory methods for fulfilling the recommendations in this standard).

2.15
manufacturer
party or parties responsible for the manufacture and construction of an offshore wind turbine or other assets of an offshore wind farm (e.g., offshore substations, cables)

2.16
marine growth
surface coating on structural components caused by plants, animals, and bacteria

2.17
mean sea level
average level of the sea over a period of time long enough to remove variations due to waves, tides, and storm surges

2.18
mean lower low water
The average of all the low water heights observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch

2.19
metocean
abbreviation of meteorological and oceanographic. Oceanographic is sometimes referred to as “marine” in IEC and other documents.

2.20
monopile
Structure type with foundation and sub-structure consisting of a single vertical pile (see Figure 2-1[3])

2.21
National Tidal Datum Epoch
The specific 19-year period adopted by the National Ocean Service as the official time segment over which tide observations are taken and reduced to obtain mean values (e.g., mean lower low water, etc.) for tidal datums. It is necessary for standardization because of periodic and apparent secular trends in sea level. The present National Tidal Datum Epoch (NTDE) is from 1983 through 2001 and is actively considered for revision every 20-25 years. Tidal datums in certain regions with anomalous sea level changes (Alaska, Gulf of Mexico) are calculated on a Modified 5-Year Epoch.”

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[3] Monopiles are equivalent to the caisson structures referred to in some offshore structural standards.

2.22
offshore wind turbine

Rotor-nacelle assembly and support structure located offshore

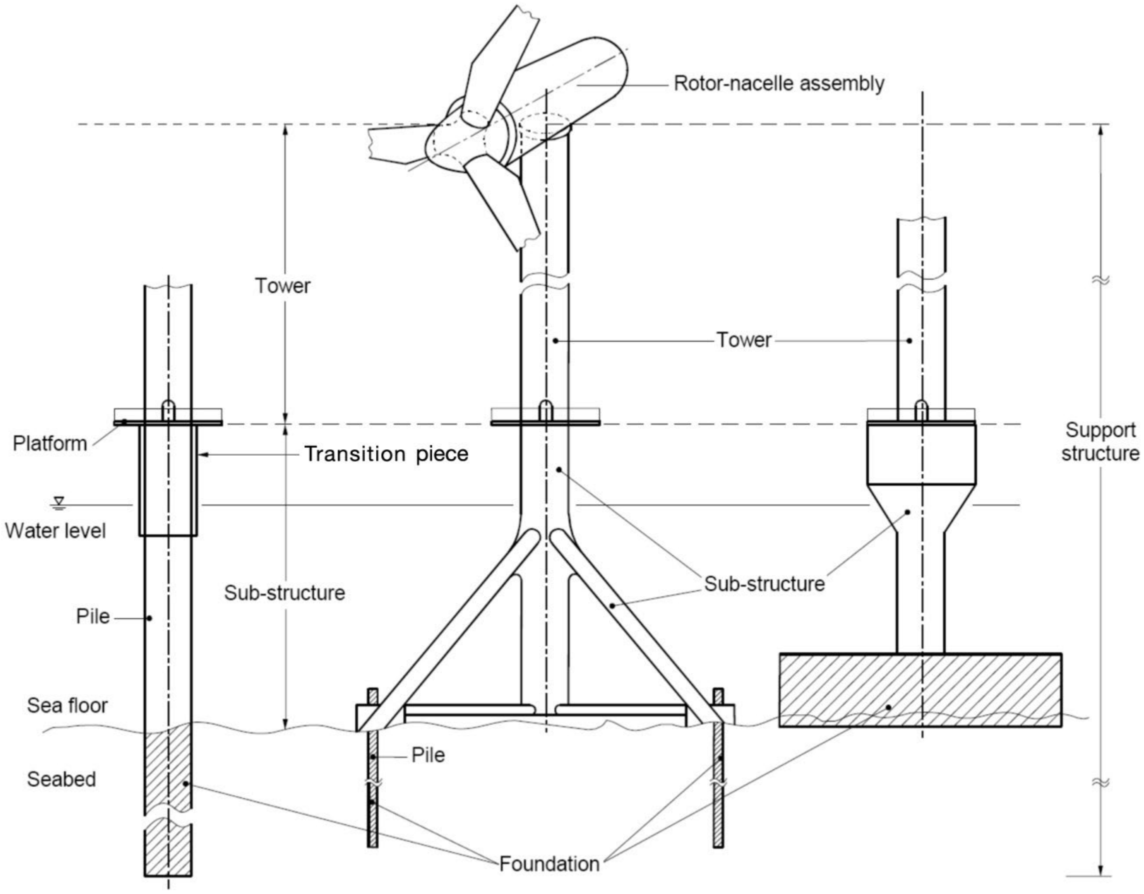
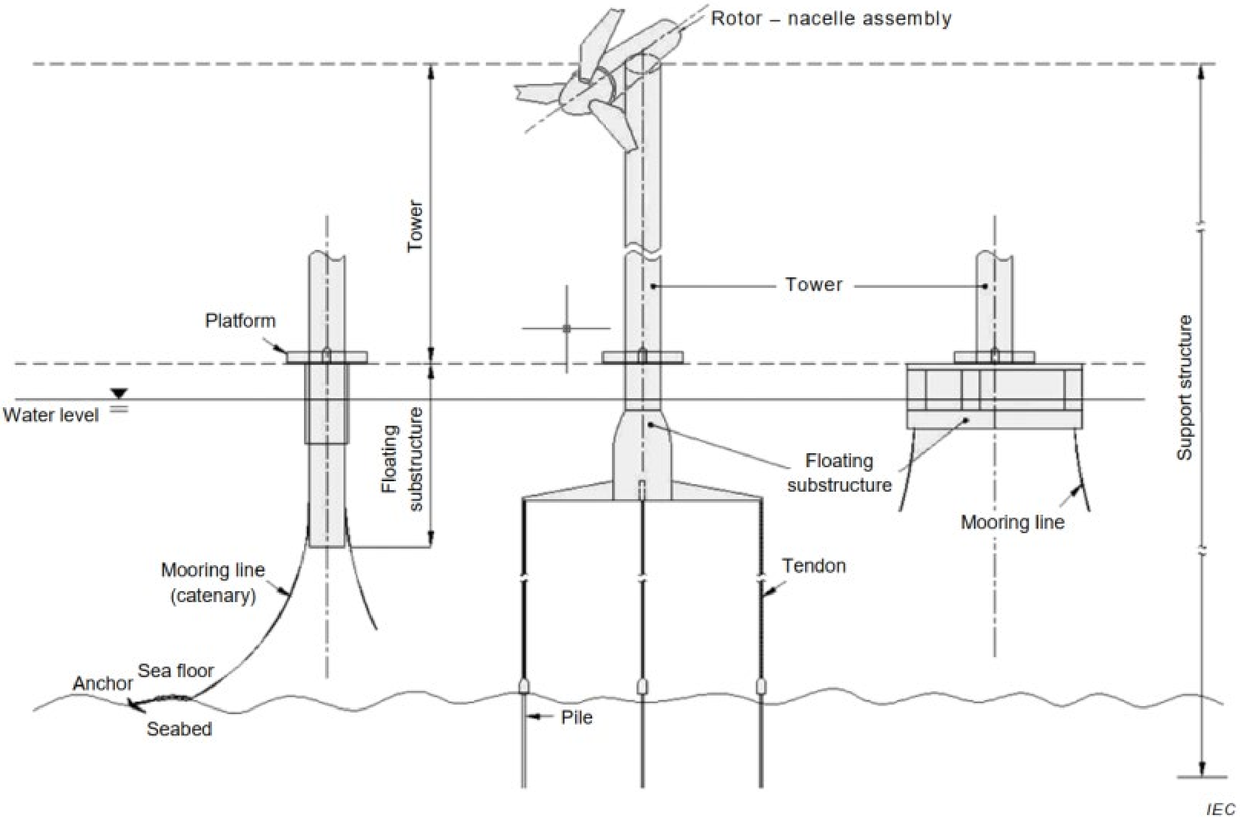


Figure 2-1: Parts of a fixed offshore wind turbine (Source: Adapted from IEC 61400-3-1)



From left to right: Spar, TLP, and Semi.

Figure 2-2 Parts of a floating offshore wind turbine (Source: Adapted from IEC 61400-3-2)

2.23
pile penetration
vertical distance from the seafloor to the pile tip

2.24
rotor-nacelle assembly
part of an offshore wind turbine carried by the support structure (see Figure 2-1)

2.25
seafloor
interface between the sea and the seabed

2.26
sea state
condition of the sea in which its statistics remain stationary

2.27
seabed
materials below the seafloor in which a support structure is founded

2.28
seabed movement
movement of the seabed due to natural geological and hydrodynamic processes

2.29
scour
removal of seabed soils by currents and waves or caused by structural elements interrupting the natural flow regime above the seafloor

2.30
still water level
abstract water level calculated by including the effects of tides and storm surge but excluding variations due to waves; still water level can be above, at, or below mean sea level

2.31
storm surge
change in water level caused by atmospheric change and/or wind associated with a storm

2.32
sub-structure
part of an offshore wind turbine or OSS support structure that extends upwards from the seabed or seafloor and connects the foundation to the tower (see Figure 2-1) or topside for fixed-bottom structures, and that part of an offshore wind turbine or OSS support structure that extends upwards from the top of the mooring lines and connects to the tower (see Figure 2-2) or topside for floating structures.

2.33
tides
regular and predictable movements of the sea generated by astronomical forces

2.34
tower
part of an offshore wind turbine support structure which connects the sub-structure to the rotor-nacelle assembly (see Figure 2-1)

2.35
transition piece
part of a monopile offshore wind turbine sub-structure that provides a level connection between the tower and the pile (see Figure 2-1)

2.36
useful life
The period of time that equipment is economically viable and can be operated safely considering its current condition.

2.37
water current
flow of water past a fixed location, usually described in terms of a current speed and direction

2.38
water depth
vertical distance between the seafloor and the still water level

3 General

This chapter includes general information, background, and other guidance useful to interpret and implement the recommended practices contained in this document. Material presented herein is extracted as applicable from the similar section in ACP OCRP-1-2022. Some additional material specific to this document is also provided.

3.1 Use of Language Giving Direction

In this document and in accordance with the latest edition of the ISO/IEC Directives, Part 2, the following verbal forms are used:

- ‘shall’ and ‘shall not’ are used to indicate requirements strictly to be followed in order to comply with this document and from which no deviation is permitted;
- ‘should’ and ‘should not’ are used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required, or that (in the negative form) a certain possibility or course of action is deprecated but not prohibited;
- ‘may’ and ‘need not’ are used to indicate a course of action permissible within the limits of this document;
- ‘can’ and ‘cannot’ are used for statements of possibility and capability, whether material, physical, or causal.

Where documents are referenced without use of these terms, it is implied that the reader may use these documents for guidance.

In this document, the term “codes and standards” is used to refer collectively to regulations, codes, standards, specifications, requirements, recommended practices, and guidelines. However, individual documents are referred to by the title given by their authors.

3.2 Applicable Versions of Reference Standards

Where a reference is given to a document without a specific year or edition, the most recent published edition of that document should be considered the intended reference. In these cases, references to chapters or sections of the referenced document may change with subsequent editions. Where a reference is given to a document with a specific year or edition, only that version of the document should be used. Website URLs are provided for reader's convenience and are subject to change. Section 9 References include the year of each reference verified at the time of publication, which are subject to change.

3.3 Environmental, Health and Safety

The recommended practices for environment, health, and safety for U.S. wind farms are still being developed. The objective is to provide recommended practices for design features and activity planning which will support implementation of occupational health and safety processes and environmental protection during site investigations, construction, and operation.

3.4 US Jurisdictional Definitions

This document covers offshore wind farm assets in U.S waters, which includes federal and state waters. Users should be aware that assets installed in state waters may be subject to different requirements than federal facilities depending on the individual state. Nothing in this document relieves any party from complying with requirements of U.S. state and federal regulations, including occupational health and safety regulations.

3.5 BOEM Lease and COP Requirements

BOEM will issue a lease for each offshore wind development area in federal waters, and for commercial leases will subsequently require the submittal of a Site Assessment Plan (SAP) and Construction and Operations Plan (COP) as described in 30 CFR 585. BOEM has promulgated guidelines for SAPs[4] and COPs[5], which may be modified from time to time. The BOEM requirements for geotechnical and geophysical surveys described in a lease, SAP, SAP guidelines, COP, or COP guidelines may differ from the requirements in this Recommended Practice. In all cases, BOEM requirements shall be fulfilled.

3.6 Professional Engineer

The practice of engineering is regulated through Boards of Professional Engineering which are appointed by individual states. Engineers shall be licensed through a state board in order to perform public or private work which requires engineering training, education, and experience. An individual who has passed the licensing requirements of a state board - including successful testing, evidence of experience, professional recommendations, and ongoing education – may maintain their status as a licensed P.E. and may perform and/or direct engineering work in accordance with the regulations of that state. P.E.s shall follow strict professional and ethical rules as issued by the Board granting the license and shall not perform work outside of their qualifications. Failure to follow these rules can result in consequences for the P.E. including loss of license, fines, or criminal penalties. Each P.E. is assigned a unique license number, and status of license and good standing may be verified through a public database.

Construction drawings and design calculations submitted to regulators for project construction approval are typically required to be stamped by a P.E. For portions of the offshore wind project located onshore or in state waters, the engineer of record is expected to be licensed in the state where the facilities will be used, installed, or operated. For portions of the offshore wind project located in federal waters, it is generally understood that licensing by any state will be accepted by the federal regulator.

3.7 Certified Verification Agent

30 CFR 285 requires the project developer to use a Certified Verification Agent (CVA) to conduct an independent assessment of the design of the facility as well as the fabrication and installation activities. The CVA is nominated by the project developer and is subject to approval by the regulator based on criteria that include technical capabilities, experience, personnel qualifications, global presence, financial stability, and track record, and the specific verification plan for the proposed project.

The CVA reviews design documentation, fabrication drawings and specifications, and installation plans, which are all included in the Facility Design Report (FDR) and the Fabrication and Installation Report (FIR). The CVA certifies in a report that the facility is designed to withstand the environmental and functional load conditions appropriate for the intended service life at the proposed location. This includes evaluation of any aspects of the design that deviate from standard practice and/or requirements. The CVA also monitors the fabrication and installation activities through periodic on-site inspections and certifies in a report that the project is fabricated and installed in accordance with accepted engineering practices, the Construction and Operations Plan (COP), and the FDR/FIR.

The CVA shall use good engineering judgement and practices in conducting their independent assessment. CVA requirements are addressed in 30 CFR § 285.

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[4] https://www.boem.gov/sites/default/files/renewable-energy-program/BOEM-Renewable-SAP-Guidelines.pdf

[5] https://www.boem.gov/sites/default/files/documents/about-boem/COP%20Guidelines.pdf

4 Symbols and Abbreviated Terms

4.1 Symbols and Units

Term	[meaning]
D	Foundation outer diameter
Eir	initial Young’s Modulus
f	Skin friction
G	Soil shear modulus
L	Pile embedment
N	Number of cycles
p	Lateral load
Q	Axial tip resistance
t	Wall thickness
τ_cyc	Cyclic shear stress amplitude
γ_{cyc}	Cyclic shear strain amplitude
y	lateral deflection
z	axial deflection

4.2 Abbreviations and Acronyms

2D	Two-Dimensional
3D	Three-Dimensional
A.D.	Anno Domini
ALARP	As Low As Reasonably Practicable
ALE	Abnormal Level Earthquake
ALS	Accidental Limit State
ACP	American Clean Power Association
ANSI	American National Standards Institute
API	American Petroleum Institute
ASL	Above Mean Sea Level
ASTM	American Society for Testing and Materials
AUV	Autonomous Underwater Vehicle
AWEA	American Wind Energy Association
BOEM	Bureau of Ocean Energy Management
BSEE	Bureau of Safety and Environmental Enforcement
CDP	Common Depth Point
CFR	Code of Federal Regulations
CIRIA	Construction Industry Research and Information Association
CMECS	Coastal and Marine Ecological Classification Standard
CPT	Cone Penetration Testing
CVA	Certified Verification Agent
DOI	U.S. Department of Interior
DTM	Digital Terrain Model

DTS	Desk Top Study
EFH	Essential Fish Habitat
ELE	Extreme Level Earthquake
EM	Electromagnetic
EPA	Environmental Protection Agency
FGDC	Federal Geographic Data Committee
FLS	Fatigue Limit State
FVT	Field Vane Test
GBA	Gravity Base Anchor
GBS	Gravity Base Structure
GNSS	Global Navigation Satellite System
HSE	Health Safety and Environment
ICP	Imperial College Pile
IHO	International Hydrographic Organization
IEC	International Electrotechnical Commission
ISO	International Organization for Standardization
LMB	Luftmine B (German air dropped ground mine Type B)
LRFD	Load and Resistance Factor Design
M	Magnitude
MAG	Magnetometer
MBES	Multibeam Echo Sounder
MEC	Munitions and Explosives of Concern
MEDIN	Marine Environmental Data and Information Network
MLLW	Mean Lower Low Water
MSL	Mean Sea Level: the arithmetic mean of hourly heights overserved over 19 years
M-UHRS	Multichannel Ultra High Resolution Seismic
MWS	Marine Warranty Surveyor
NFA	Natural Frequency Analysis
NMFS	National Marine Fisheries Service
NMO	Normal Move-Out
NREL	National Renewable Energy Laboratory
OCRP	Offshore Compliance Recommended Practices
OCS	Outer Continental Shelf
O&M	Operations & Maintenance
OSIG	Offshore Site Investigation Group
OSS	Offshore Sub-Station
OWF	Offshore Wind Farm
OWT	Offshore Wind Turbine
OWTAP	Offshore Wind Technical Advisory Panel
P.E.	Professional Engineer
PISA	Pile Soil Analysis
pMEC	Potential Munitions and Explosives of Concern
PPDT	Pore Pressure Dissipation Test

PSHA	Probabilistic Seismic Hazard Analyses
PTDR	Pile Top Drill Rig
PV	Plan View
RP	Recommended Practices
QA	Quality Assurance
QC	Quality Control
QRA	Quantitative Risk Analysis
ROV	Remotely Operated Vehicle
SBP	Sub-Bottom Profiler
SCPT	Seismic Cone Penetration Test
SI	Système International
SLS	Serviceability Limit State
SPI	Sediment Profile Imaging
SRC	Seismic Risk Category
SRD	Soil Resistance to Driving
SSS	Side Scan Sonar
SSD	SubSea Drill
SPT	Subsea Pile Template
S-UHRS	Single channel Ultra High Resolution Seismic
SUT	Society for Underwater Technology
TMB	Torpedo Mine Bomb
THU	Total Horizontal Uncertainty
TVU	Total Vertical Uncertainty
UHRS	Ultra-High Resolution Seismic
ULS	Ultimate Limit State
U.S.	United States
USACE	U.S. Army Corps of Engineers
USBL	Ultra-Short Base Line
USEPA	U.S. Environmental Protection Agency
USGS	U.S. Geological Survey
VH	Vertical-Horizontal
VHM	Vertical-Horizontal-Moment
VLA	Vertical Load Anchor
WSD	Working Stress Design
WTG	Wind Turbine Generator

5 Data Acquisition

This document provides requirements for geophysical surveys and geotechnical investigations to support offshore wind developments in marine or lacustrine environments. Various types of surveys, and the wind farm development components they are used to support, are described herein.

The general objectives of marine/lacustrine geophysical surveys and geotechnical investigations are to:

- Provide information about the seafloor/lakebed (seafloor) and sub-seafloor
- Clarify the geological processes affecting the site to provide insights into the existing conditions and future changes
- Identify geohazards and anthropogenic objects at or below the seafloor
- Provide sufficient engineering data to locate and design the wind farm assets and installation processes.

Marine and lacustrine geophysical survey (geophysical survey) information should be integrated with geotechnical information to develop a ground model tailored to achieving these objectives.

Geophysical and Geotechnical surveys should provide sufficient information to meet the requirements of ACP OCRP-1-2022.

This section is organized in five subsections: Section 5.1 addresses the incorporation of innovation and new technologies; Section 5.2 overviews possible phasing/stages in the data acquisition process; Section 5.3 and 5.4 provide recommended practices for Geophysical and Geotechnical Site Investigations, respectively and Section 5.5 addresses metadata and storage requirements.

5.1 Innovation and New Technologies

Technological innovations in multiple cross-disciplinary industries are being adopted into the renewable energy sector at great speed. While this document covers current industry best practice, new technologies covering the same functional purpose(s) should be welcomed, evaluated, and pursued. Alternative technologies and methodologies may be utilized as long as end results are demonstrated to meet the requirements for compliance with the standards recommended in this document. These recommended practices are not intended to preclude or limit the adoption of new technologies and methods.

5.2 Data Acquisition Stages

Geophysical surveys and geotechnical investigations may be conducted in a single stage or sequenced as multiple stages. The staged site investigations shall incrementally provide all necessary data ultimately delivering a comprehensive dataset sufficient to support a compliant detailed design of the site facilities. The phasing/staging, extent of site investigations, and testing methodologies to be scoped are project specific. The site investigations for informing the detailed facility design shall be tailored to target the facility foundation concepts, foundation geometries, design methods, loading conditions and installation methodology considering the complexity of the ground conditions at the site and along the cable corridor. The geophysical surveys and geotechnical investigations should allow for a thorough assessment of potential geohazards.

> **Guidance Note:**
> The Society for Underwater Technology Offshore Site Investigation Group (SUT OSIG) document "Guidance Notes for the Planning and Execution of Geophysical and Geotechnical Ground Investigations for Offshore Renewable Energy Developments" sets out the philosophy and process for investigating the seafloor and managing risk for offshore renewable energy developments. A key part of this philosophy is the staged nature of seafloor surveys, such that project knowledge increases, and risk decreases, in an incremental manner that is appropriate to the stage of development of the project.

5.2.1 **Desktop Studies**

An initial desktop study (DTS) should be performed ahead of conducting offshore surveys. Desktop studies should research and collate all pre-existing pertinent data available for the offshore development area and landfall with a view to establishing:

- Bathymetry and spatial and temporal variation thereof
- Seafloor features, such as bedforms, boulders, sand borrow areas, submerged valleys, and sediment type
- Geological succession, origin and types of soil and rock
- Geotechnical characteristics of the various geological units
- Anthropogenic constraints at seafloor such as known archaeological sites, anchorages, artificial reefs, pipelines and cables
- Environmental concerns such as regions of known sensitive habitats, endangered species concerns, and
- An initial view and appraisal of the likely geohazards and geotechnical risks.

Guidance Note:
Establishing an understanding of the seafloor conditions prior to undertaking offshore survey provides the immediate benefit of being able to scope and design marine survey in a bespoke manner which is a cost-effective means of delivering better data, more predictably and with greater value to the project.

5.2.2 **Reconnaissance/Preliminary Surveys**

Reconnaissance-level or preliminary surveys (Reconnaissance Surveys) may be conducted as part of the initial stage(s) of site investigations. Objectives of reconnaissance surveys include, but are not limited to, the following:

- Provide an early understanding of site conditions, geohazards and potential natural and anthropogenic constraints that may affect wind farm developments
- Provide information used to plan cable routes, wind turbine and OSS locations
- Provide information used to inform further geotechnical exploration activities and more detailed marine investigations conducted in later stages

Reconnaissance surveys are often conducted prior to defining the locations and layouts of wind turbines, offshore substations, and cable routes. Therefore, while the scope and design of the surveys should be bespoke and tailored to the anticipated seafloor conditions, these surveys may not target the final location of offshore wind infrastructure, or a specific design solution for buried infrastructure.

Reconnaissance surveys should be broad in spatial extent and ensure that the full range of geological conditions are captured in order to facilitate an initial wind farm layout to be sited in a risk mitigated manner, avoiding the need for later, further reconnaissance level survey, though later detail survey may be required. Consideration should be given to the depth of penetration of reconnaissance surveys; while foundation systems for offshore wind developments may rarely exceed 60 m below seafloor, deeper penetration may be desirable to understand the formation processes and origin of soils and rocks at the site in order to properly bound the geological understanding of the site. Reconnaissance surveys should be designed to give an indication of variability of structure, materials, and engineering properties across the site to inform the design and specification of subsequent soil investigations.

The planning of reconnaissance geotechnical investigations should be informed by the outcome of the geophysical surveys and/or DTS. Reconnaissance geotechnical investigations should be scoped to investigate key geological strata mapped by the geophysical survey, and to provide sufficient density of data to profile the broad variability of the site conditions. Geological features of particular interest or risk may also be targeted. The depth and type of geotechnical investigation at this stage should be guided by the site seafloor conditions, and on the outcome of any initial foundation concept selection studies.

5.2.3 Detailed Surveys

Detailed surveys shall be conducted to encompass areas that will directly experience seafloor and sub-seafloor disturbance activities because of facility activities and shall be completed prior to the commencement of installation activities. Such activities may include the following:

- Metocean exploration
- Seabed preparation activities
- Installation of data collection structures or systems (e.g., meteorological towers or buoys or other site assessment equipment)
- The installation of offshore wind turbines, foundations and any associated equipment and structures
- Installation of inter-array and export cables
- Planning of operations and maintenance activities
- Any other project-related activities that have the potential to impact the seafloor

Detailed surveys may include the following:

- Engineering and Site Characterization surveys
- Benthic Habitat surveys
- Marine Archaeological surveys
- Dedicated MEC/UXO (Munitions and Explosives of Concern/Unexploded Ordnance) mitigative survey
- Bathymetric survey to define the seafloor elevation in high seafloor mobility areas depending on perceived project risk from increased uncertainty in the bathymetric surface.

These survey types are considered further in the following sections.

5.3 Geophysical Site Investigation

Geophysical surveys for offshore wind facilities can have a range of purposes including a) engineering and site characterization; b) characterization of marine archaeology; c) evaluating benthic conditions with regards to ecology and critical habitats; d) assessing MEC/UXO risks and e) for assessing in-service life. The tools and sensors required for these surveys may be common and survey planning should consider the potential for achieving multiple objectives when surveying with a given set of equipment.

Guidance on survey objectives and planning (Section 5.3.1) and data interpretation and reporting (Section 5.3.4) is provided by survey purpose (e.g. engineering and site characterization), whereas guidance on data acquisition (Section 5.3.2) and processing (Section 5.3.3) is provided by equipment type (e.g. multibeam echo sounder (MBES) or sub-bottom profiler (SBP)).

General adherence to the guidance detailed in ISO 19901-10:2021 (2021)is recommended. Specific deviations from or augmentations to these recommendations are made explicit in the following sections.

5.3.1 Survey Objectives and Planning

5.3.1.1 General

The general objective of a geophysical survey is to provide information about the seafloor and sub-seafloor that is relevant to design, placement, installation, operation, assessment, and decommissioning of a wind farm facility. Surveys should encompass the area of potential impact related to the installation and operation of the facility.

It is important to document a clear set of objectives for geophysical survey operations (e.g., vertical resolution, maximum depth of interest, spatial resolution, sensitivity) and to document performance against these objectives in reporting. The objectives have a significant influence on the type of equipment deployed, its configuration and the QA metrics used to verify performance.

Where relevant, project specifications should refer to methods described in this document. If method-specific information is not contained in this document, in the project-specific documentation or in local regulation documents, then contractor's practice can apply. For some parts of a project specification, it can be necessary to provide preliminary specifications that require future finalization. An example of this could be the future finalization of operational parameter values for equipment, so that actual conditions encountered during data acquisition can be adequately accommodated.

> **Guidance Note:**
> Section 4 of SUT OSIG (2022) provides useful guidance on planning and scoping offshore investigations for Offshore Wind facilities. Guidance for facilities along the Atlantic OCS including descriptions and suggested best practices for a range of equipment types is provided in OCS Study BOEM 2017-049.

5.3.1.2 **Quality Requirements for Marine Geophysical Surveys**

Guidance on these topics is provided in Sections 5.4 and A.5.5 of ISO 19901-10:2021 (2021), and in Section 2.5 of the ACP OCRP-1-2022 document.

Quality assurance for geophysical surveys is generally planned and documented using a Quality Plan; this should be closely cross referenced with the survey scope and long-term project objectives. Quality should focus on the full cycle of survey operations from initial survey preparation, through mobilization, operations, data processing, interpretation, and reporting. Additional quality procedures should be undertaken for the submitted data package to ensure that the delivery is compiled and formatted correctly.

QA of data during acquisition is a particular focus point and often involves both onshore personnel and offshore personnel within client and survey contractor organizations. It is good practice to brief the offshore client representative and contractor team using the survey scope and specification, and long-term project objectives to provide assurance that they have appropriate project-specific knowledge.

Health, safety, and environment (HSE) issues should be given a high priority when planning and executing ground investigations, however that is beyond the scope of this document.

5.3.1.3 **Engineering and Site Characterization Surveys**

Engineering and site characterization surveys are conducted to provide information pertaining to a site's physical characteristics, seafloor conditions, sub-seafloor conditions, geohazards, and obstructions, especially as they apply to the design of planned wind farm assets and their installation processes.

Planning, design, installation, operations, and maintenance, and decommissioning of wind farm infrastructure requires knowledge of the engineering properties of the seafloor and sub-seafloor. The combination of geotechnical and geophysical measurements is used to create a ground model: a knowledge base containing information on the structure and engineering properties of the seafloor and sub-seafloor (refer to Section 7 for further discussion of the Ground Model). Engineering surveys develop the ground model and should be planned and designed to fulfil that objective.

Optimally, engineering surveys should be informed by the facility design envelope to tailor requirements for sensitivity, resolution, and 3D extent of the logged datasets. Since designs often evolve as the project matures, a wide envelope, offering flexibility, is recommended at an early stage. Where a staged survey approach is employed, a survey strategy may capture planned refinement of the ground model in line with narrowing of the design envelope. This strategy should represent a balance between project schedule, regulatory requirements, the complexity of the site, the design method(s) anticipated, seasonality of survey quality at the site, and cost.

To support survey planning for engineering, the following considerations (see Table 5.3-1) should be considered:

Table 5.3-1 Considerations for Designing Engineering and Site Characterization Surveys

Survey Design Parameter	Considerations	Guidance
Coverage	Sufficient survey to characterize a constructible and de-risked area	Consider maximum potential area of impact
		Consult DTS to shape extent of earliest surveys according to ground risks
		Should a staged approach be employed, coverage requirements should evolve through the progression of each stage.
Resolution, data density and positional accuracy (vertical and horizontal)	De-risking according to geological features that could negatively impact development	Design according to scale and type of features (geological, morphological, anthropogenic, etc.) which need to be mapped.
	Optimization of infrastructure cost via informed design	Design resolution to optimize correlation with geotechnical measurements
		Design resolution in alignment with planned construction activities and infrastructure
Line Spacing* and Orientation	Geological heterogeneity and ability to interpolate	Define the geological detail and minimum feature size required at each survey stage
	Coverage, resolution, data density and positional accuracy (vertical and horizontal) requirements for multiple sensors	Line spacing shall be limited by the ability of the available equipment to meet these requirements. Tie lines should be performed for data quality control and to facilitate 3D ground modeling
	Water depth	Ensure equipment performance in full range of water depths; line spacing will likely vary according to water depth, especially where full coverage is required.
	Intersection with geotechnical locations	Ensure sufficient flexibility and de-risking of subsequent geotechnical campaigns
		Where reasonable, plan lines to tie in with previous or planned geotechnical investigations to facilitate data integration

Survey Design Parameter	Considerations	Guidance
Depth of Penetration	Investigate to a sufficient depth below the termination depth of the installation (cable or foundation) to minimize the risk from underlying geohazards	Should be tailored to expected installation and interpreted risk profile
	Investigate to a sufficient depth to clarify relevant geological framework	Refer to DTS and any available previous survey data to determine required depth of penetration to resolve interpretation of relevant geological units
	Suitable depth of investigation may not be consistent throughout all survey lines	Vary according to project specifics and design envelope

* Line-spacing indicates planned separation between survey run lines and/or separation between sensor track lines.

Based on these considerations, typical recommendations for engineering and site characterization geophysical surveys (but not other types of surveys such as MEC/UXO or benthic habitat surveys) in water depths less than ~ 100 m are given in Table 5.3-2. Survey specifications for engineering surveys are ultimately dependent on-site specific requirements and the facility design envelope, deviations from the table below are, therefore, reasonably expected. However, this table is included to provide the context of commonly employed parameters for achieving survey requirements as detailed in this document.

Table 5.3-2 Typical Coverage/Resolution for Engineering and Site Characterization Surveys on the Continental Shelf

Data Type	Engineering and Site Characterization Surveys		
	Parameter	Reconnaissance Stage	Detailed Stage
		Typical range of requirements	Typical range of requirements
MBES Bathymetry	Coverage/Swath	Maximized while still achieving data specifications; dependent on individual project requirements for full or partial coverage: 2.5 to 5 times water depth	Dependent on project priorities, may provide full coverage of foreseen footprint of development by final survey stage
	Resolution/Data Density	Balanced against requirements for coverage, allowing characterization of seafloor sediments as well as partial to full mapping of objects to the required size for narrowing the engineering envelope: Sufficient for 1-to-2-meter objects/Sufficient for 0.5 meter to 2-meter grid; statistical mapping of objects may be sufficient.	According to asset design envelope and installation tool envelope: Sufficient for 0.5 meter to 1-meter objects/Sufficient for 0.5 meter to 1 m grids in WD <50 m and 2% of WD for WD> 50 m; statistical mapping of objects may be sufficient.
	Error Budget	As per IHO Order 1a to IHO Special Order or tighter to facilitate asset design and installation parameters	As per IHO Order 1a to IHO Special Order or tighter to facilitate asset design and installation parameters

Data Type	Engineering and Site Characterization Surveys		
	Parameter	**Reconnaissance Stage**	**Detailed Stage**
		Typical range of requirements	**Typical range of requirements**
MBES Backscatter	Coverage/Resolution/Data Density/Error Budget	Matched to MBES bathymetry	Matched to MBES bathymetry
	Normalization Required?	No	Dependent on coverage from other systems (i.e., SSS): required if no SSS coverage is achieved
	Mosaic bin size	Dependent on coverage from other systems (i.e., SSS): required if no SSS coverage is achieved: 0.5 meters to 1 meter	Dependent on coverage from other systems (i.e., SSS): required if no SSS coverage is achieved: 0.25 meters to 1 meter
SSS	High Frequency (HF) (NB: Corresponding lower frequency expected to be acquired as part of dual frequency configuration)	For seafloor sediment mapping: 300 kHz For feature identification: 600 kHz	For seafloor sediment mapping: 300 kHz For feature identification: 600 kHz (for objects ≥ 0.5 meters) to 900 kHz (for objects ≥ 0.3 meters)
	Coverage	Partial to 100% coverage	Depending on risk profile of site: 100% (+ nadir coverage) to >200% coverage
	Resolution/Data Density	Sufficient for 1 meter to 2-meter objects; statistical mapping of objects may be sufficient.	Depending on project's need for risk mitigation: sufficient for 0.5 meter to 1-meter objects; statistical mapping of objects may be sufficient.
	Mosaic bin size	Balance between size of dataset and variability of the site, as well as intentions to carry out interpretations on mosaic or waterfall data: 0.1 meter to 2 meter	Balance between size of dataset and variability of the site, as well as intentions to carry out interpretations on mosaic or waterfall data: 0.1 meter to 1 meter
MAG	Line spacing	Default to minimum line spacing required for other systems Additional lines or denser line spacing can be planned as required if areas of specific interest/high risk are known (shipwrecks, debris fields, etc.)	Default to minimum line spacing required for other systems Additional lines or denser line spacing can be planned as required if areas of specific interest/high risk are known (shipwrecks, debris fields, etc.).
	Towing Altitude	Depending on expectations to use this data for estimations of debris density, etc.: piggybacked on SSS to <6 meters above seafloor	<6 meters above seafloor

Data Type	Engineering and Site Characterization Surveys		
	Parameter	**Reconnaissance Stage**	**Detailed Stage**
		Typical range of requirements	**Typical range of requirements**
SBP/S-UHRS	Primary Line Spacing	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 250 meters to 5000 meters Line planning should be optimized according to project specific needs.	Depending on expected geological variability and expectations for further data acquisition: 100 meters to 200 meters Additional lines or denser line spacing can be planned as required if areas of very high geological complexity are identified and have relevance for e.g., cable design and installation Line planning should be optimized according to project specific needs.
	Cross Line Spacing	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 500 meters to 5000 meters Line planning should be optimized according to project specific needs.	Depending on expected geological variability and expectations for further data acquisition: 100 meters to 1000 meters Cross lines can be integrated from multiple survey stages, reducing the need for dense cross line spacing during detailed survey stages Line planning should be optimized according to project specific needs.
	Vertical Resolution	Depending on asset design envelope and sensitivity of expected installation tools: ≤0.3 meters in upper 30 meters	Depending on asset design envelope and sensitivity of expected installation tools: ≤0.3 meters in upper 30 meters

Data Type	Engineering and Site Characterization Surveys		
	Parameter	Reconnaissance Stage	Detailed Stage
		Typical range of requirements	Typical range of requirements
M-UHRS	Primary Line Spacing	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 250 meters to 5000 meters Line spacing may be larger for M-UHRS than S-UHRS, though M-UHRS generally provides greater depth of penetration and improved potential for application of multiple suppression as well as other signal processing techniques. Line planning should be optimized according to project specific needs.	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 100 meters to 200 meters. Additional lines or denser line spacing can be planned as required if areas of very high geological complexity are identified. Line spacing may be larger for M-UHRS than S-UHRS, though M-UHRS generally provides greater depth of penetration and improved potential for application of multiple suppression as well as other signal processing techniques. Line planning should be optimized according to project specific needs.
	Cross Line Spacing	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 500 meters to 5000 meters Line spacing may be larger for M-UHRS than S-UHRS, though M-UHRS generally provides greater depth of penetration and improved potential for application of multiple suppression as well as other signal processing techniques. Line planning should be optimized according to project specific needs.	Depending on expected geological variability, amount of pre-existing geological data available and assumptions to be based on this stage of survey data: 100 meters to 1000 meters Line spacing may be larger for M-UHRS than S-UHRS, though M-UHRS generally provides greater depth of penetration and improved potential for application of multiple suppression as well as other signal processing techniques. Line planning should be optimized according to project specific needs.
	Vertical Resolution	Depending on asset design envelope and sensitivity of expected installation tools: ≤1 meter in upper 30 meters	Depending on asset design envelope and sensitivity of expected installation tools: ≤0.5 meter in upper 30 meters
Seafloor Sampling (See also Section 5.3.2.2.2.5)	Density	1 per significant reflectivity class as identified from SSS/MBES Backscatter data	1 per significant reflectivity class as identified from SSS/MBES Backscatter data, if not yet sufficiently sampled in previous campaigns

Data Type	Engineering and Site Characterization Surveys		
	Parameter	Reconnaissance Stage	Detailed Stage
		Typical range of requirements	Typical range of requirements
3D Surveys	Considerations		Based on previous surveys. 3D techniques are used to reduce interpolation between survey lines in sites with complex and variable geology and/or to provide accurate information on contact positioning and dimensions. 3D M-UHRS surveys may reduce the numbers of boreholes and amount of geotechnical testing required for foundation design and installation.

5.3.1.4 **Marine Archaeology Surveys**

Marine archaeology studies and surveys are conducted to provide information regarding the nature and location of historic properties that may be affected by the installation, operation, and decommissioning of the proposed wind farm development.

Good practice is to let the likelihood of the existence and, not least, preservation of archaeological relicts (Historic property) determine the Marine Archaeology survey program. This should dictate the geophysical methods to be used including, for instance, geophysical line spacing, etc. This initial overview of the archaeological risk should be obtained by carrying out an Archaeological/Geoarchaeological desktop study based on existing knowledge, models, and data.

An Archaeological/Geoarchaeological desktop study shall examine the natural and cultural setting of the offshore investigation area. The natural setting section examines the likelihood of the existence and preservation of potentially buried landscapes conducive to supporting late-glacial and post-glacial pre-contact period human occupation. The natural setting section is often an integrated part of a geological desktop study. The cultural setting examines the probability of finding intact significant historic shipwrecks, sunken aircraft, and other maritime infrastructure, within or immediately adjacent to the investigation area.

> **Guidance Note:**
> An Archaeological/Geoarchaeological desktop study shall examine the natural and cultural setting of the offshore investigation area. Desktop studies should, at minimum, include information on known or reported shipwrecks, downed aircraft, and geological features that may contain preserved paleo-landscape features. Examples of archaeological desktop studies include ICF 2013, Sassorossi et al. 2024, and TRC 2012. The natural setting section examines the likelihood of the existence and preservation of potentially buried landscapes conducive to supporting late-glacial and post-glacial pre-contact period human occupation. The natural setting section is often an integrated part of a geological desktop study. The cultural setting examines the probability of finding intact significant historic shipwrecks, sunken aircraft, and other maritime infrastructure, within or immediately adjacent to the investigation area."

The geophysical equipment spread for Marine Archaeology Surveys is similar to and can be the same as that used for Engineering and Site Characterization Surveys, namely bathymetry (MBES), side scan sonar (SSS), sub-bottom profiler (SBP) and magnetometer (MAG).

- Acoustic data (MBES and SSS) primarily for the detection of indications of objects of potential cultural significance lying on or partly buried within the seafloor. Acoustic data may also be used to inform the potential impacts of development on any cultural

heritage through analysis of seafloor dynamics and can, thereby, aid the recommendation for exclusion zones.
- Sub-bottom profiling (SBP), where both high-resolution and mid-penetration methods may be recommended depending on geological conditions, for the detection of the now-drowned, pre-Holocene paleo-landscapes with the potential for having supporting pre-contact habitation and providing related paleoenvironmental evidence.
- Magnetometry (MAG) for the detection of ferrous artefacts lying on or within the seafloor.

Marine archaeological surveys can be initially performed as a standalone archaeology-specific geophysical survey or, more commonly, as an integrated part of a geophysical site survey. At a later stage, they may involve geotechnical exploration; this may include, for example, visual inspection of vibracores for the presence of intact paleosols, subsampling of organic materials for paleoenvironmental / macrofossil analysis, radiometric dating, or other applicable analyses, as well as reconstruction of the paleo-landscape via integrating the geophysical and geotechnical results into a 3D ground model.

For Historic property lying on or partly buried in the seafloor such as wrecks and debris thereof, other methods of direct investigation may be warranted for confirming the presence or absence of archaeological sites in the investigation area. These methods may include diver investigation, remotely operated underwater vehicle (ROV) survey, including video/camera footage, underwater excavation, etc.

A staged approach to survey should be considered. In this approach, a regional mapping, allowing a generalized assessment of paleo landforms and identification of larger object finds (e.g., large/known wrecks, etc.) can be carried out to inform the requirements for further survey. As the area of impact expected during wind farm development is matured, further surveys offering higher resolution data can be used to refine paleoenvironmental assessment and to mitigate risk to smaller and/or unknown features of potential archaeological significance. Due consideration should be given to the presentation of such staged information to regulatory authorities and stakeholders to ensure adequate dissemination and evaluation of the data ahead of construction.

Guidance Note:
On May 15, 2024, BOEM released the Renewable Energy Modernization Rule which reflects the staged approach discussed in this paragraph. This rule takes effect on July 15, 2024. 89 FR 42602 - Renewable Energy Modernization Rule, Federal Register 89:95 (May 15, 2024) p. 42602.

5.3.1.5 **Benthic Surveys**

Benthic habitat surveys are required to identify the presence, distribution, and condition of seafloor habitats and taxa present within the project area and establish pre-construction baselines of seafloor ecological condition and presence and distribution of essential fish habitat. From a geophysical perspective, the purpose of benthic surveys is to ground-truth geophysical data by conducting grain size analysis, and support development of sediment transport, dispersion, turbidity, and scour susceptibility modeling initiatives. While not always necessary, surveys for benthic ecology may be productively combined with geophysical surveys to efficiently gather applicable ground-truth and ecological data. There are many survey solutions covered in this document; the selection of appropriate strategy will be dependent on site-specific drivers. The following sections include discussion of industry standard approaches and rationale for the selection of the best fit strategy according to individual project conditions.

A comprehensive benthic survey that is designed to identify possible habitat constraints to permitting and construction should be conducted in a timely manner to support siting decisions. When planning the timing of the survey(s), seasonal dynamics of the ecosystem to be surveyed should be considered.

The potential for disturbance to particular types of benthic habitats and taxa shall be assessed and habitat maps shall be developed. Specifically, habitat maps should ascertain those benthic habitats that are designated as essential fish habitat (EFH) for managed fish and shellfish species. Particular attention should be paid to the various life history stages of these species, and review of the potential construction impacts on these species through permanent and temporary disturbance of their habitat shall be assessed. Habitat maps that integrate geophysical and benthic data serve multiple needs in the development process, including resource assessments for permitting, stakeholder engagement, engineering considerations for foundation micro-siting and cable routing.

A comprehensive benthic survey approach may include a combination of sampling and imaging techniques. Consideration should be given to the sediment and habitat complexity and variability within the survey area when designing a benthic survey. In selecting equipment and methodologies for benthic biotic characterizations, it is important to consider the efficiencies and limitations of the potential approaches, as presented in Table 5.3-3. Imaging technologies provide data across a range of fields of view and resolutions. Grabs provide direct samples of seafloor sediments for later laboratory testing, as well as positive macrofaunal species identification. Combining imagery and grabs usually permits landscape scale observations to be made and provides a frame of reference for sediment/habitat heterogeneity.

For direct sampling with grabs, real-time video is best used in conjunction with the grab to ensure the grab deployment does not contact or disturb sensitive or dangerous archaeological features and provides seafloor imagery up to tens of square meters per grab deployment. Video data collected from a variety of platforms (ROV, dedicated sled/frame, mounted on a grab sampler, etc.) records more benthic surface area and can document the existence of mobile pelagic and demersal species, as well as patchy epibenthic invertebrates, seagrasses, and other species. The Sediment Profile Imaging (SPI) tool has the advantage of imaging the seafloor/seawater interface in high resolution, albeit at small field of view. The design of the sediment profile imager mitigates turbidity or water clarity concerns. The Plan View (PV) imager often associated with modern SPI systems is also of high resolution, but image clarity and field of view may be reduced by turbidity issues.

Benthic habitat data collection should be adaptive, if possible, to allow for higher density of data collection in areas suspected to exhibit increased seafloor complexity or heterogeneity as these areas often are highly valued as essential fish habitat. Best practice for these areas is to supplement point data (i.e., still images or grabs) with continuous visual data (i.e., video) to effectively characterize the complexity and extent of habitats. Often this adaptive approach requires access to desktop study results, regional fishing effort data, as well as any reconnaissance geophysical data and therefore comprehensive benthic habitat surveys are best conducted during or after the geophysical surveys.

While, in general, collection of most oceanographic parameters is not critical for benthic surveys, it is recommended that parameters such as temperature, dissolved oxygen, pH, etc. be considered for acquisition given that low-cost and low-logistic sensors can be placed on benthic survey equipment and their data will contribute to interpretation of the survey results.

A fisheries liaison should be selected to engage stakeholders and to inform the fishing community of survey activity and proactively avoid potential gear conflicts.

Table 5.3-3 Benthic Survey Objectives and Associated Tools

Benthic Survey Objectives	Surface Imagery (Video, Plan View)	Sediment Profile Imagery	Grab samples (grain size, benthic community analysis)
Grain size analysis*	-	✓	✓
Classification of CMECS sediment type	✓	✓	✓
Identification of rock outcrops and boulders	✓	-	-
Identification of bedforms (sub-meter to meters)	✓	-	-
Characterization of epifaunal and infaunal community (e.g., CMECS Biotic, Invasives, Sensitive Taxa)	✓	✓	✓
Identification of potentially sensitive seafloor habitat	✓	✓	✓

Infaunal species ID, biomass, population densities, and taxa diversity			✓

*Grain size in SPI is determined optically and can be resolved from large pebbles and small cobbles down to very fine sand at 62.5 microns (4 phi), with silt/clay optically determined as >4 phi

5.3.1.6 **Munitions and Explosives of Concern/Unexploded Ordnance Surveys**

Determining that Munitions and Explosives of Concern (MEC), which also encompasses unexploded ordnance (UXO), risks have been reduced to an acceptable level involves an assessment of the probable MEC hazard and implementing control measures to avoid or mitigate those risks.

Terms relating to explosive hazards in the marine environment are complex and many terms have overlapping definitions. Generally, the term MEC is used in line with the BOEM Research Study OCS 2017-063 and as defined by the Department of Defense Explosives Safety Board as:

- Unexploded ordnance (UXO).
- Discarded military munitions.
- Munitions constituents (e.g. TNT, RDX) present in high enough concentrations to pose an explosive hazard.

MEC does not cover inert munitions, munitions with a low net explosive quantity (NEQ) that do not pose a concern, or stable explosive constituents. Training or practice munitions are not MEC as they have either low or no explosives within them, however, practice munitions should be included in project specific assessment and should be considered due to the difficulty in differentiating these items from explosive ordnance (EO) after many years spent submerged.

A MEC Risk Management Strategy shall be implemented in the wind farm development process. At present, there is no explicit definition of what constitutes a suitable type or level of MEC/UXO geophysical survey. However, it has become both good practice and industry standard to ensure that all MEC/UXO risks are reduced to ALARP (as low as reasonably practicable). The ALARP risk tolerability principle is outlined within both BOEM Research Study OCS 2017-063 and CIRIA Report C754 (2015). Detailed guidance on geophysical survey for UXO/MEC risk mitigation is given within the Carbon Trust (2020) on this explicit topic.

The management of MEC hazards requires personnel with skills in a wide range of disciplines primarily focused on geophysical survey, data processing and interpretation together with UXO specialists and those with hydrographic and positioning, logistics and offshore project management skills. While there is currently no explicit definition of a ‘competent specialist’, developers should seek guidance from those with relevant skills and demonstratable expertise in these areas.

A MEC/UXO mitigation process begins with a Desktop Study (DTS) and Risk Assessment, as described in Section 6.5. BOEM Research Study OCS 2017-063 and CIRIA Report C754 (2015). The purpose of this task is to identify potential sources of MEC hazard, to assess the baseline (pre-mitigation) risk that MEC poses to the Project and then to recommend a strategy to mitigate that risk to a tolerable level. If a previous geophysical survey has been undertaken within the project area, a ‘Gap Analysis’ may also be conducted to determine if the existing data are sufficient to mitigate the risk, and, if not, to specify further survey. Research shall be drawn from the most convenient and reliable sources, cognizant of the need to limit unnecessary cost and delay. Data presented shall be complete and appropriate for risk assessment purposes and fully in line with current good practice.

Following the production of a MEC Risk Assessment, a realistic Risk Mitigation Strategy, following applicable risk tolerability models that cover all activities/stages of the Project that interact with the seafloor shall be defined.

As with other seafloor hazards, geophysical surveys are routinely performed for MEC. However, it is important to match the geophysical data collection with the MEC risk levels and the activities expected to take place during development. Surveys may include operations using geophysical techniques (MBES, SSS, SBP and MAG (potentially in an array configuration)) and inspection campaigns using ROV mounted cameras.

For the specification of geophysical survey operations as part of the risk mitigation strategy, the following shall be considered:

- the set of data quality objectives for the geophysical survey
- the detectable characteristics of the target MEC items
- The possible positions that potential hazards may be found: the corridor to be surveyed; any zonation regarding UXO hazards, conditions, or consequence; and the depth range to be targeted
- the required precision with which hazard anomalies are to be located.

Where pre-existing survey data proves sufficient to mitigate the identified MEC/UXO risk, it may not be necessary to undertake additional survey. However, consideration should be given to the age of the data and the potential for any MEC migration and/or significant seafloor mobility. The time elapsed between collection of any data contributing to risk mitigation and the date of seafloor operations may be significant. While the lateral migration of objects may or may not occur, migration of bedforms certainly could, and a significant elapsed time may lead to migration of a bedform such that a previously undetected hazard becomes apparent or comes into the depth range of interest for an installation or maintenance operation. However, geophysical data itself does not have a ‘shelf life’. It is recommended that a review of the origin of any existing geophysical data considered for use in UXO risk mitigation is performed, with survey objectives and data quality objectives set up as they would be for a new survey. The precautionary principle should then hold while only using existing data when it is positively evaluated as having satisfactory quality for the required purpose.

For MEC geophysical survey, the following are the primary technologies:

- Magnetometry (including gradiometers, multiple arrays, and single magnetometers) measures variation in the magnetic field and is often used for detecting ferrous items.
- Multi-Beam Echo Sounder (MBES) and Side Scan Sonar (SSS) data used in combination for mapping and understanding the distribution of objects on the seafloor
- Sub Bottom Profiler (SBP) used to understand the sub-seafloor structure and may inform analysis of MEC. 3D seismic methods of various resolutions may be implemented, which may provide an accurate location of a sub-seafloor object
- Electromagnetic (EM) methods may be used to detect anomalously conductive material buried to shallow depths in the seafloor.

Ahead of all survey scopes used for MEC risk management, the survey contractor should undertake an equipment verification test (EVT) using a known test item. However, dispensation may be given if existing data are used for lower risk activities or in low hazard areas.

Typically, most items of MEC contain ferrous materials. As such, magnetometry is suitable to detect them when they are on the seafloor, partially buried or fully buried. However, there is a range of UXO items (such as TMB and LMB mines) that have bodies constructed from non-ferrous materials, and, when buried, there are currently few survey methods that can reliably, accurately, and provably detect them without prior knowledge of their location. While experience has shown the 3D SBP methods are technically capable of detecting such items, it has not (to-date) been proven with an actual buried low-ferrous mine find. The majority of previous non-ferrous mine finds have been identified within acoustic datasets with the mine only partially buried. Furthermore, these mines also displayed a magnetic anomaly detectable from a magnetometer array setup. It, therefore, demonstrates that these types of UXO contain components that create a magnetic anomaly. Ultimately, it is important to consider the effort of performing a high-resolution 3D SBP in the overall assessment and management strategy, as it is significantly higher than conventional MEC/UXO survey (magnetometer, side scan sonar and multibeam echosounder).

5.3.1.7 **Construction Phase Surveys**

Construction surveys may include:

- MBES surveys to measure seabed levels to derive the thickness of all placed rock or mattresses installed on the seabed, (e.g. scour protection, cable crossings, jack-up rock pads), by means of comparing pre and post rock installation seabed levels. The survey should be of appropriate resolution for the required installation tolerances, include the entire area of rock, and extend to undisturbed seabed.
  - Reason: To ensure that the protection has been placed within design tolerances in XY, and Z.
- MBES survey of all seabed preparations, e.g., dredging, levelling, boulder removal, by comparing pre and post preparation seabed surveys.
  - Reason: To verify the engineering requirements have been met, and to quantify the activity, e.g., volume of seabed removed, or size/area of boulders cleared.
- Applicable survey techniques to confirm foundation and/or jacket installation height, orientation, and inclination.
  - Reason: To ensure installation meets the engineering requirements
- MBES surveys to support jack-up vessel activities
  - Reason(s):
    - Pre-jacking: to ensure the area is clear of debris, and to ensure that the seabed slope under the spud cans/legs is suitable for the vessel jacking system.
    - Post-jacking: to monitor potential scour around temporary structures, e.g., at a temporary accommodation jack up.
    - Post-departure: to measure post-installation spudcan depressions, if required.
- MBES surveys to measure pre-installation seabed levels, and subsequent trenching and back filled seabed levels during cable installation. MBES may also be used to detect the top of cable where visible on the seabed surface, or in an open trench. Where the cable is buried, another method should be used to detect the top of cable.
  - Reason: To determine if the cable is installed to the engineering requirements.

5.3.1.8 **In Service Life (Condition Assessment) Survey**

Marine surveys conducted during the operation of the offshore wind farm are used to monitor site conditions and determine if these conditions deviate from the design assumptions. The monitoring program should be reviewed following each survey to assess if assumptions still hold; the future monitoring program should be updated accordingly. In service surveys include:

- Using MBES to monitor cable burial depth by means of comparing the new seafloor level with the seafloor level following construction. Monitoring could cover a corridor of 50 m width centered over the installed cable.
    - Reason: To ensure asset integrity and mitigation of obstruction on seafloor for other users of the seafloor
- Monitoring scour conditions at wind turbine or offshore substation foundation locations by means of MBES, which can be of high resolution depending on the scope of the monitoring e.g., in case of exposed cables and/or Cable Protection Systems (CPS) exiting the structures. Monitoring could cover a 50 m radius around the structure, or to the extent of scour, whichever is greatest.
    - Reason: To ensure asset integrity, to validate predictions used in the design, and to assess the need for additional scour protection.
- Monitoring conditions of scour protection at wind turbines and substation foundations by means of MBES. Monitoring could cover a 50 m radius/buffer around the installed scour protection, or to the extent of scour whichever is greatest.

- MBES monitoring of conditions of mattress or rock cable cover works at e.g., cable crossing or as cable remediation could cover a corridor 50 m width centered over the installed cover, or to the extent of scour whichever is greatest.
    - Reason: To ensure asset integrity, to validate predictions used in the design, and to assess the need for remedial actions on the scour protection or the need for additional scour protection.

The monitoring program should comprise monitoring of site representative wind turbines and their associated cables. The selected wind turbine positions should be representative of, but not limited to, the following site conditions:

- The deepest and the shallowest positions, as well as positions affected by strong currents (if any)
- a representative position of each design for wind farms which include positions both with and without scour protection or with different scour protection designs,
- Wind turbine positions representing the varying sub-seafloor sediments on the project.

The survey frequency of the monitoring program is to be suggested on a project-by-project basis and should reflect the seafloor dynamics and design approach of the specific offshore wind farm project. This means that the frequencies are based on actual site conditions, since specific water depth, currents, and sub seafloor geology, etc. play important roles in seafloor change and seafloor dynamics. These surveys may productively be tied in with the scour monitoring survey strategy recommended in Section 8.5.5.

Depending on the site conditions, additional or rescheduled monitoring following a major storm event, typically >50-year event, might be carried out.

Due to the existing infrastructure, a full geophysical UXO-specified survey, including magnetometry, is rarely considered feasible; the only meaningful way to reduce the risk further from buried UXO would be to conduct an electromagnetic survey from an ROV, with the objective of identifying any possible UXO contacts and ensuring the area around each work areas is clear. However, when considering the potential cost to mobilize and perform this survey against the assessed UXO risk, in accordance with the ALARP principle, it is considered disproportionate to the benefit such a survey would offer.

Therefore, within ALARP-certificated surveyed work areas, the UXO level of risk to personnel can be considered to remain ALARP for O&M activities, without the need for additional mitigative geophysical survey, as long as the original residual risk mitigation strategy (possible UXO avoidance and procedural measures) continues to be adhered to. Also, in the absence of a substantial change in modus operandi or other circumstances necessitating a further ALARP test, the UXO risk will remain ALARP through the life of the Project.

5.3.2 **Geophysical Data Acquisition**

Geophysical data acquisition considerations are herein provided for individual equipment types. Where these differ significantly according to the survey objectives discussed in Section 5.3.1, these are divided by sub-section.

5.3.2.1 **Navigation and Positioning Requirements**

5.3.2.1.1 **General**

Navigation and Positioning systems should adhere, at a minimum, to the specifications within ISO 19901-10:2021 (2021), Section 6. However, general standards for wind farms may, in many cases, be tighter than those within ISO 19901-10:2021 (2021); common augmentations are discussed in the following sections.

All geophysical and hydrographic survey data shall be associated as robustly as possible with their navigation data either as a separate file or as records within data headers.

5.3.2.1.2 Coordinate Reference Systems

In the horizontal plane, geodetic regional co-ordinate reference systems are most common. As such, coordinate transformation parameters from the global coordinate reference system used by the surface positioning system(s) (e.g., ITRF2014) to the project coordinate reference system should be established. These should be validated in each acquisition/processing software that will be carrying out geodetic transformations to ensure survey control. Due to the spatial extents (spanning nearshore and offshore areas) of many offshore wind projects, a regional/national vertical coordinate reference system is preferable to local datums. Mean Lower Low Water (MLLW) is the most employed vertical reference. Further, a modeled geoid model (e.g., VDatum) should be employed to improve accuracy beyond that offered by a fixed offset from ellipsoidal height.

5.3.2.1.3 Surface Positioning

ISO 19901-10:2021 (2021) standards reference Differentially corrected GNSS (DGNSS) or clock and orbit corrected GNSS and Precise Point Positioning (PPP) accuracies. For wind farm surveys, requirements for Real Time Kinematic (RTK) or Post Processed Kinematic (PPK) solutions to improve positional data quality and QA should be considered depending on individual project requirements and feasibility of implementing these solutions. This will often depend on the distance of the survey area from a suitable base station. GNSS heights should be seen as a requirement for the vertical reduction of all geophysical data, as shore-based observed tides are often not seen to provide sufficient accuracy for wind farm surveys.

High precision motion sensors, interfaced to the positioning system should be a minimum requirement. The installation of this system shall be well documented and provided with any data transmissions.

For determining the minimum accuracy of the vessel heading sensor, the length of the vessel should be considered. Especially for smaller, nearshore vessels, a larger tolerance for deviation from a baseline may be required, due to the short baseline from bow to stern. Impacts on the resulting positional accuracy of all datasets shall be considered while agreeing minimum acceptable standards.

5.3.2.1.4 Sub-Surface Positioning

In accordance with ISO 19901-10:2021 (2021), an Ultra-Short Baseline (USBL) system should be used for positioning towed equipment. For wind farm surveys, USBL systems should be used in preference to a layback solution where feasible, including water depths significantly less than 25 meters. A tilted-head solution may improve geometry in these cases, thereby improving system performance in shallow water. Where the degree of slant range in shallow water depths inhibits USBL accuracy, alternative towing and positioning solutions, e.g., bow-mounting, sled-towing, etc. may be considered to enable surface positioning before a layback solution is used. However, it is accepted that, in some cases a compromise in positional accuracy may be required to preserve the overall quality of the dataset.

5.3.2.2 Seafloor Mapping

5.3.2.2.1 General

Seafloor mapping is required to determine the water depths, sediment types, morphological features, and objects on the seafloor of wind farm development areas, as well as to characterize localized seafloor dynamics. Where possible, this should be well integrated and informed by interpretation of the immediate sub-surface geology.

Equipment requirements and dataset specifications should be determined by the survey objectives and aims as described in Section 5.3.1. Commonly used equipment types are discussed in the following sections, as well as the main principles which should dictate their configuration, calibration, and overall requirements. The applicability of these equipment types is summarized in Table 5.3-4.

Table 5.3-4: Geophysical Methods Recommended Per Seafloor Mapping Type

Seafloor Mapping Type	Multibeam Bathymetry	Multibeam Backscatter	Side-scan* Sonar	Magnetometer**	Grab Sampling Visual Techniques
Reconnaissance	XXX	XX	XX	X (dependent on survey aims)	X
Engineering and Site Characterization	XXX	XX	XXX	X	X
Marine Archaeology	XXX	X	XXX	XXX	
Benthic Seafloor	XXX	XXX	XXX		XXX
MEC/UXO	XXX	X	XXX	XXX	
Construction/As Built survey/verification	XXX				
In Service Life	XXX	XX			

Notes:

XXX indicates a strongly recommended method, which should be performed. XX indicates a recommended method, which should be performed, and X indicates an optional method which could be performed depending on the site conditions and project objectives.

*dependent on requirements for object detection

**Can be in a gradiometer setup

5.3.2.2.2 **Instrumentation and Acquisition Parameters**

5.3.2.2.2.1 Multi-beam Echosounder (Acquisition)

MBES acquisition should be conducted in accordance with Section 7.2 of ISO 19901-10:2021 (2021).

Relevant standards for offshore wind sites and cable installation corridors are IHO Special Order (0-40 m water depths) and Order 1a (greater than 40 m); depending on survey objectives, tighter standards may be required. The positioning of the survey platform selected, its auxiliary sensors (sound velocity probe, pressure-depth sensor, etc.), and the MBES unit itself shall be integrated and operated such that the required standards are met for feature detection, for limits in uncertainty, and for gridded data resolution.

Density requirements per meter bin should be determined according to the minimum size of object required to be detected, dimensioned, and resolved. Further, the impact of beam footprint should be considered, and, as such, system frequency should be taken into consideration.

Acquisition parameters and MBES survey design (e.g., line spacing and vessel speed restrictions, etc.) should be configured to facilitate adherence to density and Total Propagated Uncertainty (TPU) requirements.

System selection should be based on the ability of the equipment to meet specifications as described above. ISO 19901-10:2021 (2021) specifies that hull-mounted MBES systems have a minimum of 400 beams (soundings per ping) in water depths >30 meters. For water depths typical in wind farm developments (generally <60 meters), the MBES model chosen may have fewer soundings per ping, but this should be based on coverage requirements and dataset specifications (density, TVU/THU, etc.), and should optimize survey efficiency. As such, a dual-head system may be considered to optimize achievement of required densities.

The optimal sounding pattern for nearly all full-coverage seafloor mapping applications will be equidistant; the exceptions being Marine Archaeology investigations and detection of linear seafloor infrastructure (e.g., In Service Life cable inspections), for which collecting MBES in equiangular mode may in select cases be preferable.

Mounting of a system on the survey vessel hull, pole mount or on a sub-towed platform should be determined by potential for adherence to the dataset specifications (absolute positioning, Total Vertical Uncertainty (TVU)/Total Horizontal Uncertainty (THU), density, coverage, etc.), and should maximize overall survey efficiency.

In all offshore wind survey applications during MBES acquisition, sound velocity in the working area shall be assessed via both a continuous sound velocity probe at the MBES transducer, and by periodic or continuous Sound Velocity Profiles (SVP).

A patch test of the MBES system will account for all timing and angular misalignments between the MBES transducer and the Inertial Measurement Unit (IMU) onboard, and should be carried out and the residual values applied to the ensuing data at the start of each campaign and following any mobilization or adjustment of systems, pole mounts, etc.

Bathymetric surfaces are utilized in the processing, interpretation, and reporting of several other aspects of geophysical survey (e.g., SSS contact and MAG anomaly positioning and characterization, sub-bottom and seismic data positioning Quality Assurance (QA), cable-tracker depth of burial calculation, etc.) Requirements for these datasets and final interpretations should also be considered when determining MBES specifications.

5.3.2.2.2.2 Multibeam Echosounder Backscatter (Acquisition)

In general, MBES backscatter data are used to augment interpretation of seafloor sediments and seafloor features, especially in the absence of SSS data of sufficient quality. System selection and acquisition settings are largely determined by the bathymetric requirements and are optimized for the accuracy of the bathymetric surface(s) over the quality of the backscatter data. This may impact normalization of the backscatter data during final processing. Where the quality of backscatter data is of high priority, this should be considered and balanced against the requirements for the bathymetric data.

5.3.2.2.2.3 Side Scan Sonar (Acquisition)

SSS data acquisition parameters should be in accordance with the standards set out in ISO 19901-10:2021 (2021).

In determining the appropriate system frequency, survey objectives for seafloor sediment mapping, seafloor feature discrimination and seafloor object mapping shall be taken into consideration. While lower frequency systems (i.e., nominal 300 kHz) may be most appropriate for seafloor sediment mapping, higher frequencies (i.e., nominal 600 kHz) may be required. For most objects, a frequency of 600 kHz should be adequate; however, higher frequencies (i.e., 900 kHz) may be necessary for objects <0.5 m. The impact of weather, vessel motion and local environmental conditions on the higher frequencies shall be considered during survey design, as shall the limitations online spacing introduced by attenuation of high frequencies at larger ranges; in general, it can be assumed that 900 kHz data will be limited to a 30 m range.

Like the specifications for MBES data, where object detection is a significant component of the survey objectives, survey design should be set for sufficient data density to ensure targets above the established threshold are correctly insonified and resolved. SSS range settings should be tailored accordingly.

Consideration should be given to the towing configuration to ensure overall data quality:

- Minimizing snatch/tugging artefacts
    - Tailoring of tow-point and layback to compensate for survey platform motion during expected survey weather conditions
- Minimizing interference with vessel propulsion
    - Ensure sufficient layback at correct tow-altitudes (10% to 20% of range) While on a stern-tow, or employ a bow-mount or other towing strategy

- Cable type (soft-tow or hard-tow)
    - Shall be tailored to water depth expectations to ensure a balance between layback and towing altitude
- Depressor wing
    - Consider if a wing is required to achieve correct tow-altitude in expected water depths without increasing slant-range beyond the performance of the positioning system and/or to the point of inflating entanglement risk.

In particularly shoal or shallow water environments (<10 m water depth), additional consideration should be given to the towing configuration of the side-scan sonar to preserve data quality and positional accuracy while reducing the effects of interference from the vessel's propulsion.

Dry and wet-testing should be carried out prior to survey operations to ensure data quality, especially with the tow-cable under tension at survey speed in expected current conditions.

The position of the side-scan sonar is typically monitored using an Ultra-Short Base Line (USBL) system. Further, the accuracy of the heading solution, either determined by an internal gyro or other methodology, should be considered to determine the accuracy of the positioning of far-field sonar data. A check of positional accuracy, using a box-in methodology should be required.

5.3.2.2.2.4 Magnetometer/Gradiometer (Acquisition)

For all magnetometer/gradiometer surveys, a system verification should be conducted to ensure the repeatability of the signal in all expected survey directions. Where multiple magnetometers are employed, each system should show a similar, repeatable response in all survey directions. Further, a verification of positional accuracy should be conducted. For archaeological and engineering surveys, a simple box-in or test in reciprocal directions over a ferrous object may be sufficient, whereas for a MEC/UXO survey, a more rigorous verification trial, involving more lines over a known, deployed object, may be required, depending on the risk profile of the site.

The towing configuration of the magnetometer(s) should be designed to ensure adequate separation from the towing platform and other towed equipment to reduce interference. In shallow water environments, particular care shall be taken to ensure that sufficient separation can be achieved while preserving sufficient positional accuracy and safety of the equipment.

5.3.2.2.2.4.1 Acquisition Requirements for Marine Archaeology and Engineering/site Characterization Surveys

Magnetometry for the purposes of engineering should be tailored to debris detection and mitigation of risk to sampling and installation equipment. Additionally, the determination of dykes, faults, and channels, relevant for layout and installation decisions may be augmented by magnetometer data interpretation.

Magnetometry for the purposes of maritime archaeology should be designed according to the assessment of cultural heritage in the specific survey area, whether this be relict landscape or archaeological artefacts with ferrous content. Line spacing, tow altitude and equipment setup should be tailored to the expected risk to cultural heritage and guided by an understanding of the spatial limitations of magnetic anomalies associated with ferrous objects; depending on size and amount of ferrous content, objects can only be detected by magnetometry at limited distances from the sensor.

In areas where a large degree of background magnetic interference is expected, i.e., near ferrous infrastructure, in areas of extreme geological background noise, a gradiometer configuration may be required for confident interpretation of anomalies related to individual artefacts with ferrous content. The optimal configuration of a gradiometer, horizontal or vertical, should be determined according to the expected source of the magnetic interference. In the absence of these sources of interference, and where survey design can mitigate noise introduced by the survey vessel or platform and other equipment, single magnetometers may be sufficient.

For paleo landscape mapping, magnetometry may aid in the identification and interpretation of features such as dykes and channel systems. In combination with sub-surface geological mapping using sub-bottom profilers or seismic sources, this may support the interpretation of landscapes with significance for pre-inundation human habitation. The landscape-scale of this interpretation should be considered in survey design for magnetometry; a line-spacing of approximately 150 meters or greater may be sufficient. Tow-altitudes, where consistent and recorded for consideration during data interpretation, will be of lower importance.

For the detection of potential artefacts with ferrous content, the size and likelihood of encounter should be considered, as well as the geological characteristics of the site; these should be balanced against the collection of data from other sensors to achieve a practical mitigation of risk to cultural heritage. As such, the line spacing, tow altitude and sensitivity of the magnetometer should be advised by an understanding of the archaeological potential of the specific site. An along-track update rate of 10 Hz (nominal 0.25 m sample interval) and equipment sensitivity of 0.1 nanotesla (nT) are typical to ensure adequate data quality and density.

5.3.2.2.2.4.2 Acquisition Requirements for MEC/UXO Surveys

Magnetometry is used in UXO detection campaigns to detect items with significant content of ferrous material; many classes of UXO have significant ferrous content with some notable exceptions. The design of magnetometer acquisition parameters for MEC/UXO detection shall be directly related to the Hazard Assessment to achieve a suitable level of mitigation status. Line spacing and equipment specification should be linked to the identified smallest signal related to the minimum hazard item, and will, therefore, differ according to site-specific risk.

The detectable magnetic anomaly associated with objects containing ferrous material is spatially limited. The amplitude of the anomaly is a function of the mass of ferrous material and an inverse function of the cube of its distance from the sensor. The spatial extent of the anomaly is a function of the distance between the target and the sensor paths.

While gradiometry may be used for MEC/UXO detection, it is not exclusively required, and single magnetometers towed individually or in an array may be used according to the site-specific risk assessment.

Confident identification and location of targets requires that the spatial pattern of an anomaly can be interpolated from the collection of profiles. Derived data products such as analytic signal, while apparently simplifying the map, demand adequate sampling and processing of the Total Magnetic Field anomaly to be accurate (analytic signal requires a calculation of the spatial gradient of the Total Magnetic Intensity). Thus, an objective design criterion for line spacing should be used to assure the viability of the dataset for its intended purpose.

The calculation of line spacing required to give ‘full coverage’ is typically a function of the altitude of the sensor above the maximum depth below seafloor of investigation required, the size of the signal anticipated from the hazard with the smallest magnetic signal, and the noise floor of the sensor. Consideration should also be given to the detection range available from a single magnetometer, using the same principles it is possible to ensure that point activities such as ground investigation sampling are placed within magnetometer coverage to de-risk for MEC.

In practice, the sensors deployed are generally very similar in performance, allowing relatively simple tables to be used to establish an altitude and line spacing tolerance for a given target anomaly size and amplitude. Typical altitudes above seafloor are of the order of 1-5 m, and instrument line spacing ≤5 m, with an appropriate allowance for data gaps. Towfish may have an altimeter incorporated, or another means of determining and controlling towfish altitude shall be established. Where MEC/UXO risk does mandate an instrument line spacing of ≤5 m, a magnetometer array of ≥4 magnetometers are typically employed for survey efficiency.

An along-track update rate of 10 Hz (nominal 0.25 m sample interval) is typical to ensure adequate data density, though this, too, should be determined according to the MEC/UXO risk assessment and mitigation strategy.

Equipment sensitivity of 0.1 nT should be ensured.

System noise should be limited to a maximum of approximately one third of the expected peak-to-peak amplitude of an anomaly related to the minimum threat item at the maximum expected distance from the magnetometer sensor.

The position of the magnetic sensor is typically monitored using a USBL system with a transponder positioned as close as possible to the sensor without inducing signal distortion.

An appropriate allowance for data gaps and short-distance deviations from the agreed specifications shall be determined according to the ALARP principle, MEC/UXO risk assessment and local geological and environmental conditions.

5.3.2.2.2.5 Benthic Data Acquisition: Grab Sampling and Visual Techniques

A combination of the following survey equipment may be used to achieve the overall benthic habitat survey objectives. General information and considerations to use for selection are presented in Table 5.3-5.

Physical sediment and macrofauna taxa sampling may be accomplished with the following systems:

- Modified Van Veen-style grab sampler in single or double bucket configuration (0.1 or 0.04 m^2 bucket sizes are recommended).
- Hamon Day Grab, or similar for hard bottoms,
- Benthic imagery may be collected via one or more of the following techniques:
    - Real time video imagery (often attached to one of the grab devices above)
    - Plan–view (PV) digital still photographic imagery
    - Sediment profile imagery (SPI)
    - Underwater recorded video, such as ROV, towed video sled, drop camera on frame,
    - Note - All imaging devices should have auxiliary lighting sources and laser scaling capability.

Table 5.3-5 Benthic Survey Equipment Characteristics

Equipment	Benefits	Considerations
Modified Van Veen-style Grab Sampler (0.1, or 0.04 m^2)	Collection of physical sediment samples permits multiple types of laboratory analyses per collection, allows positive taxonomic identification of cryptic or non-conspicuous fauna. Can be coupled with real time video system to monitor for sampling biases (bow wave effect) due to lowered sampling equipment, sensitive habitats, and to assess station sediment heterogeneity	If sieving samples for benthic taxonomy, long duration of sample preparation may be required between stations. Multiple attempts may be required to collect acceptable grab or adjust for sediment conditions (can be mitigated somewhat by real time imaging system installed on grab).
Drop Down Still/Video Camera	Adaptability: can be fixed to multiple types of frames and deployed from multiple types of vessels Low logistical complexity Improved weather tolerance due to weighted frame Real-time data and power capability permit flexibility of lighting and imaging systems.	May suffer reduced visibility in high current and turbidity situations Surface conditions may dictate image quality Real time systems may require specialized winches and cable handling equipment, especially in deeper waters.

Equipment	Benefits	Considerations
SPI/PV	SPI Captures abiotic and biotic benthic structures *in situ* SPI imagery is immune to turbidity and therefore can take high-resolution images with limited/no visual artefacts. SPI/PV Images downloaded at regular intervals provide the opportunity to assess seafloor conditions during survey and adaptatively sample as appropriate, such as in area of sensitive and/or heterogeneous habitats. PV imagery with strobe lighting capable of producing (conditions permitting) high-resolution seafloor imagery	Limited field of view. SPI imagery dimensions are 14.5 cm by 25 cm. Extra seafloor area can be imaged by taking multiple replicates per station. SPI/PV system can weigh upwards of 450 kg. Therefore, considerations for vessel size/lift capability required. PV imagery may be degraded by turbidity and water conditions, resulting in smaller field of view or decreased clarity Limited number of SPI/PV systems exist for commercial hire. While that number is growing, current inventory of commercial systems is believed to be below 25 units.
Towed Video	Lightweight, can image larger areas of seafloor during drift transects. Records presence of mobile pelagic and demersal fauna Can be added to other sampling gear (such as sediment grabs) for improved survey efficiency	Optimal image quality obtained when camera movement is limited to <1 knot. Transects may therefore take an appreciable amount of time per station/transect – dependent upon transect length. May be impacted by reduced visibility in high current and turbidity situations. Surface conditions may dictate towed video flight stability and image quality.
ROV/ Autonomous Underwater Vehicle (AUV)	Navigation independent of vessel, multiple cameras, often of very high resolution and excellent illumination. Some ROV systems can collect samples if configured with manipulator arm(s). AUVs can host a range of sensors, and are less influenced by surface wave weather conditions, therefore increasing imaging platform stability and image quality.	In high current situations, a larger ROV may be required. Tether management needs to be considered; larger vessels are usually required for capable ROVs. Small ROV's may allow hand-tendering of tether, but this increases risk of entanglement and back deck HSE issues. May require larger crew to operate/pilot. Larger vessels and Launch and Recovery Systems may be required for larger/more capable ROVs. Increased operational complexity may proportionately increase survey time. AUV methods are often limited by battery duration.

Use of a modified Van Veen-style grab sampler is recommended to increase sample size and penetration of hard sand and gravel.

Guidance Note

The Young grab, or the Ted Young-modified, Van Veen grab sampler is a commonly modified version of the Van Veen grab sampler, with a single or double clamshell bucket made from stainless steel mounted to a supporting frame. The sampling area extracted with this instrument can vary depending on its size. With the modifications, this version of the Van Veen grab sampler is heavier than the traditional version, which allows for better stability, level sampling and provides suitable mounting areas for USBL beacons, real time video and light systems, and water quality sondes or other instruments. The bucket can also be weighted to increase penetration or skids can be mounted to ensure the gear does not sink too deep in soft sediments.

A wide "watch circle" should be established around each benthic station to allow multiple replicates, transects, images, grabs, etc. to be collected from each "station". Twenty-five- to fifty-m diameter zones should be established around each station location to best evaluate the level of habitat heterogeneity or homogeneity of each station.

Grab samples should be examined for acceptability before processing where typical acceptability criteria would include:

- The sampler is not over-filled, indicating over-penetration of the grab.
- Overlying water is present (indicates minimal leakage).
- The desired penetration depth is achieved (usually >50% of bucket is filled).

For sediment analysis, digital photos should be collected of the grab surface with a label included in the photo and a sample log should be kept that may include details on collection attempts, visual descriptions, penetration of the grab, and odor.

Surficial samples for grain size analyses are typically collected from just below the sediment/water interface. Depending upon the purpose of the sediment samples, various guidelines may be used to direct laboratory processing and reporting. It is recommended that clear guidance is provided as to what purpose sediment grain size (i.e., particle size analysis) data will be used for. That guidance is critical to establishing the guideline(s) to be used by each laboratory and to indicate if extra sediment samples are to be taken in the field to account for multiple testing procedures. Guidelines from multiple agencies exist including ASTM (2000) for engineering and modelling studies, US EPA (2001, 2014) for benthic habitat and toxicity tests, the USGS (2000) for surficial sediment mapping, and Regional Sampling programs such as the US Army Corps of Engineers dredged material sampling protocols (USACE, 2016) and the Puget Sound National Estuary Program (WADOE, 2015) which address combinations of the above, plus sediment toxicity studies. All sampling methods should be suited to the individual purpose and needs of a particular development or project area.

For benthic macroinvertebrate taxonomic analysis, sediment samples shall undergo some on-board processing. The sediment from the top of the grab should be gently sieved through a 0.5 mm mesh screen and the excess water drained with care taken such that there is no sample loss or damage. The sieve contents should be preserved in a solution of filtered seawater and 10% buffered formalin. Benthic macroinvertebrate samples should be stored at room temperature, out of direct sunlight. They do not require laboratory processing in a specific time frame. Sediment grabs should be equipped with real-time video systems. Real time video guidance is a best practice to assist with collecting continuous habitat imagery data and prevent the grab from sampling sensitive or dangerous habitats.

Drop Down Video systems can range from simple implementations of a battery, light, and camera in a pressure vessel, to cameras with broadcast quality resolution, multiple lights, and ancillary sensors powered and interfaced with fiber-optic and multi-connector tethers – with a range of systems in between. Drop down systems often include a weighted frame which promotes a consistent field of view and the ability to create a uniform light field by mounting multiple light sources.

Sediment Profile Imager (SPI) involves deploying an underwater camera system to photograph a cross-section of the sediment–water interface. The PV camera is located on the same frame as the SPI camera and both SPI and PV images will be collected during each “drop” of the system. SPI/PV images pairs should be collected at a minimum of 4 replicate locations per sampling station and at least 3 images should be analyzed from the replicate locations. It should be noted that SPI/PV systems are not normally equipped with real time imagery capability.

Underwater video transects provide the largest visual coverage of the seafloor. Video transects are particularly useful in areas of hard or sensitive bottom that are preventative to SPI and grab sampling and in areas of high heterogeneity or complexity. Transects should be collected at a speed of about 1 kt (preferably less) over lengths that cover a representative distance. Depending on survey objectives, transect lengths could range from 10s of meters to several hundred meters. Transects can be collected with a towed video sled, video equipped grab sampler, ROV, or AUV positioned less than a few meters above the seafloor. Note that motion and water clarity will impact the image quality of all these systems.

For all imagery data collection, care should be taken to synchronize the internal clocks of the cameras with each other and that of the navigation system. Ideally, information from the vessel’s navigation system (e.g., GNSS position, heading, speed, etc.) and equipped sensors (e.g., depth/altitude) should be overlaid onto collected imagery (or at least one copy of the imagery to avoid obstruction) through topside integration. During acquisition, imagery should be reviewed in real-time or at frequent intervals for the presence of sensitive, rare, or unexpected species, any nonindigenous species, and potentially sensitive habitat. Additional non-disruptive sampling to increase sampling density may be considered in these areas. Positioning of the sub-seafloor equipment may be improved with the utilization of acoustic positioning systems such as Ultra Short Baseline (USBL) techniques, however in shallow waters <10-20 m these systems are less useful and may not be required altogether if large watch circles around each station, as recommended above, are utilized for survey design. GNSS positioning over the lifting point of the equipment (e.g., A-frame block) should provide suitable positioning accuracy during benthic habitat surveys in most waters.

5.3.2.3 **Sub-Seafloor Mapping**

5.3.2.3.1 **General**

Sub-seafloor mapping is required to image the shallow and medium stratigraphy of the site to understand the structural arrangement of the different geological formations that will be encountered by the wind turbine generator (WTG) and offshore sub-station (OSS) foundations. The distribution and thickness variability of these formations across the development area shall therefore be detailed using sub-seafloor data. Geohazards, such as faulting, paleovalley, or slumping will also need to be identified at an early development stage, as they will constrain the wind turbine layout.

Shallow sub-seafloor sediment layering distribution and thickness variation should also be characterized for cable routing, burial assessment, and installation, as well as for jack-up risk assessments.

5.3.2.3.2 **Resolution and Signal Penetration**

Seismic data vertical resolution and penetration are two important parameters to be considered for the specification of sub-seafloor data acquisition. The vertical resolution (¼ of the dominant wavelength) gives the minimum thickness of a layer that can be detected. It depends on the wave velocity and the dominant frequency. The frequency, which is a function of the medium, will decrease with time on seismic records. An increase in specified data vertical resolution means a decrease in signal penetration. Therefore, there should be a balance between source power, bandwidth and dominant frequency required to deliver the combination of resolution and penetration needed at the site.

Anticipated representative sub-seafloor conditions and contractors' experiences at similar sites can be used to establish appropriate configuration, and a site-specific set of QA metrics defined to assure continuity of performance within and between survey stages.

More information on resolution and signal penetration can be found in Section 8.1.1 of ISO 19901-10:2021 (2021).

5.3.2.3.3 **Equipment Selection for Sub-Seafloor Mapping**

The available equipment for sub-seafloor mapping can be divided into three main categories as standard within the industry:

- Sub-bottom profilers (SBP), such as pinger, chirp, or parametric systems, can be hull-mounted, pole-mounted, or towed. SBPs work with very high frequencies, typically 0.4 to 22 kHz and will enable characterization of the very shallow stratigraphy.
- Single channel ultra-high resolution seismic (S-UHRS), with a towed single seismic source (such as boomer or sparker) and a separate single channel array, consisting of either one hydrophone or several hydrophones that are closely spaced and recorded as one group, such as an 8-element streamer. These systems provide more powerful power supplies than an SBP and can enable better penetration in some soils.
- Multichannel ultra-high resolution seismic (M-UHRS), with a towed seismic source (such as boomer, sparker, or air gun) and a separate receiver consisting of a multichannel

streamer, often 24 channels or more. These systems will enable medium penetration, down to the depths required for foundation design of wind turbines and offshore sub-stations.

Other equipment, such as 3D SBP and cable trackers, can be locally used for sub-seafloor mapping of features, such as buried MEC/UXO or cable detection. Similarly, 3D M-UHRS systems may be used to gain insight into specific features; they would typically not be used to gain general information on a whole wind farm site.

Equipment applicability to each of the survey types is summarized in Table 5.3-6.

Table 5.3-6 Sub-seafloor Mapping Equipment Applicability

Sub-seafloor Mapping Type	SBP	S-UHRS/ M-UHRS*	3D SBP	3D UHRS	Cable Tracker
Reconnaissance sub-seafloor mapping	XXX	XX			-
Engineering and Site Characterization sub-seafloor mapping	XXX	XXX	X	X	
Marine Archaeology sub-seafloor mapping	XX	XX			-
MEC/UXO sub-seafloor mapping	X	-	X		
Construction/As Built survey/verification	X	-			XX

Notes:

XXX indicates a strongly recommended method, which should be performed. XX indicates a recommended method, which should be performed, and X indicates an optional method which could be performed depending on the site conditions and project objectives.

*Depends on penetration requirements and local geology, may not be relevant to cable surveys

5.3.2.3.4 Check of Equipment Performance

Proper equipment tests (manufacturers' tests) should be performed during mobilization, and prior to the start of survey acquisition.

Both source and streamers should be evaluated for integrity and in-survey performance, including positioning systems.

In particular, the equipment should be tested for noise check and repeatability of the source. The source test will also enable to test the tow depth and optimal shooting rate based on the charging time of the Power Pulse Supply with the vessel generator.

A tap test of the main and spare streamers should be performed to check the response of the streamer.

Planned bandwidth and penetration should be verified on-site and QA metrics validated, as per the survey specifications.

5.3.2.3.5 Assessment of Data Quality

Seismic data quality will depend on several parameters, such as sea state and current, the positioning accuracy of the source and streamer, the equipment choice regarding the soil conditions, the frequency used, the shooting interval, etc. Section 8.1.4 of ISO 19901-10:2021 (2021) provides a list of factors affecting data quality and section 8.2.3.2 of ISO 19901-10:2021 (2021) provides guidance on assessing data quality. Additional recommendations are given below.

A set of quality metrics should be stated in the survey specifications. The data quality of seismic lines should be assessed against these specifications onboard the vessel as soon as the line acquisition is completed to evaluate if the survey objectives are being achieved.

Data quality control shall be performed throughout the survey, including checks on signal quality and geometry. Signal and noise analysis, such as shot gathers to assess types of noise, should be performed. Source and streamer position should be carefully checked, including source heave, cable heave and streamer depth. Ideally, Common Depth Point (CDP) fold track plots should be used to assess the CDP bin fold is as expected.

Inadequately collected, seismic data shall be rejected based on the data quality analysis, for example if there are too many dead traces, or if the source is not performing well.

It should be possible to view the SBP raw data in real-time to check for noise.

Seismic data resolution and data penetration should be monitored throughout the survey to confirm that the survey specifications are met.

3D M_UHRS surveys require a set of coverage metrics and tolerances to monitor source and cable navigational data and the trace fold in each CDP bin, as this is critical to define acceptable completion of the survey. Checks on the performance of equipment, bandwidth and penetration are like 2D M-UHRS.

5.3.2.3.6 **Instrumentation and Acquisition Parameters**

Typical acquisition parameters and settings for sub-bottom profiling and seismic reflection systems are provided in Table 5.3-7. More information on S-UHRS and M-UHRS systems is provided in the following sections.

Table 5.3-7 Equipment and common settings for SBP and seismic reflection systems

	SBP	S-UHRS	M-UHRS
Source			
Type	Pinger, chirp, parametric system	Boomer, sparker	Boomer, sparker, or air gun
Frequency	0.4 to 22 kHz for pinger and chirp and up to 200 kHz for parametric systems	0.2 to 5 kHz for boomer 0.1 to 4 kHz for sparker	0.2 to 8 kHz for boomer 0.1 to 8 kHz for sparker 20 to 500 Hz for air gun (NB: typical, may be exceeded)
Firing interval (NB: May be triggered on distance or time)	0.05 m-1 m	Typically 0.5 m	Typical rates: 0,5 m, 1 m, 1.56 m, 3.125 m (NB: tailor to ½ group interval and survey objectives)
Tow depth	Vessel-mounted or sub-towed	0.3-0.5 m	Tailored for ghost-notch
Receiver			
Type	Transducer (same as source)	Single channel hydrophone, typically 8-element streamer	Multichannel with typically 24 to 96 channels, depending on streamer length.
Active length	N/A	Approximately 2 m	Tailored to equal target penetration

	SBP	S-UHRS	M-UHRS
			where possible and dependent on number of channels
Group interval	N/A	0.3 m	Fixed (typically: 1 m to 3.125 m) or variable (typically: 1 or 2 m) between 1st half and 2nd half of streamer.
Streamer depth	NA	Surface-0.5 m	Flat or Slanted streamer If slanted, 0.5° to 1°

5.3.2.3.6.1 Shallow Penetration Seismic: SBP and S-UHRS (Acquisition)

High resolution sub-bottom profiling (SBP) systems, with frequencies between 0.4 and 22 kHz, should be used to investigate near seafloor shallow stratigraphy. Priority shall be given to the use of high-resolution systems, of at least 0.3 m vertical resolution (defined as ¼ of dominant wavelength in depth) or better. Penetration of such systems is expected to be in the order of 5 to 10 m below seafloor, depending on the soil conditions. This information is typically used for cable routing, burial assessment and installation, and shallow foundation zone characterization.

For S-UHRS systems, penetration up to 15 m below seafloor should be targeted, though this will, again, depend on soil conditions. Source-receiver separation should be controlled to minimize the effect of Normal Move Out (NMO) and maximize return of primary energy without introducing noise and/or inflated entanglement risk.

As a minimum, sub-bottom profiler and S-UHRS systems should be provided with a fire control unit. Band-pass, swell filters and gain should be available for display purposes only during the survey; no filters should be applied in real time to the acquisition software.

5.3.2.3.6.2 Medium Penetration Seismic: M-UHRS (Acquisition)

Medium penetration seismic systems include high and ultra-high resolution multichannel seismic systems and should provide sub-seafloor data down to a depth of 50 m to 100 m below seafloor.

The seismic source should be chosen considering the power supply for the required penetration, resolution, and along-track data density. Generally, the use of Sparker systems is recommended, with a variable power supply to be tested.

The streamer should be neutrally buoyant or controlled. The streamer length will depend on the water depths across the site to be investigated and the target penetration depths.

A boom arm can be used to tow the source and position it out of the vessel's wash and propeller noise.

Considerations should be given to ghost effects, which should be minimized by the chosen system. The simultaneous use of a stacked sources and configuration with a slanted streamer can be considered to improve the signal to noise ratio and improve ghost attenuation. Adequate and proven data processing will also be required to avoid unwanted loss of resolution due to the ghost effect.

Accurate data positioning shall be required by using GNSS buoys at the source, and at the start and end of the streamer, where practical. Alternatively, for deep towed systems, acquisition geometry could be logged using tracked buoys or underwater positioning.

A feathering angle of less than 7° should be targeted during data acquisition. Higher feather angles of up to 12° and greater can be accepted, depending on data quality and the survey objectives to be achieved. Acceptable feathering limits shall depend on the cause of the feathering and, consequently, their effect on the data.

The record length should be suitable for the maximum depth required, considering the water depth and the target depth. A record length of 250 ms is often appropriate.

5.3.2.3.6.3 3D Medium Penetration Seismic (Acquisition)

Where the Desktop Study or reconnaissance surveys of the site of interest anticipates particularly complex and variable soil conditions, a 3D medium penetration seismic survey could be considered over targeted areas. This survey would be designed on a site-specific basis.

3D M-UHRS data has advantages over 2D UHRS data in that it:

- has a continuous distribution of seismic traces over an area, where 2D data are arranged in lines; and
- should, in principle, correctly position reflections in 3D space where 2D data may be distorted by assumptions and approximations of the 2D method.
- Its relative disadvantages are associated with additional cost, acquisition and processing time, and sensitivity to conditions at the survey site.

Where geological structure is complex and design requirements require high confidence in the model, or where accurate positioning of sub-seabed features is imperative, 3D M-UHRS data may prove cost effective.

The type of equipment for such a survey is like those for 2D medium penetration seismic (see section 5.3.2.3.6.2), but multiple sources and streamers are used to cover a swath around a vessel track line. Survey track line spacing is set to provide overlap between swaths to deliver continuous areal coverage of the sub-seabed.

Seismic sources and streamers are generally towed from a geophysical survey vessel either directly or via subsidiary arrangements (*e.g.,* frames, paravanes, transverse cables) resulting in a substantial towed array and a requirement for high quality line keeping and positional accuracy.

3D M-UHRS data is generally delivered as a set of time or depth traces associated with surface locations described as a set of ‘bins’ with defined nominal inline and crossline spacing. The survey design should define the bin dimensions to assure adequate resolution of the target structural complexity. Considerations for depth of penetration are like 2D M-UHRS data. A good rule of thumb is to assume that at least 125% of the desired maximum reflection time should be recorded.

Equipment and acquisition settings should, therefore, be specified to deliver:

a) Streamer length at least equal to maximum depth of interest to deliver adequate seismic velocity control;
b) Streamer spacing, hydrophone spacing and shot spacing to support the bin distribution required to image the smallest target feature and minimize aliasing;
c) Source energy and bandwidth sufficient to return the resolution and signal penetration required for the entire interval to be imaged; and
d) Record length adequate to support target dip at the maximum depth of interest.

3D M-UHRS surveys generate large data volumes and processing is time consuming; production of brute stacks for offshore QA may be limited. Careful specification of QA products and metrics, and their inspection to support the survey objectives is imperative and should include checks to address each of points a) - d) above.

5.3.2.3.6.4 Cable Tracker (Acquisition)

Cable depth of burial may be determined through calculating the distance from the cable tracker to the top of the cable and comparing this against simultaneously logged bathymetric data to calculate depth of cover using an EM cable tracker. It is noted that other cable tracking technologies are available and may be considered in lieu of an EM system.

Several parameters should be considered to ensure appropriate system selection and survey design:

- Cable type
    - Diameter – of cable bundle and outer diameter including armoring
    - Material of both cable and armoring
- Status of cable
    - Magnetized or energized
    - Ability to send a tone down it
- Installation information
    - Surface laid or buried (if buried, the cable tracker manufacturer's maximum detection range should be consulted against the asset(s) diameters)
    - Target depth of burial during installation and site report on what was achieved
    - Information on any cable protection to determine expected depth of cover following installation
    - Location and status of existing infrastructure crossings (for safe cable tracking piloting and effective tracking near crossings)
- Seafloor/shallow geology
    - Consider if the system can be tracked on the seafloor or should be flown through the water column
    - Consider the impact of sediment type on equipment performance (e.g., density of material in which cable is buried, morphology of seafloor, etc.)

Ahead of operations, correct calibration values for the system shall be determined for each cable diameter/material to be surveyed. For this, sample cables may be produced and deployed, or test lines may be run across sections of exposed cable.

A key factor limiting the accuracy of the resulting calculation of depth of cable burial is the installation of the MBES system/profilers on the ROV itself. Ideally, these should be mounted on the same platform as the cable tracker to remove common sources of error. If the MBES system is mounted on the survey vessel instead, and cable tracker height is determined using altimeters, the resulting misalignments may be extensive and will need to be mitigated.

The cable tracker, MBES or profilers, and video (if included as a deliverable in the scope) should be mounted with a clear view of the seafloor, ensuring no interference or obstruction from the ROV itself or from other installed equipment.

The position of the ROV should be determined using a USBL system; layback is unlikely to provide sufficient accuracy. Should the cable tracker deviate away from the x,y position of the cable, an error will be introduced, artificially increasing the calculated depth of burial as the system will measure an inflated distance to the top of the cable. This cannot be mitigated in data processing and will necessitate a re-run. Thresholds for acceptable altitudinal and cross-course deviations should therefore be set relative to the asset's diameter and the tracking system employed.

To provide adequate data quality, continuous profiling with real-time display is strongly recommended over solutions providing either cross-sections at intervals along the cable, or data that should be post-processed with no real-time data visualization. An overall accuracy of ≤±10 cm should be expected for burial at ≤0.5 m, with an additional error budget of 10% of burial depth beyond 0.5 m.

5.3.2.3.6.5 Seismic Refraction (Acquisition)

Seismic refraction acquisition can help characterize the sub-seafloor, by providing compressional velocities of soil and rock and associated stratigraphy. The penetration of such seismic refraction techniques is usually around 20 to 25 m for dynamic systems. Static seismic refraction can be particularly useful near landfalls, where the water depths are very shallow and seismic reflection acquisition and results are more challenging. More details on the seismic refraction method and acquisition parameters can be found in section 8.3.1 of ISO 19901-10:2021 (2021).

5.3.3 Geophysical Data Processing

Processing techniques for each geophysical dataset should be tailored according to project priorities with the effort to maximize data interpretability. General principles of industry good practice are highlighted in the following sections.

5.3.3.1 Seafloor Mapping

5.3.3.1.1 Multibeam Echosounder (Processing)

The objective of the MBES processing is a Digital Terrain Model (DTM) which meets IHO S-44 minimum standards and survey-specific specifications for coverage and sounding density, THU/TVU, and feature detection.

MBES data shall be corrected for vertical and horizontal misalignments in the first stage of processing, which involves the integration post-processed navigation, tidal corrections (either observed tides or GNSS-derived), and sound velocity measurements. The resulting merged DTM should minimize artefacts associated with refraction, tidal busts, and poor positioning (horizontal misalignment of adjacent overlapping lines).

The second stage involves the de-spiking of MBES sounding data to minimize the influence of outliers on the resulting DTM and improve the THU/TVU of the surface. While careful selection and monitoring of the acquisition settings will reduce the noise and spurious soundings present in the data, an objective, efficient cleaning routine (often dependent on site-specific conditions) should be enacted and followed. Soundings should be rejected rather than deleted from the MBES database. Where automated filters are employed, diligent Quality Control (QC) should be performed to ensure valid soundings and features have not been rejected.

Gridding algorithms and the selection of shoal, deep or average elevations should be decided based on survey priorities. Depending on the objectives of an individual survey campaign, multiple surfaces may be required; a point cloud deliverable may also be more suitable than a gridded surface. While a Combined Uncertainty and Bathymetric Estimator (CUBE) algorithm may be optimal for the generation of contours, it may not be best suited to feature detection. The end use of the dataset should be considered.

5.3.3.1.2 Multibeam Echosounder Backscatter (Processing)

The level of processing required for backscatter data will vary according to project aims. In all cases, backscatter data processing should be carried out on processed MBES files with corrected positioning, as described in Section 5.3.3.1.1.

Gain should be optimized to emphasize changes in seafloor sediment composition and morphological features on the seafloor. Depending on survey aims, normalization of intensity between survey lines may be required. Due to the time investment in this processing step, the purpose of this and volume of data should be taken into consideration before applying this step.

5.3.3.1.3 Side Scan Sonar (Processing)

Side scan sonar data processing should be tailored to the specific priorities of the survey campaign.

Processing steps may consist of:

- Navigation processing
    - USBL processing
    - Heading processing
    - Map corrections/rubber sheeting/bathymetric alignment. NB: the overall effect of these corrections on the data interpretability and general positional accuracy shall be considered before these can be applied
- Seafloor tracking for nadir removal
- Gain application
    - Consideration should again be given to survey aims during the selection and application of gain algorithms, whether this be to optimize feature detection or seafloor reflectivity detection
- Artefact reduction
    - While tools are available to reduce artefacts such as snatch/tugging in the data, the overall impact of these methods on the interpretability of the data in line with project priorities should be taken into account
- Mosaic construction
    - Similarly, the end-use of the products should be considered in choosing the ordering of data, the construction algorithm, and the form of exports (i.e., single-line images or full mosaics).

5.3.3.1.4 Magnetometer/Gradiometer (Processing)

5.3.3.1.4.1 Data Processing Requirements for Marine Archaeology and Engineering/site Characterization Surveys

Magnetic data processing, whether single magnetometer or in a gradiometer configuration, shall include the following basic processing and result in a total field magnetic signal without diurnal ionosphere variation and a residual magnetic signal, bias from vessel or shallow geology:

- Removal of noise and diurnal effect
- Calculation of the residual field
- Anomaly picking, including reconciliation and measuring of identified anomalies
    - This may be done on gridded or individual line profile data.

For transverse gradiometer data, magnetic data processing should also include the following steps:

- Removal of noise
- Derivation of quasi-analytic signal
- Anomaly picking, including reconciliation and measuring of identified anomalies
    - This may be done on gridded or profile data.

Batch processing and automated algorithms should be manually assessed for suitability across the full dataset.

While magnetic data processing generally focuses on identification of anthropogenic items, particularly for archaeological and site characterization processes, data should be analyzed for the magnetic signature of geological formations (i.e., faults, dykes, etc.) which may have relevance for paleo-landform interpretation and/or engineering parameters.

The impact of gridding algorithms and parameters shall be considered to optimize the identification of anomalies and/or geological features according to the survey objectives.

5.3.3.1.4.2 Data Processing Requirements for MEC/UXO surveys

Magnetic data processing for MEC/UXO surveys should follow the same basic requirements as outlined. However, anomaly mapping for these surveys is typically carried out on gridded surfaces with a strong focus on the accurate positioning, dimensioning and characterization. Dimensions should include inflection point to inflection point wavelength and peak to peak amplitude; characterization should include description of the nature of the anomaly, i.e., monopole, dipole, complex, etc.

Generally, results of mass and depth of burial estimations using the half-width rule and Euler's equation are not thought to be sufficiently reliable to use in the mapping of items of MEC/UXO and should not be relied upon for engineering purposes. Automated target selection processes may be used but should be complemented with both manual target measuring and QC processes.

Anomalies detected and mapped during MEC/UXO surveys should be listed with accurate positions, dimensions, characterization, and correlation to objects mapped on overlapping acoustic datasets.

5.3.3.1.5 Data Processing: Grab Sampling and Visual Techniques

Benthic habitat survey data should be used to classify the Substrate and Biotic components of benthic habitats using standard methods, namely, the federal Coastal and Marine Ecological Classification Standard (CMECS; FGDC 2012); applying modifications to the CMECS Substrate component according to NMFS (2020) should be considered and applied to the extent practicable. Sediment analysis results from grab samples, using procedures appropriately defined by developers for this purpose, paired with images of the grabs onboard the vessel (to identify shell content), can be used to classify the CMECS Substrate component according to NMFS (2020) modified CMECS for the purposes of habitat mapping. Alternatively, SPI/PV or towed video can be used in combination to classify CMECS Substrate Group and Subgroup. SPI can be used to identify the distribution of sizes and dominant grain size present within the sediment column; these results are often well correlated with grain size classifications derived from grab samples. Additionally, PV images (or other still-camera or video with scaling lasers) provide valuable information on intra-station variability; for example, at a heterogeneous station replicate PV images or video transects are more likely to detect patchy presence of cobbles than a single grab sample collected at the same location. Conspicuous epifaunal (PV) and infaunal (SPI) community analysis metrics (abundance, diversity, etc.) and/or quantitative and qualitative characterizations (percent cover, dominant functional biotic community) derived from imagery can be used to classify the CMECS Biotic component.

5.3.3.1.5.1 Grab Sampling

Benthic grab samples shall be analyzed by a qualified analytical laboratory using standard American Society for Testing and Materials (ASTM), EPA, or other recognized standard analytical methods (refer to Section 5.4.7.2 for guidance regarding accreditation of laboratories).

Samples should be handled in accordance with Section 5.4.5.6.2. Grain size of samples shall be analyzed as described in Section 5.4.6. Results shall also be expressed as percent gravel, sand (very fine, fine, medium, coarse, and very coarse), silt, and clay using the Wentworth (1922) scale.

Benthic grab samples slated for benthic macroinvertebrate analysis shall be sieved with a 0.5 mm sieve and preserved onboard the vessel (see section 5.3.2.2.2.5) and shipped to a qualified analytical laboratory. The receiving laboratory should use USEPA National Coastal Condition Assessment (2015), or other documented standard protocols for benthic community analysis. Common preservation chemicals used in the field include buffered formalin (pre-diluted 10% concentration preferred).

5.3.3.1.5.2 Visual Techniques

Processing techniques for imagery will vary depending on the survey goal(s) (ground-truthing geophysical data, or benthic habitat survey), type of camera used, and imagery software selected for processing and editing.

For benthic habitat surveys, the following information should be identified within the collected imagery identified or selected for image analysis:

- The dominant benthic macrofaunal and macrofloral communities and substrates present;
- Potentially sensitive seafloor habitats, specifically associated with essential fish habitats (EFH), and other biologically sensitive resources in the vicinity of proposed structures;
- Hard bottom substrates (especially with epifauna or macroalgae cover);
- Presence of any invasive taxa;
- Vegetated habitats (e.g., submerged aquatic vegetation, seagrass);
- Bedforms (e.g., sand ripples); and
- Other important biogenic habitats (i.e., structure formed by organisms).

Additionally, sediment profile images should be analyzed for apparent grain size, infaunal successional status estimation, and biogeochemical indicators such as apparent redox potential discontinuity depth.

5.3.3.2 Sub-Seafloor Mapping

5.3.3.2.1 Shallow Penetration Seismic: SBP and S-UHRS (Processing)

Shallow penetration seismic data may be acquired by vessel-fixed or towed instruments. Data processing may be relatively limited, but attention should be given to optimization of the signal pulse and bandwidth, compensation for swell and tide, mitigation of noise originating from the vessel and other instruments (particularly MBES or other acoustic sources), mitigation of random noise and appropriate gain recovery to compensate for attenuation. The frequency content should be maintained during processing.

The SBP data shall be tidally corrected, reduced to the project vertical datum, and shall be associated with accurate positional information preferably loaded in trace headers.

5.3.3.2.2 Medium Penetration Seismic: M-UHRS (Processing)

As a minimum, the processing sequence should include the following steps (listed in no particular order), which should be tested beforehand to select suitable parameters and confirm that the data processing sequence delivers the penetration and the resolution requirements as per the survey specifications:

- Noise filtering
- Static corrections to correct for swell-related source and receiver motion during seismic data acquisition.
- Pre-stack deconvolution
- Velocity analysis, at intervals to be determined depending on the variability of the soil conditions anticipated at the site. At locations of interest such as turbine locations, or areas of rapidly changing geology (e.g., paleochannel), some more velocity picks should be performed, to improve the processed data.
- Amplitude recovery
- Normal Moveout (NMO) correction
- Common Depth Point (CDP) ensemble Stacking
- Receiver de-ghosting
- Multiple removal

- Migration to recover true geometry of primary reflections.
- Tide correction and reduction to project vertical datum.
- Time to Depth conversion.

More detail on seismic data processing can be found in Section 8.2.3.3 of ISO 19901-10:2021 (2021).

5.3.3.2.3 3D Medium Penetration Seismic: M-UHRS (Processing)

Processing of 3D M-UHRS data requires a set of operations to attach positioning information to traces and assign the geometry of the survey to assemble the contributing traces to each CDP 'bin'. Subsequent pre-stack processing and QA may be like 2D M-UHRS, (refer to section 5.3.3.2.2). It is possible to exploit additional benefits of 3D data in noise attenuation, demultiple, migration and other pre-stack processes. Post-stack, single trace processes are similar but again multi-trace processes are performed in 3D.

The importance of the seismic velocity field (as a parameter for migration) is increased in 3D data and intensity of velocity analyses and its QA should be defined to suit the target structure of interest.

5.3.4 Geophysical Data Interpretation and Reporting

5.3.4.1 Engineering and Site Characterization Surveys

Geophysical investigations require diligent and comprehensive reporting. Reporting may be split into operational, data processing and interpretation stages for most measurements, but care should be taken to identify where data processing operations have an interpretative component. In the case of multi-channel seismic data for instance, velocity analysis may be such a process.

The geophysical data interpretation and reporting shall cover all the requirements defined as part of the scope of work. Interpretation of geophysical data should consider the type of survey and its purpose, which shall be included in the Introduction of the report.

The geodetic parameters, project projection and vertical reference used for the interpretation and reporting should be clearly stated at the beginning of the report. Units shall always be reported and preferably should be SI (e.g., meters rather than feet). Where alternate units are used, the SI conversion may be stated next to these in parentheses.

To facilitate appropriate use of geophysical datasets, data accuracies and uncertainties shall be detailed in a separate section. Uncertainty in the vessel positioning and sub-sea positioning of equipment and sensors shall be mentioned, along with accuracy in depth measurements. All assumptions used for the data interpretation, such as the time to depth laws for seismic interpretation should also be clearly stated.

Statements on quality of each dataset should be given, with data examples showing the achieved data resolution and penetration for sub-seafloor data.

A section detailing the geological and structural background of the area of interest should be included in the report, with clear references. This section may be concise but should give enough details to quickly understand the geology of the area and particular site conditions. Relevant information from the Desktop Study should be incorporated.

Minimum, maximum, average water depths and water depths at proposed infrastructure locations should be detailed. The general water depth trend in the survey area should be described, with general slope and maximum seafloor gradients given, with data examples where appropriate.

Seafloor features shall be interpreted from multibeam bathymetry, side scan sonar data and gradiometer data. The orientation of large features, such as scarps or ridges should be given, and their dimensions where possible. Areas of suspected mobile seafloor, presenting ripples, sand waves or banks shall be defined with their orientation and wavelength, and compared to current direction. The different relative seafloor reflectivities in the side scan sonar mosaic should be interpreted using relevant available ground truthing information.

A listing of the interpreted seafloor obstructions, with their dimensions and interpretation (debris, boulder, wreck, etc.) shall be given in the report, as an appendix and/or as a digital deliverable.

Interpreted magnetic anomalies shall also be listed and correlated with side scan sonar/MBES contacts where applicable. The potential match between side scan sonar and magnetometer contacts should be used to aid the interpretation of the contact.

Sub-seafloor features and shallow geological units are to be interpreted from the sub-bottom profiler data and/or S-UHRS and/or M-UHRS seismic datasets. All available data which can help with the interpretation of the soil units and formations encountered shall be used, such as existing vibracores or boreholes, or in-situ measurement, such as CPT. The most significant and laterally continuous seismic horizons should be picked across the area of interest. The interpretation shall be checked for consistency at seismic line crossings. These seismic horizons should be correlated to borehole information if available, by applying seismic velocities which are reasonable for the expected soil conditions. Specific sub-seafloor data interpretation deliverables are detailed in section 8.1.4 of the ISO 19901-10:2021 (2021).

For export cable routes, profiles illustrating the geological and other ground conditions should be generated.

Any relevant geohazard such as faults or shallow gas (Section 6), shall be described and mapped within the report.

5.3.4.2 **Marine Archaeology Surveys**

Interpretation of geophysical data for marine archaeology should focus on integration of available datasets. While acoustic data (MBES/SSS) may be most applicable to identification of objects of potential archaeological significance situated on or slightly buried in the seafloor, these should also be considered in the interpretation of magnetic anomalies and relict landscapes. Datasets should be considered holistically, and interpretation should be informed by the archaeological desktop study, to ensure a well-grounded assignation of risk.

It is critical that such interpretation is done with an in-depth understanding of the limitations and opportunities of geophysical data. As such, metadata and details regarding the acquisition and quality parameters of the geophysical data should be considered a critical tool to the archaeological interpretation.

To map the paleolandscape, Marine Archaeologists establish a date range and reference a specific sea-level model to delineate the approximate location of major and minor landforms and areas of interest. Where possible, interpretation of paleolandscapes should be informed by a combination of geotechnical and geophysical data. While, in a staged approach, initial interpretations on geophysical data alone may be sufficient, conclusions drawn from this stage should ideally be confirmed once further geotechnical parameters are available. Where required, marine archaeological analyses should isolate key unit packages to be sampled using vibracores and/or boreholes to acquire data to be used in paleolandscape reconstruction.

5.3.4.3 **Benthic Habitat Surveys**

Data collected during comprehensive benthic surveys provide information on several abiotic and biotic features of the seafloor. This data and derived interpretations serve multiple wind farm development purposes and are likely to be used across multiple disciplines, audiences, and reports. Sediment composition data inform geophysical descriptions and site characterizations. Sediment data may also be used to inform sediment transport and scour susceptibility modeling, as well as the project ground models, and would accompany reporting products related to these topics.

Data results on both sediment type and biotic communities, including CMECS classifications, should be used to inform comprehensive benthic assessments. These data results should be integrated with geophysical survey results (MBES, SSS) to produce habitat maps used to support assessment of potential impacts to environment (e.g., EFH).

5.3.4.4 MEC/UXO Surveys

The aim when grading geophysical targets for MEC risk management purposes is to provide a list of features that have the potential to be UXO/MEC, that can be highlighted to the project for further mitigation through either inspection or avoidance. As such, the UXO specialist should attempt to balance the impracticability of MEC target discrimination by interpreting geophysical datasets, with the aim of identifying targets which have the greatest probability of being MEC within the threat spectrum identified by the project specific risk assessment.

Thorough discrimination of benign features from MEC is an important stage in the MEC risk management process hence the requirement to consider all available datasets and the findings from the desktop hazard assessment.

For potential MEC target discrimination, discrete anomalies and polygon areas will be identified from the processed data flagging objects that model as MEC (anomalies) or have the potential to mask the presence of MEC (polygons). ‘Potential MEC’ and polygon areas are given exclusion distances to mitigate risk until or in lieu of these objects being removed, or masked areas being resurveyed.

The output from the data interpretation stage is then presented within a ‘Potential MEC Target List’, containing all anomalies that are potentially MEC with coordinates of their precise locations and a unique designation for each target (usually following the naming convention of the survey contractor).

5.3.4.5 Construction Phase Surveys

Data interpretation and reporting for the construction phase surveys detailed in this document will focus primarily on the use of the acquired MBES datasets to demonstrate that engineering requirements have been met.

5.3.4.6 In Service Life (Condition Assessment) Survey

Data interpretation for In Service Life surveys will focus primarily on the comparison of updated bathymetry against highly accurate construction-stage bathymetric data to ensure asset integrity and adherence to design assumptions. Differences in the bathymetric surfaces between years of survey data acquisition will highlight changes in i.e., cable depth of burial or scour around assets. Where these deviate from expectations established in the design stage, this may flag the need for remedial action or simply for further monitoring surveys.

5.4 Geotechnical Site Investigation

5.4.1 Objectives, Planning, and Requirements

5.4.1.1 Objectives

Marine site investigation plans should ensure relevant and adequate soil data are available as appropriate to the project phase. In particular, the acquired data should be sufficient to enable site characterization with respect to foundation and cable design and installation, construction activities, site operational aspects of the offshore facility, and the level of acceptable risks for the foundation and integrity of the offshore structures, including cables.

> **Guidance Note:**
> ISO 31000:2018 provides guidance on risk-management principles.

The general objectives of a marine geotechnical soil investigation are to establish the characteristics and mechanical properties of the seabed and sub-surface soils by acquisition, evaluation, and presentation of geotechnical information derived from methods relying on tools penetrating the seabed.

Guidance Note:
Valuable guidance on the requirements for scoping geotechnical site investigations for offshore wind farm facilities can be found in ISO 19901-8:2023, DNVGL-ST-0126 (DNV GL, 2021), DNVGL-RP-C212 (2021), and the SUT OSIG guidelines (2022). Guidance on the requirements for scoping site investigation for subsea cable routes can be found in DNVGL-RP-0360 (DNV GL, 2016) and SUT OSIG Guidelines (2022).

The ISO 19901-8:2023 Marine Soil Investigation standard is referred to in this section as the basis to perform a geotechnical site investigation for offshore wind facilities. Deviations from and supplementary guidance to ISO 19901-8:2023 that should be applied to offshore wind facilities are highlighted in this document.

5.4.1.2 Planning

The planning of the marine site investigation should follow the recommendations presented in Section 5.2 of ISO 19901-8:2023

5.4.1.3 Scope of Work

5.4.1.3.1 Development of Scope of Work

As per Section 5.3.1 of ISO 19901-8:2023 with the following additions:

Engineering and site characterization surveys typically comprise various types of investigations, including geological and geophysical surveys along with geotechnical investigations. The latter consists of relevant in situ testing such as cone penetration testing (CPT), supported by sampling for subsequent laboratory testing. A qualified and experienced geotechnical engineer with knowledge of the site should lead the scoping and execution of the site surveys and investigations, including the preparation of laboratory testing schedules.

The geotechnical investigations and interpretations informing the detailed design should at each specific position provide adequate information about the ground conditions, their layering, and the range of classification and engineering properties (e.g., strength and stiffness properties, stress conditions, etc.) to a depth below which the existence of a formation has a low likelihood of influencing the safety or performance of the facility support structures, or of the installation vessels or rigs to be used. Clause 7.3.1.5 of DNVGL ST-0126 and Section 7.3 of SUT OSIG (2022) provide specific guidance to such depths dependent on facility and foundation concept.

In cases where the ground conditions across a larger area show low spatial variability, geotechnical data for detailed design may be assessed for a subgroup of turbine foundations instead of individually at each position. On the other hand, in cases of high spatial variability, it may prove necessary to carry out one or more borings or CPTs to adequately inform detailed design of a foundation, especially where such a structure covers a large area of seabed (e.g., for offshore substation jacket structures). A similar principle can apply to the assessment of ground conditions along cable routes.

In the event of unavailable position-specific investigations for a facility design, appropriate assumptions based on literature or representative site data will have to be made for the engineering properties for design, which shall then be confidently validated by the ground model or by supplementary site investigations performed before installation begins.

Site Investigation data should be obtained for the full range of ground conditions and geological features encountered over the length of the cable route and cover a corridor of sufficient width to provide adequate information for design, installation activities, and operational activities. The site investigation at landfall should be tailored to the proposed installation methodology whether open cut or horizontal directional drilling (HDDs). Adequate geotechnical testing at sufficient and optimized intervals should be undertaken to ensure sufficient input to design.

Specific guidance on the requirements related to scoping of site investigations for subsea cables is presented in Section 3.4 of DNVGL-RP-0360 (DNV GL, 2016) and Section 7.3 of SUT OSIG Guidelines (2022).

A geotechnical engineer should specify adequate in-situ testing and/or sampling at foundation locations and at sufficient and optimized intervals based on geophysical investigation along the proposed transmission cable route to shore. The investigations should provide data for all important sediment and rock strata to determine their strength classification, deformation properties, dynamic characteristics, and thermal properties as appropriate to the facility of the wind farm being designed. The following in-situ techniques are typically used ; seabed or downhole CPT performed with a friction cone or with a piezocone, pore pressure dissipation tests, seismic cone penetration test, in situ thermal property measurements, P-S logging test, ball and T-bar penetration tests, field vane test, temperature and depth profiling test, and sampling (downhole sample boring, vibracore sampling, piston core sampling, box core sampling) at foundation locations and at sufficient and optimized intervals based on geophysical investigation along the proposed transmission cable route to shore. .

As discussed in Section 5.3.1.4 marine archeology surveys may extend into the geotechnical site investigations; If relevant, the requirements for marine archaeological analyses (sampling, age dating etc.) should be considered when planning geotechnical surveys. Archaeological analyses may require laboratory established sample dates for each geological unit of interest.

5.4.1.3.2 Default and Project-Specified Application Classes/Methods

This section presents the default application classes and default methods that shall be used if not otherwise given in project specifications. As per Section 5.3.2 of ISO 19901-8:2023.

5.4.1.4 Other Requirements

As per Section 5.5 of ISO 19901-8:2023.

5.4.2 Deployment of Investigation Equipment

5.4.2.1 Deployment Modes

5.4.2.1.1 General

As per Section 6.1.1 of ISO 19901-8:2023.

5.4.2.1.2 Non-Drilling Mode

As per Section 6.1.2 of ISO 19901-8:2023 with the following addition at the end of paragraph 1:

Recent advances in high-thrust capacity (200 kN thrust) seabed CPT units have achieved comparable penetrations in stiffer and denser soils.

5.4.2.1.3 Drilling Mode

5.4.2.1.3.1 General

As per Section 6.1.3.1 of ISO 19901-8:2023.

5.4.2.1.3.2 Vessel Drilling

As per Section 6.1.3.2 of ISO 19901-8:2023.

5.4.2.1.3.3 Seafloor Drilling

As per Section 6.1.3.3 of ISO 19901-8:2023.

5.4.2.2 Accuracy of Vertical Depth Measurements

5.4.2.2.1 General

As per Section 6.2.1 of ISO 19901-8:2023.

5.4.2.2.2 Factors Affecting the Accuracy of Vertical Depth Measurements

As per Section 6.2.2 of ISO 19901-8:2023.

5.4.2.2.3 Specification of Depth Accuracy Class

As per Section 6.2.3 of ISO 19901-8:2023.

5.4.2.3 Positioning Requirements

As per Section 6.3 of ISO 19901-8:2023.

5.4.2.4 Interaction of Investigation Equipment with the Seafloor

As per Section 6.4 of ISO 19901-8:2023.

5.4.3 Geotechnical Drilling and Logging

5.4.3.1 General

As per Section 7.1 of ISO 19901-8:2023.

5.4.3.2 Project-Specific Drilling Requirements

As per Section 7.2 of ISO 19901-8:2023.

5.4.3.3 Drilling Objectives and Selection of Drilling Equipment and Procedures

As per Section 7.3 of ISO 19901-8:2023.

5.4.3.4 Drilling Operations Plan

As per Section 7.4 of ISO 19901-8:2023 with the following addition to the third bullet underneath paragraph a:

- Including risk of encountering shallow gas, existing infrastructure, or unexploded ordnance.

5.4.3.5 Recording of Drilling Parameters

As per Section 7.5 of ISO 19901-8:2023.

5.4.3.6 Borehole Geophysical Logging

As per Section 7.6 of ISO 19901-8:2023.

5.4.4 In Situ Testing

5.4.4.1 General

As per Section 8.1 of ISO 19901-8:2023 with the addition of the following third deployment method:

e) top-push mode from a platform in shallower water, where the tool is lowered through a casing string between the deck of the platform and the start depth of the test.

5.4.4.2 General Requirements for the Documentation of In Situ Tests

As per Section 8.2 of ISO 19901-8:2023 with the following addition to section a:

a) site geographical details, including:
    - water depth (e.g., relative to lowest astronomical tide (LAT); mean sea level (MSL) or mean lower low water (MLLW) in areas with significant tidal fluctuation,
    - reference datum.

5.4.4.3 Cone Penetration Test (CPT)

5.4.4.3.1 General

As per Section 8.3.1 of ISO 19901-8:2023.

5.4.4.3.2 Equipment

As per Section 8.3.2 of ISO 19901-8:2023.

5.4.4.3.3 Test Procedures

5.4.4.3.3.1 Selection of Equipment and Procedures

As per Section 8.3.3.1 of ISO 19901-8:2023.

5.4.4.3.3.2 Preparation for Testing

As per Section 8.3.3.2 of ISO 19901-8:2023.

5.4.4.3.3.3 Pushing of Cone Penetrometer

As per Section 8.3.3.3 of ISO 19901-8:2023

5.4.4.3.3.4 Dissipation Tests

As per Section 8.3.3.4 of ISO 19901-8:2023.

5.4.4.3.3.5 Test Completion

As per Section 8.3.3.5 of ISO 19901-8:2023 with the following addition:

For top-push CPT from a surface vessel/platform, the zero reference shall be at deck level, and the reference readings at the bottom of the casing string shall be treated with caution. The top-push CPT may commence close to the seafloor or may be continued from a deeper depth like the drilling mode.

5.4.4.3.3.6 Equipment Checks and Calibration

As per Section 8.3.3.6 of ISO 19901-8:2023.

5.4.4.3.4 Presentation of Test Results and Reporting

As per Section 8.3.4 of ISO 19901-8:2023 with the following addition:

In general, the test results should be plotted with a depth scale of 1 scale unit = 1 m, but for shallower profiles (e.g., cable route investigations), an enlarged scale can be used provided it is maintained across the acquired data set.

5.4.4.4 Pore Pressure Dissipation Test

5.4.4.4.1 General

As per Section 8.4.1 of ISO 19901-8:2023.

5.4.4.4.2 Equipment

As per Section 8.4.2 of ISO 19901-8:2023.

5.4.4.4.3 Test Procedure

As per Section 8.4.3 of ISO 19901-8:2023.

5.4.4.4.4 Presentation of Results

As per Section 8.4.4 of ISO 19901-8:2023.

5.4.4.5 Ball and T-Bar Penetration Tests

As per Section 8.5 of ISO 19901-8:2023.

5.4.4.6 Seismic Cone Penetration Test (SCPT)

As per Section 8.6 of ISO 19901-8:2023.

5.4.4.7 Field Vane Test (FVT)

As per Section 8.7 of ISO 19901-8:2023.

5.4.4.8 Other In Situ Tests

As per Section 8.8 of ISO 19901-8:2023 with the following addition:

P-S logging to measure the velocity of shear waves and compressional waves can be needed for a complete marine soil investigation program.

5.4.5 Sampling

5.4.5.1 General

As per Section 9.1 of ISO 19901-8:2023.

5.4.5.2 Purpose of Sampling

As per Section 9.2 of ISO 19901-8:2023.

5.4.5.3 Sampling Systems

As per Section 9.3 of ISO 19901-8:2023.

5.4.5.4 Selection of Samplers

5.4.5.4.1 General

As per Section 9.4.1 of ISO 19901-8:2023.

5.4.5.4.2 Drilling Mode Samplers

As per Section 9.4.2 of ISO 19901-8:2023 with the following addition:

Large diameter rotary core samplers in particular may be applicable to heterogenous deposits containing oversize material (e.g., glacial till).

5.4.5.4.3 Non-Drilling Mode Samplers

As per Section 9.4.3 of ISO 19901-8:2023.

5.4.5.5 Sample Recovery Considerations

As per Section 9.5 of ISO 19901-8:2023.

5.4.5.6 Handling, Transport, and Storage of Samples

As per Section 9.6 of ISO 19901-8:2023.

5.4.5.6.1 General

As per Section 9.6.1 of ISO 19901-8:2023.

5.4.5.6.2 Offshore Sample Handling

As per Section 9.6.2 of ISO 19901-8:2023.

5.4.5.6.3 Offshore Storage

As per Section 9.6.3 of ISO 19901-8:2023.

5.4.5.6.4 Onshore Transport, Handling, and Storage

As per Section 9.6.4 of ISO 19901-8:2023.

5.4.6 Laboratory Testing

5.4.6.1 General

As per Section 10.1 of ISO 19901-8:2023 with the following addition:

During the execution of a geotechnical laboratory test program, tests shall be performed within the framework of recognized standards or codes or other recognized procedures. The standards cited in Annex F of ISO 19901-8:2023 are the recommended ones and are primarily those of ISO and ASTM, where available, although other standards may be used.

If testing requirements are not specifically given for a project, then the standards and procedures described in Annex F of ISO 19901-8:2023 shall apply.

Annex F of ISO 19901-8:2023 provides procedures for conducting the more common laboratory tests, with a primary focus on laboratory testing of saturated soils.

Requirements presented in Clause 10 and Annex F of ISO 19901-8:2023 are primarily for testing conventional soils such as siliceous sands and clays. Samples selected for a particular laboratory test should match the intended scope of that test. Consideration should be given to alternative and supplementary requirements when performing marine soil investigations in unconventional soils such as micaceous soil, carbonate soil, glauconitic soil, silt, sensitive clay, boulder clay, and contaminated soils (see Annex A, Table A.3 of ISO 19901-8:2023 for a list of more unconventional soils). Section 8.2.4. presents a discussion of unconventional soils.

ISO 19901-8:2023 does not cover details of laboratory testing of rock; however, Annex F, Subclause F.13 provides references to other standards containing guidance for classification and laboratory testing of rock materials.

The applicability of measured data from tests that require the use of intact samples is significantly influenced by sample quality. It is therefore important to evaluate sample quality whenever possible.

5.4.6.2 Presentation of Laboratory Test Results

As per Section 10.2 of ISO 19901-8:2023.

5.4.6.3 Instrumentation, Calibration, and Data Acquisition

As per Section 10.3 of ISO 19901-8:2023.

5.4.6.4 Preparation of Soil Specimens for Testing

As per Section 10.4 of ISO 19901-8:2023.

5.4.6.5 Evaluation of Intact Sample Quality

As per Section 10.5 of ISO 19901-8:2023.

5.4.7 Reporting

5.4.7.1 Definition of Reporting Requirements

The intention of reporting is to provide a clear and concise summary of the geotechnical investigations performed in support of developing the proposed offshore wind farm. This shall include detailed summaries of the work performed, factual reporting of measurements, and interpretation of the geotechnical parameters to be used for design. As such, geotechnical investigation reports should be characterized as either Factual or Interpretative, as further discussed in Sections 11.2 and 11.3 of ISO 19901-8:2023 and Sections 5.4.7.2 and 5.4.7.3 below.

Guidance Note:
Due to the large size and development timeline of offshore wind farms, several geotechnical investigation reports will be developed, often by several different companies. As a result, there is often inconsistency in the naming of investigation points (e.g., borehole and CPT locations) between the factual and interpretative reports. Although the reason for changes in naming cannot always be avoided, special attention should be made by the project to ensure either consistent naming schemes between reports or that each report includes a clear conversion table indicating the old and new naming scheme.

For a detailed discussion on the purpose, content, and examples of reporting of Geotechnical Investigations, please refer to Section 11.1 of ISO 19901-8:2023 as well as the informative guidance provided in Annex G of that standard.

The scope and extent of reporting and the reporting structure should be defined as part of the project-specific requirements. If the reporting format is not given in project-specific documents, then the format presented in ISO 199901-8:2014 (Table G.1 in Annex G) applies.

5.4.7.2 Presentation of Field Operations and Measured and Derived Geotechnical Parameters

The intention of this section is to outline the requirements for the presentation of data and results gathered from the various geotechnical investigations performed. As previously discussed, this can be broadly characterized as the reporting of Factual data, although some interpretation is often required to develop these reports.

Guidance Note:
The data described in this section will still require additional interpretation to establish appropriate design values. Establishing these design values requires a clear understanding of the foundation type, loading, and methods for geotechnical analysis required to size the chosen foundation. As a result, it is often too early in the development timeline for those performing the geotechnical investigations.

With respect to projects in which third-party certification is being performed, please note most certification schemes require the laboratories performing the testing work to be accredited. The most widely recognized accreditation standard is the ISO/IEC 17025:2017—General Requirements for the Competence of Calibration and Testing Laboratories. However, because there are numerous standards for which laboratories can be accredited, alternative accredited laboratories or even those without accreditation can be used if sufficient documentation that meets the intention of ISO/IEC 17025:2017 can be provided. For additional guidance, please refer to DNVGL SE 0073 (DNV GL, 2018a).

For a detailed discussion on the purpose, content, and examples for the presentation of data and results, please refer to Section 11.2 of ISO 19901-8:2023.

5.4.7.3 Data Interpretation and Evaluation of Representative Geotechnical Parameters

The intention of this section is to outline the requirements for the interpretation and evaluation of geotechnical data for the establishment of geotechnical design parameters. As previously discussed, this can be broadly characterized as the Interpretative report, which is responsible for clearly reporting the geotechnical design parameters. Due to the complicated relationship between representative parameter selection and design methodology as they relate to the overall target safety level, it is important that the approach utilized is clearly documented.

Guidance Note:
Representative parameter selection should be based on the type of foundation, design situation, and knowledge of the mobilized soil volume. As a result, it is likely that several representative soil profiles will be required to fulfill all the various analyses necessary to design an offshore wind turbine foundation. For example, the choice of a representative parameter for use in a pile drivability analysis will not be the same as one chosen for an axial pile analysis.

Good engineering judgement should always be exercised, but the use of statistical methods may also be appropriate as they can provide a consistent approach to interpretation of factual data. However, caution shall always be exercised when using global data sets for site-specific design. For additional guidance, please refer to DNVGL-ST-0126 (DNV GL, 2021).

For a detailed discussion on the purpose, content, and examples for the presentation of data interpretation and evaluation of representative geotechnical parameters, please refer to Section 11.3 of ISO 19901-8:2023.

5.5 Metadata and Storage Requirements

Metadata for each spatial feature shall be included in accordance with the FGDC Content Standard for Digital Geospatial Metadata (CSDGM) or ISO 19115-1:2014 metadata format, and shall include at least

- Description of the data set
- Contractor and project details
- Dates of data acquisition
- Horizontal and vertical control details
- Spatial extent
- Information on horizontal and vertical accuracy and resolution of the survey data

All data and associated interpretation, charting and reporting are recommended to be stored by the acquisition contractor and the developer for a minimum of five years following the completion of the campaign and full delivery to all stakeholders. Data should be stored digitally and should be adequately mirrored in case emergency recovery is required.

Guidance Note:
Detailed metadata standards have already been developed and are in use globally. An example of a detailed metadata standard is that given in MEDIN (2019). Offshore wind developers may also have specific requirements and project standards in excess of any statutory requirements.

6 Geohazards and Anthropogenic Hazards and Constraints

6.1 Geohazards

6.1.1 Approach

Development of offshore wind farms may occur in a diverse range of geological and physical oceanographic environments. Projects should develop an understanding of the existing site conditions through the ground model development process described in Section 7. An understanding of site conditions, geohazards and anthropogenic constraints should begin formulation prior to conducting surveys and this information should be used to inform and plan geophysical and geotechnical investigations.

Due to the challenges in the marine environment and associated development risks, it is necessary to carry-out a geohazard assessment. The objectives of geohazard assessments are to characterize the location, severity, and frequency of geohazard events such that potential adverse interactions with development infrastructure can be minimized to an acceptable risk level. Multi-disciplinary site investigations are essential for characterizing geohazard processes, as developers need to consider geohazard events with different probabilities of occurrence and related consequences, ranging from: (i) frequent, high probability geohazard events that can be expected to occur, but with consequences that can be managed through appropriate engineering; to (ii) infrequent, low probability geohazard events that cannot be easily avoided, but whose consequences are severe. The process of understanding and characterizing these different risks inevitably involves careful collection, integration, interrogation, and interpretation of geological, geophysical, and geotechnical datasets.

Guidance Note:
The following references provide descriptions of geologic conditions and potential geohazards relative to future offshore wind developments. General geologic descriptions for the US East Coast are also included in OCS Study BOEM 2017-049. For more information on Earthquake, Landslide, Tsunami, and Geohazards on the U.S. Offshore Pacific Wind Farms see https://www.boem.gov/sites/default/files/documents/renewable-energy/Selected-BOEM-Research-Renewable-CA.pdf.pdf and U.S.G.S. National Seismic Hazard Maps.

6.1.2 Overview

Geoscience data should be collected to develop information that can be used to: characterize the seafloor processes and sub-seafloor geological conditions in the area of interest; assess the relevant geological hazards; and document the nature of geological or engineering constraints that might affect the development. Some of the technical issues that should be addressed during the integrated assessment of data include, but are not limited to:

Seafloor Geomorphology and Site Stratigraphy
  - Nature and characteristics of seafloor geomorphic features
  - Vertical and lateral variability of stratigraphic units
  - Characteristics and variability of geotechnical properties associated with key stratigraphic units
- Geohazards
  - Earthquake strong ground shaking
  - Surface fault rupture and shallow faults
  - Slope instability and mass transport
  - Liquefaction and lateral spread/flow failures
  - Shallow gas or gas seeps
  - Buried channels
  - Boulders
  - Tsunami
  - Mobile seafloors
  - Scour
  - Inclined seafloors, scarps, or areas with irregular seafloor relief
  - Soft and/or organic soil conditions
  - Hard ground, bedrock, and debris fields
  - Karst and dissolution features
  - Ice-related hazards (e.g., ice loading, seabed gouge, etc.)
  - Creep, subsidence, or other forms of ground deformation.

6.1.3 Surface and Sub-Seafloor Variability

The integrated geoscience assessment should develop information for preparing a detailed ground model that represents a three-dimensional interpretation of the geological conditions within project's defined area of interest (see Section 7). This study should integrate geophysical data from seismic reflection or sub-bottom profiler surveys with geological boring data and geotechnical measurements. The integrated geophysical, geological, and geotechnical databases can be used to depict the distribution and variability of the seafloor and sub-bottom conditions. Particular care should be taken when interpreting seismic data based on borehole description. Appropriate time to depth laws shall be used to accurately correlate seismic horizons with geological markers from the boreholes. In particular, in situ velocity measurements at several locations across the wind farm site should be used in geologically complex areas.

Documenting the variability of stratigraphic conditions is important, as geotechnical strength and density parameters may vary as stratigraphic conditions change across the site area. Emphasis should be given to defining the potential locations of unusual or differential conditions, their nature and engineering implications. Understanding the geological origins of the variable conditions (e.g., paleovalleys, back-bay features; glaciation, faulting etc.) will typically improve a continuous ground model based on discrete data sets as well as better define decisions regarding installation and design of wind farm infrastructure.

6.1.4 Scour and Seafloor Mobility

The stability of the seafloor shall be assessed. If predicted levels of seafloor variation are estimated to adversely affect the structural integrity of the foundation beyond design tolerances, then scour protection measures are required. These variations in the seafloor level can be due to structure-induced scour or the morphological evolution of the seafloor itself. Structure induced scour is discussed in Section 8.5.

The interrelationship between the bottom currents produced by various oceanographic conditions, and the seafloor and seafloor sediments, produces the seafloor geomorphology and creates the potential for erosion, transport, and redeposition of the seafloor sediments. That interrelationship varies both spatially and temporally. Subtle changes in the seafloor have been documented during minor storms, while large storms can produce significant changes in the seafloor due to erosion, transport, and redeposition of the seafloor sediments.

The dynamic equilibrium among the ocean currents, seafloor conditions, and seafloor sediments is complex. Small changes in any of the conditions can affect the equilibrium. Moreover, certain seafloor sediment types (e.g., sand) can respond to hydrodynamic changes in time scales of an hour or less. Conversely, for cohesive sediments significant topographic changes may require months or years to occur. The introduction of offshore wind farm (OWF) structures is generally not considered to alter the natural rates and magnitudes of seafloor changes apart from localized scour.

Seafloor mobility refers to overall seafloor movement due to the migration of sand waves, sand banks, ridges and shoals which would occur in the absence of a structure and may include scour or deposition. Such movements can result in the lowering or rising of the seafloor. Seafloor mobility and scour can result in removal of vertical and lateral support for foundations, cause undesirable settlements and displacements of shallow foundations, overstressing of foundation elements and change the dynamic properties of the wind turbine structure.

Seafloor mobility is most commonly assessed on the basis of repeat bathymetric surveys to determine the growth rate, migration rate and migration direction of large-scale bedforms. In the absence of repeat surveys, migration direction may be inferred from bedform morphology with the maximum range of seafloor variation taken as the maximum amplitude of bedforms adjacent to the foundation. Survey frequency recommendations for in-service life surveys are addressed in Section5.3.1.8.

Seafloor mobility shall be considered at all structure and cable locations. Potential long-term general seafloor level changes due to the migration of sand waves banks, ridges and shoals shall be considered for all foundation types. Where scour is a possibility, it shall be taken into account in design and/or its mitigation/avoidance shall be considered. Due to the complex range of issues to be addressed, scour hazard assessment and mitigation may require input from several disciplines. In addition to physical forcing data (e.g., currents and wave conditions), an accurate analysis of scour requires detailed knowledge of the bathymetry and bed characteristics (e.g., sediment grain size distribution) in the vicinity of the planned structures.

Guidance Note:
Discussions of sediment transport and scour are available in Sumer and Fredsøe (2002), Richardson and Davis (1977), Whitehouse et al (2011), Whitehouse (1998), Petersen et al. (2015), Soulsby (1998), and USACE (2008). Seafloor scour considerations for offshore wind development on the Atlantic OCS are provided in TAP-656, 2011 available at https://www.bsee.gov/research-record/tap-656-seabed-scour-considerations.

6.1.5 Boulders

Boulder risk should be evaluated according to the expectations for installation equipment and clearance methodologies. Size and density thresholds for boulder detection should be tailored to the sensitivity of the intended equipment and techniques. Risk presented from both surface and sub-surface boulders should be analyzed. If required, surveys for boulder mapping should be incorporated into Engineering and Site Characterization surveys (Section 5.3.1.3).

While mapping all boulders over the defined threshold can provide a thorough mitigation of risk, consideration should also be given to applying a statistical analysis based on representative datasets. In wind farm areas characterized by glacial moraines and tills, where boulders of any size are mapped, a full-size distribution of boulders and cobbles should be expected. In these areas, selecting equipment on the assumption of high boulder risk may be sufficient mitigation, negating the need for detailed boulder mapping.

Guidance Note:
Guidance on geophysical surveys for boulders supporting cable installations and associated risk assessments is provided in Carbon Trust (2020). Whilst the geophysical data specifications and references to data densities required for mapping boulders of particular size detailed in the Carbon Trust document are not endorsed by this recommended practice, the document provides a useful background.

6.1.6 Seismicity and Earthquake Effects

6.1.6.1 General

Actions and action effects due to earthquake events shall be considered in the structural and geotechnical design of the wind farm structures in seismically active areas.

The design of offshore wind farm assets such as offshore wind turbine (OWT), offshore sub-stations (OSS), and other bottom fixed structures (such as meteorological towers) should be in accordance with the guidelines of the American Petroleum Institute API RP 2EQ. In seismically active areas, the effects of seismic events on subsea equipment such as export cables should be addressed by special studies.

API RP 2EQ focuses on the characterization of earthquake ground motions and defining criteria for offshore structures. However, other seismic hazards (Section 6.1) shall also be considered in the design. The potential for such hazards to impact the offshore wind farm assets should be considered during the Desktop Studies, and where risks are identified, should be addressed by special studies.

Per the requirements of API RP 2EQ, areas are considered seismically active based on the frequency and magnitudes of previous earthquakes. Maps provided in Annex B of API RP 2EQ provide indicative seismic accelerations for Offshore North America, and the document provides guidance defining when seismic studies are required. The understanding of offshore geologic conditions continues to evolve. Many areas where wind farms are planned may not have been extensively studied previously and the site surveys may provide additional information about the seismic setting. The tectonic setting and seismic activity of the area should be considered in the project Desktop Studies to confirm the appropriateness of the Site Seismic Zone assessment. The potential presence of tectonic features, such as active faults, should be considered in scoping and planning geophysical and geotechnical surveys, and the collected data should be evaluated for the presence of such tectonic features. Where elevated risks are identified, site-specific studies may be warranted to quantify earthquake ground motions.

> **Guidance Note:**
> For inland lakes and waterways in the conterminous United States, Hawaii, Alaska or other territories not covered by the maps in Annex B of API RP 2EQ, the USGS 975-year return period Uniform Hazard Maps (https://earthquake.usgs.gov/hazards/interactive/index.php) for oscillator periods of 0.2- and 1-second can be used.

6.1.6.2 Surface Fault Rupture Hazards

Fault rupture occurs when the stress across a fault zone exceeds the static frictional strength of the fault plane. As the two adjacent blocks slip and relieve the stress across the fault plane, energy radiates out from the fault zone as seismic waves that produce earthquake strong ground shaking (described below). The sudden displacement across the fault plane also has the potential to rupture the ground surface, which can severely damage infrastructure elements if located across the fault zone.

Tectonic geomorphological, geophysical, and geological investigations should be carried out to identify the locations of potentially active fault zones. Where faults may affect infrastructure elements within a development area, data should be collected to document the fault location, style of deformation, displacement, and slip rate. In offshore environments, this information is typically developed through a combination of detailed seafloor geomorphological mapping, high-resolution seismic reflection and sub-bottom profiling, and sample collection and dating.

6.1.6.3 Earthquake Ground Motions

Earthquake ground motions shall be assessed in accordance with the requirements of API RP 2EQ and the exposure categories specified in Section 5.3 of the ACP OCRP-1-2022 document. Additional guidance for OWT and OSS structures is given below. Depending on the Seismic Risk Category (SRC) of API RP 2EQ, either simplified action procedures can be used, or detailed action procedures shall be followed. The simplified action procedures use API (or local) maps to define bedrock or stiff soil acceleration response spectra with modifications for local site conditions based on site classification. The detailed action procedures involve Probabilistic Seismic Hazard Analyses (PSHA) and site response analyses to define near surface earthquake ground motions.

6.1.6.3.1 Offshore Wind Turbine (OWT)

For the assessment of earthquake ground motions, the abnormal level earthquake (ALE) of API RP 2EQ should be considered as the ALS event, a 475-year return period should be considered for the ULS event, and a 95-year return period may be considered for the SLS event. The need for ALS assessments is not normally required for OWT support structures except in specific cases where an increased safety demand is deemed necessary.

> **Guidance Note:**
> Section 5.6.2 of the ACP OCRP-1-2022 document indicates that the design of OWT should be in conjunction with IEC 61400-3-1, where earthquake load cases are defined in IEC 61400-1. IEC 61400-1 specifies a single 475-year return period earthquake with a partial load factor of 1.0 and partial material factor for steel of 1.0. Although a limit state designation is not clearly indicated, it appears from the designation of design load cases that the 475-year return period is associated with a ULS. Additional details for the ULS and SLS are provided in DNV-RP-0585.
>
> Section 2.2 of DNV-RP-0585 provides examples of cases where an increased safety demand warrants an ALS assessment and associated performance requirements.

Section 5.3 of the ACP OCRP-1-2022 document states that the exposure category as defined in API RP 2A-WSD or API 2A-LRFD may be designated as L-2 for an OWT. Since API RP 2EQ does not define target Annual Probabilities of Failure (P_f) and Seismic Risk Category (SRC) for

L2 exposure category structures, these can be assessed in accordance with the guidance for L2 exposure level in ISO 19901-2:2022.

> **Guidance Note:**
> In assessing earthquake ground motions in accordance with API RP 2EQ, the exposure category is associated with a P_f and is used together with the Site Seismic Zone to establish the SRC. API RP 2EQ is a modified version of ISO-19901-2

6.1.6.3.2 Offshore Substation (OSS)

For the assessment of earthquake ground motions, the abnormal level earthquake (ALE) and Extreme level earthquake (ELE) of API RP 2EQ should be considered as the Accidental Load State (ALS) and Ultimate Limit State (ULS) events, respectively.

> **Guidance Note:**
> Section 5.7.2 of the ACP OCRP-1-2022 indicates that API RP 2A -WSD or API RP 2A-LRFD can be regarded as the top-level code for the design of OSS. For seismic design in accordance with API recommended practices a two-level seismic design is adopted in which the structure is designed to an ultimate limit state (ULS) for strength and stiffness and then checked to the abnormal or accidental limit state (ALS) to ensure that it meets reserve strength and energy dissipation requirements.

Section 5.3 of the ACP OCRP-1-2022 document states that the exposure category of OSS structures should be L1.

6.1.6.4 Local Site Effects

Published uniform hazard maps and PSHA studies typically define ground motions at bedrock or stiff soil conditions. As the earthquake ground motions propagate up the soil column they are modified by the local geotechnical conditions. These changes affect both the amplitude and frequency content of the ground motions. Many offshore sites consist of a surface layer of soft soils overlying the stiffer soils and/or bedrock. In particular, when soft soils are present there is a potential for amplification of the long-period components of shaking, while de-amplification of some spectral content may also occur under strong shaking.

Local Site Response Analyses should be in accordance with Section 8.5 of API RP 2EQ.

A range of equivalent linear and nonlinear techniques are available for site response analyses. Equivalent linear techniques are typically most appropriate for stiffer sites and lower shaking levels, whereas nonlinear techniques should be used for relatively soft sites with higher shaking levels. Effective stress techniques that can model the development and dissipation of pore pressures, triggering of liquefaction and modelling of post-liquefaction deformations should be considered for sites with potential liquefaction risk. Two-dimensional (2D) models may be required in cases of significant topographic or sub-seafloor variability, or to capture potential slope instability.

Guidance on the dynamic parameters required for site response analyses is in Section 8.2.3.

Local site effects may be directly accounted for in PSHA for soil sites that classify as Site Class C or D per Table 5 of API RP 2EQ. Nonlinear site response analyses techniques should be adopted for sites that classify as Site Class E and shall be adopted for sites that classify as Site Class F.

6.1.6.5 Liquefaction

Soil liquefaction can generally occur in deposits that are very loose to medium dense, saturated, and have cohesionless or low plasticity properties. These types of deposits are considered to be susceptible to liquefaction given that the opportunity (strong ground shaking) occurs. Deposits susceptible to liquefaction are known to occur in a relatively narrow range of depositional environments and sedimentary ages. Liquefiable deposits are generally (but not always) Holocene-age, with the highest susceptibility being deposits that are less than 3,000 years old.

As susceptible soils are shaken, their tendency to contract and compress may lead to the development of positive pore pressures. If the seismic shaking is strong and long enough, the build-up in pore pressure can produce a significant loss of shear strength. Liquefaction is said to occur when the excess pore pressure equals the initial effective stress in the soil. After the onset of liquefaction, ground distress may occur (e.g., sand boils, settlement, lurching, and lateral deformation). The occurrence of liquefaction also alters earthquake ground motions.

The potential for liquefaction and associated effects shall be evaluated in seismically active areas. Liquefaction resistance is most commonly determined from in-situ testing (such as cone penetration test (CPT) data) and/or from stress-controlled cyclic simple shear testing. The assessment of soil liquefaction triggering, and post-liquefaction deformations may require site-specific dynamic laboratory tests and such tests should be done when dealing with unusual soils (e.g., carbonate or micaceous soils, low plasticity silts) that are not well represented in the databases used to develop empirical methods. Cyclic demands for the evaluation of liquefaction triggering potential can be estimated using simplified techniques. Where a liquefaction risk is identified, or the factors of safety against liquefaction are marginal, cyclic demands should be defined by total stress dynamic site response analyses or an effective stress site response analysis should be conducted.

> **Guidance Note:**
> Guidance on the phenomena of soil liquefaction is available in Idriss and Boulanger (2008). Approaches for assessing liquefaction triggering and post-liquefaction deformations of siliceous soils from in-situ test results are available in Boulanger and Idriss (2016) and Tasiopoulou et al (2020), respectively. A commentary on liquefaction impacts to Wind Turbine support structures is provided in DNV RP-0585.

6.1.6.6 **Other Effects**

Guidance on Earthquake-induced hazards such as slope instability, fault movement, tsunamis, mud volcanoes, water column shock waves are provided in Annex A.5 of API RP 2EQ.

6.1.7 Slope Instability

As US energy demand continues to grow, offshore energy projects are being planned and developed along the continental shelves of the US, as well as within the Great Lakes. One of the dominant drivers of geohazard risk in the marine environment will be slope failures and associated mass transport events. These types of failures can be amongst the largest earth movements in the world (Moore et al., 1989; Hampton et al., 1996; Masson et al., 2006) involving thousands of cubic kilometers of material, but it is not necessarily these large catastrophic failures that pose the greatest risk to marine infrastructure elements. Relatively frequent small failures also can impose sufficient loads on turbine and transmission systems to jeopardize system integrity. As such, developers of offshore wind farms should evaluate (as a minimum qualitatively during the desktop study stage) the potential for slope failure to impact the specified project development area so that the geohazard risk to the development can be identified.

Where warranted, investigations that are appropriate to the level and type or risk should be carried out so that the slope processes, geomechanical properties, and potential triggering mechanisms that impact the development area are understood. The objectives of these investigations are to:

- develop a ground model that depicts vertical and lateral sediment variability (Section 7);
- define the sliding surface slope angle using seafloor and sub-bottom data;
- map existing slides in the ground model area to provide information on expected depth of the sliding surface;
- estimate geotechnical soil strength and density properties; and,
- estimate design ground motion parameters from probabilistic seismic hazard analysis (see Section 6.1.5) results.

The information from these investigations should be used to assess the stability of the seafloor in the development area. Two analytical approaches for assessing slope stability include the

Limit Equilibrium and Shear Band Propagation methods. Although the Limit Equilibrium approach has been used to estimate slope stability for many years, the newer Shear Band Propagation approach should be considered as it allows for improved estimates of the annual probability of failure for a variety of observed landslide mechanisms, such as slab, spreading, plowing, and run-out failures.

> **Guidance Note**
> Information on technical approaches to probabilistic slope failure analysis for offshore developments can be reviewed in Puzrin et al., 2017 and publications referenced therein.

In cases where landslides represent one of the key geohazard risks, a quantitative slope stability assessment should be performed, and the results included in an overall geohazard Quantitative Risk Analysis (QRA) (see section 3.6). The QRA should support development of geohazard avoidance, mitigation, or acceptance strategies. Example outputs that can be used to support development of these strategies are maps illustrating the factor of safety and annual probability of failure.

6.2 Anthropogenic Hazards

6.2.1 Marine Archaeologic Resource Assessments

Accumulating the information compiled from desktop study and geophysical and geotechnical survey, an integrated Marine Archaeological Resource Assessment should be compiled to discuss the archaeological potential of the wind farm area and cable corridor, and to consider the risks posed by the proposed development activities.

Where exclusion zones are recommended to mitigate risk to items of potential cultural heritage, these should be tailored to the object itself, the environmental and geological conditions at the location, the local seafloor dynamics, and the proposed interactions with the seafloor at this location; the extent, duration and character of the seafloor interactions should be considered. Depending on the conclusions of such an analysis, a temporary exclusion zone may be put in place for the short duration of an interaction, and a larger permanent exclusion zone established for the longer life of the project.

For archaeological finds or areas of potential archaeological interest where the establishment of exclusion zones is not seen to be the most appropriate course of action, mitigation by documentation should be considered. In this case, the type of investigation, level of data quality, resolution, and extent, should be advised by a Qualified Marine Archaeologist (QMA). Dissemination and usage of this documentation should also be considered to ensure value is maximized.

6.2.2 MEC/UXO Risk Management Process Overview

In assessing the MEC risk to offshore projects, typically a Semi Quantitative Risk Assessment (SQRA) process is widely considered as best practice and in line with existing industry guidance (CIRIA Report C754, 2015; BOEM Research Study OCS 2017-063, Carbon Trust, 2020). The risk that MEC poses to any Project related activity is the product of three key elements:

The probability of encountering an item
If that encounter happens, the probability of the item detonating, and
If the item detonates, the severity of the consequence to vulnerable receptors (people, marine life, vessels, and equipment).

MEC risk is generally considered a low probability, but a high consequence event and it is the latter factor that usually dictates the overarching risk score. The potential consequence of a MEC detonation is by far the dominant factor in the calculation. If sufficient information is available at the time of production, then this document will remain valid for the life of the Project. If not, this may require revisiting as the Project moves through its life cycle.

Guidance on planning MEC/UXO geophysical surveys is provided in Section 5.3.1.6. Following the geophysical survey, potential MEC constraints are likely to be identified from the datasets. These potential MEC items are selected from the wider anomaly listing based on Project specific risk assessment and the smallest hazard item to the proposed engineering activity. These constraints should be treated as hazards and avoided unless they are investigated and confirmed as not as related to MEC. Any potential MEC geophysical anomalies shall be avoided by, at a minimum, a suitably safe distance for any intrusive seafloor interactions, including any seabed preparation activities (e.g., pre-ley grapnel run) and geotechnical drilling, logging, or other sampling methods. This can be achieved through rerouting or micro-siting of seafloor interactions. In accordance with the ALARP principle (see Section 5.3.1.6), activities or installation could then proceed with a *de minimis* risk of encountering MEC. However, the safety exclusion zones around the geophysical contacts should be respected. Unless these contacts are investigated and confirmed as not MEC related, they should be considered a potential hazard.

Should the avoidance exclusion zones not be tolerable or manageable within the planned operations, the pMEC targets which cannot reasonably be avoided should be inspected by ROV or diver to confirm their identity. If, after inspection, a pMEC target is confirmed as non-MEC e.g., scrap metal, the exclusion zones and pMEC are removed from the listing and are no longer a constraint to operations. Should a MEC be identified during inspection, prior to any explosive ordnance disposal operations open communications and notices with all relevant stakeholders should take place to agree the most appropriate course of action.

> **Guidance Note:**
> Guidance on such communication and course of action is described in the National Guidance for Responding to Munitions and Explosive of Concern in Federal Waters (see 88 FR 58235; Docket No. DOT-OST-2023-0117). This document is proposed but not finalized as of the publication of this standard, so it is provided not as a formal reference but for information purposes.

Documentation or certification of reducing MEC/UXO risk to ALARP should show that the process has not eliminated all MEC risks on the Project site to ‘zero’, as to do so would be impracticable and prohibitively expensive, but that a transparent and reasonable process has been adopted. While a minor residual risk may remain, it is considered to be at a *de minimis* level and thus reasonable to be carried by those parties involved. A certificate showing achievement of ALARP should be a comprehensive document that is unique to any particular location. Such certificates should be issued by a suitable organization that holds specific external Professional Indemnity Insurance for this certification and should be available for auditing by relevant external bodies.

Installation and O&M contractors should adopt a procedural MEC residual risk management and mitigation actions to conform to best practice including:

- Key MEC Policy Documentation
- MEC Training
- Delivery of MEC Constraints
- Communication with an On-call MEC/UXO Specialist.

6.3 Risk Register

This document, and the referenced documents herein, provide guidance as to how marine surveys may be planned for offshore wind developments. Marine survey and seafloor risk management occurs over the course of the development stage of the project, and beyond into operations. A project risk register provides a means of establishing an initial understanding of seafloor and sub-seafloor risk at the desktop study stage and then recording the evolution of risk understanding as successive data acquisition campaigns improve knowledge and understanding of the site conditions. The project developer is the owner of the risk register, but it may be communicated to project stakeholders and the supply chain, who may also assist in populating the register as part of their project engagement.

The project risk register should be a ‘live’ document that is revised and developed throughout the project lifespan. As such, it serves to provide:

b) A retrospective view of the evolution of project risk, and the steps taken to mitigate risk

c) A live and current assessment of project seafloor risk

d) A means of targeting and planning future risk mitigation activities through, for example, data acquisition, or design.

The nature of documented risk changes as the project develops; at initial stages the risk will be broad and not specific to a given type of foundation or burial tool for subsea cables. As the project develops and design decisions are made, the risk register is updated to reflect increasingly specific risks which are a function of both the seafloor conditions, but also the project design decisions being made.

The risk register should document:

e) The nature of the hazard

f) The frequency and consequence of the hazard, giving an initial risk score

g) The nature of risk quantification or mitigation efforts

h) The resulting frequency and consequence of the hazard, after mitigation, giving a project residual risk score.

> **Guidance Note:**
> Example Methodologies for the development of risk registers and their evolution as ground models are developed in a staged approach to offshore surveys is provided in BOEM Publication No. 2018-054 -Data Gathering Process: Geotechnical Departures for Offshore Wind Energy.

7 Ground Modelling

The ground conditions at an offshore wind farm site represent one of the largest uncertainties for the feasibility of a project. A practical tool for managing risks related to the ground conditions and potential geohazards described in Section 6, is a ground model, where geological, geophysical and geotechnical data are integrated to define and present an overview of the geological conditions at the site, i.e. geological formations present, their extent and spatial variability across the wind farm area, and their mechanical and engineering behavior. Each offshore wind project should have a ground model. Ground models may inform the project risk register.

The purpose of developing a ground model is to:

- Reduce risk related to ground conditions, e.g., identify, locate, and manage the potential geological and geotechnical hazards identified in the project risk register,
- Inform the feasibility assessment of the foundation solutions being evaluated for the various facilities at site,
- Inform the feasibility of installation techniques for cables, turbine foundations, substation foundations, substructures, and superstructures (including turbine components), e.g., address safety related to jack-up works,
- Inform the preliminary facility design for project budgeting and planning purposes and later inform the detailed foundation design,
- Inform the design of cables for the project
- Assist optimal scoping of ground investigations, when the ground investigations and the ground model are developed in phases related to the maturation scheme of the wind farm development project.
- Inform environmental and archaeological studies, e.g., mapping of benthic habitats, palaeo landform assessment and identification of archaeological resources.
- Inform on existing infrastructure and use of the seabed.

The ground model is usually constructed through an iterative process spanning most project maturation phases from the earliest reconnaissance studies to maintenance surveys performed during the operational phase. Each ground model is developed for site-specific conditions and project specific requirements for wind farm layout and individual facility designs. The ground model thus will present itself with varying degrees of complexity and format. For practical reasons, it is sometimes beneficial to separate the ground model for the turbine array area from the ground model for the export and inter-array cable routes.

If a model is primarily based on bathymetric data, it may take the form of a geomorphological model, whereas when further supported by geological data, it becomes a geological model. A geological model can include structural, stratigraphic and/or potential lithological components, typically interpreted from seismic reflection data. Once the geophysical and geological data are integrated and correlated with geotechnical data (invasive and “ground truthing” investigations), the model becomes a ground model. If classification and engineering properties of the geological formations are included, the model may become a geotechnical model. It is recognized that the term “ground model” is not always used consistently in the industry, and other terms may be used interchangeably.

For the ground model to adequately inform and assist management of the geological and geotechnical risks across a wind farm site during the project development, it should:

- identify geological formations (soil and rock units) and their distribution and spatial variability across the site,
- characterize the geological context of the site,
- present an overview of seafloor and sub-seafloor depositional environment at regional and local scales
- present an overview of geological and geotechnical hazards identified on site, including the potential for present and future natural and anthropogenic geohazards, and

– cover all areas of potential seabed disturbance in terms of horizontal coverage and depth of foundation embedment across the site, including cables routes.

Usually, a high-level approach for the staged development of a ground model initially involves a preliminary desk study of the regional and local geological conditions targeting the whole development area of interest. Then one or more early geophysical reconnaissance surveys are scoped with the purpose of collecting data used to build an understanding of the geological framework across the area of interest. The geological framework will include preliminary assessment of geological formations and their stratigraphic extent and variability across the area and preliminary assessment of geohazards. Besides informing for permitting and fulfilment of lease requirements, the reconnaissance geophysical survey will also inform subsequent reconnaissance geotechnical investigation(s). The early geotechnical investigation usually consists of several sampling boreholes and potentially cone penetration tests, vibro-coring, grab sampling and other supporting in situ testing or sampling conducted at strategic positions for validation and “ground truthing” of the geological context at site as derived from the seismic surveys. All relevant soil units are sampled for geotechnical laboratory testing to determine classification, mechanical and engineering properties. When the wind farm layout as well as turbine and facility foundation solutions are known, geophysical and geotechnical investigations are then carried out for informing the detailed design at turbine or facility specific positions.

The outcome of the ground modelling often takes the form of a geospatial model, a database and a corresponding report(s) that includes site investigation details, the raw data obtained and its quality, and the data processing and results.

The ground model can be used for facilitating micro-siting of the wind farm (e.g., cable routing and layout of the substation and turbine array for avoiding buried channels or other identified geohazards).

In case of late revisions to the project layout during permitting or design phases, the ground model may provide adequate confidence in the ground conditions present at the new position(s). In case the ground model is not able to provide adequate information or level of confidence to satisfy the technical objectives at the relocation position, additional site investigation may be required, and/or sensitivity studies investigating conservative design scenarios may be required.

The ground model may also be used for deriving statistical ground characteristics, whether related to variation of soil properties or the occurrence of geohazards.

The preparation of a ground model requires a multi-disciplinary team involving a close collaboration between geologists, geophysicists, geotechnical engineers, and facility designers, and both the site investigation contractors and the wind farm developer. It should be noted that no engineering properties for foundation design should be defined for the ground conditions based on, for example, seismic inversion techniques, without consulting a qualified geotechnical engineer.

Since ground models are constructed from input data from various sources (studies and survey), data integration can be cumbersome due to the possible range of data formats. To ease data integration, using consistent data formats throughout the project is recommended, such as the AGS format for geotechnical data, SEG-Y data for seismic data, and GIS-compatible formats for the stratigraphic layer model (elevation, depth below seabed, and thickness for each unit) and delineating locations of geohazards.

Additional information and details related to ground modelling is provided by the SUT OSIG guidelines (2022).

8 Foundations

8.1 Design Principles and Safety Concept

8.1.1 Introduction

This section describes the design principles and safety concept to be applied for the design and installation of foundations for offshore wind farm assets, namely the wind turbine generator (WTG) and the offshore substation (OSS). The design principle is a clearly defined and consistent approach for the design of foundations. In its simplest form, it is a systematic means in which to quantify relevant loads and to demonstrate by calculation that the design foundation has sufficient resistance. Typically, this is achieved by choosing a consistent set of design standards which are relevant for the application at hand. and which have an established safety concept. In general, the recommended design principle is the Load and Resistance Factor Design (LRFD) approach, but a Working Stress Design (WSD) approach can also be applied. Reference to both LRFD and WSD standards are broadly included in this document, as well as in ACP OCRP-1-2022.

Care shall always be taken to avoid mixing design principles and standards to avoid unintended reductions in targeted safety levels. Mixing of design standards should only be used in a complimentary fashion, meaning that an overarching standard is selected and adhered to, but an additional standard may be applied so long as it does not lower the target safety level of the overarching standard. If it is necessary to use a combination of mixed design principles or standards, a detailed explanation of how they will be used and what the combined safety level is should be clearly identified and documented as part of the Design Basis and confirmed with all relevant parties involved.

Although this document is written with clear reference to existing standards and those standards inherently define acceptable foundation concepts and design methodologies, novel technologies can be used. The use of novel foundation concepts or design methodologies will require additional efforts, e.g., Technology Qualification (ref. DNVGL-RP-A203) or demonstration by testing and may further warrant the use of probability-based design methods.

For a more detailed discussion on the application of design principles in foundation design, please refer to DNVGL-ST-0126, Sec. 2 and DNVGL-ST-0145, Sec. 4.

8.1.2 Safety Concept

The safety concept may apply either LRFD or WSD principles but shall at a minimum define the target level of safety through the application of safety factors, either partial or lumped. The safety concept may be further defined by safety classes, or exposure categories, which are generally defined by a combined assessment of both the probability and consequence of failure. Refer to Section 5.3 of ACP OCRP-1-2022 for additional discussion on safety classes and exposure categories applied by API RP 2A-WSD or API RP 2A-LRFD, DNV GL-ST-0126, DNVGL-ST-0145 and IEC 61400-3-1.

The application of LRFD principles requires the use of both load and resistance factors, also referred to as partial safety factors, to achieve a target safety level. Generally speaking, loads that can be seen as negatively impacting the stability or structural performance of the foundation are increased by a factor, while the foundation's ability to resist these loads are reduced by a factor. The application of these partial safety factors is further defined by identifying relevant limit states, as described in Section 8.1.4.

8.1.3 Target safety level

A characteristic value shall first be established which is representative of the load variable and resistance variable for each limit state. A characteristic value is a value assigned to a variable with a prescribed probability of not being exceeded by unfavorable values during a reference period.

The characteristic value is the main representative value. Although one characteristic value may be appropriate in some situations, generally speaking each analysis will require careful consideration of the chosen characteristic value. This is especially true in geotechnical engineering where the same geotechnical engineering parameter may be used in several analyses within the same limit state, yet each analysis may warrant a different characteristic value depending on the knowledge of site conditions and the chosen methodology.

A design value is a value derived from the characteristic value after modification by the appropriate partial safety factors and is the value to be used in the design verification procedure.

For a detailed discussion on the derivation of characteristic values and the application of partial safety factors as it pertains to foundation design, refer to DNVGL-ST-0126, DNVGL-ST-0145, ISO 19901-08, and ISO 19900.

8.1.4 Limit States for Foundation Design of OWT and OSS Substructures

A limit state is defined as the point beyond which the structure/foundation no longer satisfies the requirements. The following limit states shall be considered for the geotechnical design of OWT and OSS foundations:

- Ultimate Limit State (ULS): defines the maximum load carrying capacity
- Serviceability Limit State (SLS): defines the applicable tolerance criteria in normal use
- Fatigue Limit State (FLS): defines the possibility of failure resulting from cyclic loading
- Accidental Limit State (ALS): defines required structural integrity during accidental loading

The effects of cyclic loading such as from environmental loads and earthquake loading in geotechnical design, i.e., cyclic degradation of soil strength and stiffness, shall be considered in both ULS and SLS analyses.

Guidance Note:
The extent of the cyclic degradation considered in both limit states may vary. Please refer to DNVGL-ST-0126 Sec. 2.4 for WTG and DNVGL-ST-0145 Sec. 4.3 for OSS for further detailed discussion on the definition of limit states.

With respect to WTGs, a check on the natural frequency analysis (NFA) is required to ensure that the machines are operating within the turbine specifications.

A distinction shall be made between structural analyses and geotechnical analyses in the assessment of limit states and the application of partial safety factors.

Guidance Note:
For example, while it may be appropriate to reduce the soil resistance in a ULS geotechnical capacity check, a similar check on ULS structural capacity may simply warrant unfactored, characteristic soil parameters.

Resistance is defined as the ability of the surrounding sub-seafloor material to provide sufficient strength and stability in the required limit states. The foundation shall consider sub-seafloor material within the mobilized zone, depending on the applicable limit state and failure mechanism. The structural design of the foundation shall also consider soil actions acting directly upon the foundation.

A foundation's ability to provide geotechnical resistance is often defined by the point at which the foundation reaches any of its limit states. Although there are too many to list completely, the following is an indication of such failure modes and is further discussed in DNVGL-ST-0126, Sec. 7.4.:

- Bearing failure
- Lateral or axial pile failure
- Sliding or overturning
- Excessive settlements or displacements.

Loads can be broadly defined as any force acting directly on the foundation as defined by the relevant limit states. Each limit state will have its own corresponding load or set of loads to consider, and the foundation design engineer will need to work closely with the rest of the project designers to ensure that the appropriate load is considered. This is simply due to the different mechanisms, and which loads, particularly cyclic or earthquake loads, lead to failure in structures versus soils.

8.1.5 **Probabilistic Design**

Probabilistic design methods can be used for geotechnical design if sufficient safety of the structure is documented. The exposure categories for offshore wind farm assets are provided in the ACP OCRP-1-2022.

8.1.6 **Safety Factors for Foundation Design**

LRFD partial safety factors, often referred to as a material factor, shall be applied to either the characteristic resistance or to the characteristic material strength for all limit state analyses. LRFD partial safety factors shall be applied as specified in DNVGL-ST-0126 and DNVGL-ST-0145 or calibrated through more advanced analyses. The above references provide further information on the appropriate partial safety factor to use for each limit state. However, it should be noted that not all limit states will require a reduction or increase in the characteristic value as a factor of 1.0 may be required by the limit state.

For WSD, the global safety factors can be taken from API RP 2A-WSD.

8.2 **Soil Characterization**

8.2.1 **Characteristic Soil Properties and Resistance Values**

Characteristic soil properties are used in the determination of the design resistance and shall be defined for the geotechnical design of foundations. Soil strength and deformation properties are considered resistance variables. The selection of a soil characteristic value depends on the foundation analysis type, giving due consideration to the foundation geometry and volume of soil elements for which a characteristic value is being defined.

Characteristic soil properties are generally associated with the target values, such as the lower 5% quantile value, median value, and upper 95% quantile value of the distribution of soil properties. However, engineering judgement should always be used to establish characteristic soil properties.

For foundation capacity analysis, the characteristic soil resistance is generally a low but measurable value with a probability of being favorably exceeded. When local strength governs the design analysis, the characteristic soil resistance may, for example, be defined as the 5% quantile value. For foundation installation analysis, the characteristic soil resistance is generally a high but measurable value with a probability of being favorably exceeded. When local strength governs the design analysis, the characteristic soil resistance may, for example, be defined as the 95% quantile value. Recommendations in DNVGL-RP-C212 (2021) suggest that if the design is governed by the spatially averaged soil resistance over a large volume of soil, the characteristic soil resistance may be defined as the mean value.

> **Guidance Note:**
> Statistical analysis of the characteristic soil parameters will not always provide representative values for design. This is often the case when there is insufficient data to develop a meaningful distribution function. Engineering judgement should always be used to assess the characteristic soil parameters supported by statistical analysis if this is relevant. Statistical analysis may be omitted if the foundation design engineer deems such analysis to provide non-representative characteristic soil parameter values for design.

8.2.2 **Soil-Structure Response for OWT and OSS Foundations**

8.2.2.1 **General**

Soil parameters for foundation design consist of soil classification data, strength parameters for verification of foundation capacity and installation analysis, stiffness and deformation parameters for settlement and displacement analysis as well as for analysis of the static and dynamic soil-structure interaction. Soil engineering parameters required for foundation design and soil-structure interaction analysis include classification, stress-history, strength, stiffness, and hydraulic soil parameters. The typical soil parameters are listed below but the list is not exhaustive. Further details can be found in ISO 19901-8 Annex G.3 and G.4 and DNV GL-ST-0126.

- Soil unit weight;
- Specific gravity of solid particles;
- Water content;
- Void ratio;
- Grain size distribution;
- Carbonate and organic content, as relevant;
- Plastic and liquid limits;
- Maximum and minimum void ratios to determine relative density;
- Preconsolidation stress and over-consolidation ratio;
- Soil permeability;
- Drained friction angles for drained soil conditions;
- Undrained shear strength for undrained soil conditions;
- Initial small-strain soil shear modulus G_0;
- Soil shear modulus G as a function of strain (see Section 8.2.2.2);
- Constrained modulus or compression index;
- Coefficient of consolidation;
- Soil material damping;

8.2.2.2 **Strain-Dependent Stiffness Response**

The soil shear modulus G is a function of engineering strains. The engineering strains will vary according to the loads considered for the different limit states. Hence, the change in shear modulus as a function of strain should be determined.

The initial value of the soil shear modulus G_0 is usually the maximum shear modulus that can be measured at very small soil strains typically associated with dynamic turbine loads in the FLS and ALS limit states. The shear modulus G will degrade from its initial value G_0 as the soil experiences larger strains under operational turbine loads in the SLS and ULS limit states. For large cyclic loads, e.g., wind and wave loads considered for ULS analysis, the shear modulus can further degrade with both strain and number of load cycles. Hence, applicable soil shear modulus values should be investigated and applied for the various limit states FLS, ALS, SLS, and ULS.

8.2.2.3 **Cyclic Soil-Structure Response**

Cyclic actions can lead to possible fatigue effects and soil shear strength and stiffness degradation. Soil shear strength and stiffness degradation due to cyclic loads are usually caused by the gradual increase in pore pressure. The pore pressure build-up is typically accompanied by an increase in cyclic and permanent shear strains for each load cycle, which may lead to reduced shear strength of the soil.

> **Guidance Note:**
> Not all cyclic actions lead to degradation of soil shear strength and stiffness. In certain soil conditions, rapid cyclic loading may have rate effects that lead to an increase in soil shear strength.

The effect of cyclic actions on the strength and stiffness characteristics of the supporting soil shall be investigated and should be considered in offshore geotechnical design of wind turbine and OSS foundations. In the ULS design condition, the effects of cyclic loading on the soil shear modulus should consider applicable load conditions. The effects of cyclic loading on the soil stiffness should be considered in calculations of settlement, horizontal displacement, and rotation of the foundation. The relevant loads under normal operating conditions that may lead to permanent or long-term settlements, displacements, and rotations should be considered. Recommendations on governing load conditions and potential stress histories to be investigated are provided in DNV GL RP C212, Section 10.2.

Strain and/or pore pressure accumulation in soil as cyclic shear stresses are applied may be represented by strain-contour and pore-pressure diagrams. The strain-contour diagram provides the relation between the number of cyclic stresses (N) for a constant shear stress amplitude (τ_cyc) and the cyclic shear strain amplitude (γ_{cyc}). The strain-contour diagram is developed from a number of laboratory cyclic tests performed on a soil sample. Cyclic shear stresses are applied at various stress levels to record the increase in soil shear strain at various shear stress levels and applied number of cycles. Examples of cyclic strain-contour diagrams can be found in DNV GL RP C212, Section 10.3. The equivalent number of cycles when predicting performance under normal operating (SLS limit states) and storm loading conditions (ULS limit states) can be established from the strain or pore pressure contour diagrams.

Considerations include:

- The ratio of cyclic loading relative to the average loading.
- Low average loads, combined with high cyclic loads can result in more rapid degradation in soil strength than higher average loads, even when combined with high cyclic loads.
- SLS limit states may require much lower thresholds for cyclic shear strains that more realistically correspond to the complications that would arise under normal operating conditions.

The impact of ice loading should be considered in regions susceptible to these seasonal conditions. Ice loads can impact the design of the structure but generally will apply large long-term one-way loading against the foundation, which can affect the level of cyclic degradation of the soils.

Guidance Note:
The influence of cyclic loading from wind, waves, storms, and ice includes the potential for the generation of excess pore pressure and subsequent accumulation of displacements, loss of undrained shear strength in clays, and liquefaction in sands.

Guidance Note:
The cyclic degradation can be exacerbated by an asymmetric cyclic loading pattern that is due to the combination of the cyclic load with a sustained static stress caused by wind, and/or wave, currents, and/or ice loads. If the static load is significant enough, the result can be a ratcheting effect seen with an excessive accumulation of displacement. Prediction of deformations due to cyclic degradation can be performed using numerical analyses coupled with calibrated and validated soil constitutive models.

8.2.2.4 **Earthquake Soil-Structure Response**

While soil responses may be similar to those described in Sections 8.2.2.2 and 8.2.2.3 for cyclic response, the loading rates are typically faster, loading durations are shorter and the inertial response of the structure is important. Both the effect of the presence of the foundation on free-field ground motions and the inertial loading from the structure to the foundations should be considered. Undrained responses are expected except for the most permeable soils.

Guidance Note:
For earthquakes, cyclic degradation can occur in the free field due to site response (propagation of waves) as well as due to interaction between the foundation and the surrounding soil.

Liquefaction shall be evaluated in seismically active areas. Liquefaction may cause a loss of vertical/axial capacity of shallow or deep foundations, loss of lateral pile capacity or stiffness, and lateral ground spreading. Additional discussion on liquefaction is given in Section 6.1.6.5.

Performance-based approaches may be applied in evaluating dynamic soil-structure response. Performance-based approaches should be conducted if simplified methods confirm the potential for triggering of liquefaction or significant excess pore-pressure generation.

> **Guidance Note:**
> Performance-based approaches to dynamic soil-structure response commonly use nonlinear dynamic analyses. It is common practice in such analyses to calibrate advanced constitutive models against available lab data (example cyclic direct simple shear or triaxial tests) and/or the cumulative body of data (as captured by empirical and semi-empirical relationships) and to then use these calibrated models in simulations where more complicated loading paths develop. It is extremely challenging to predict the loading paths that would develop in a real problem and even more so to reproduce them in the lab to calibrate a constitutive model. Confidence in nonlinear dynamic analyses results depends on the ability of the selected constitutive model(s) to: (1) represent at the soil element level the loading responses important to the problem being analyzed; and (2) capture important mechanisms and reproduce known system responses (i.e., validation through large scale tests and/or case histories).

8.2.3 Soil Parameters for Earthquake Engineering

Geotechnical earthquake engineering issues to address include but are not limited to site response analyses, liquefaction potential evaluations, dynamic slope stability evaluations and dynamic soil-structure interaction analyses. In addition to the parameters developed for static and cyclic loading, specific parameters are required for geotechnical earthquake engineering evaluations.

- Small-strain shear modulus – most commonly determined in the field by shear wave velocity measurements together with soil density measurements and in the laboratory from bender element testing or resonant column tests. In the absence of direct shear wave velocity measurements, several empirical correlations are also available for estimation of shear wave velocity by correlation to in-situ (example CPT) and laboratory (example undrained shear strength) test data. A hybrid approach may be adopted by developing site-specific correlations or site-specific validation/refinement of the empirical correlations.
- Strain-dependent variations in shear modulus and damping, most commonly determined from a combination of resonant column or torsional shear tests at lower strains and strain-controlled cyclic direct simple shear tests at higher strains. Several empirical relationships (example Darendeli, 2001; Vucetic and Dobry, 1991) are also available and can be considered by correlation to index and stress parameters.
- Rate of loading effects, most commonly determined in the laboratory by monotonic testing at different loading rates
- Liquefaction resistance, most commonly determined from in-situ testing (such as CPT data) and/or from stress-controlled cyclic simple shear testing. Performance-based approaches should consider the liquefaction resistance of the soils as well as post-liquefaction deformations. The assessment of soil liquefaction triggering and post-liquefaction deformations may require site-specific dynamic laboratory tests. Approaches for assessing liquefaction triggering and post-liquefaction deformations of siliceous soils from in-situ test results are available in Boulanger and Idriss (2016) and Tasiopoulou et al (2020), respectively.

Details on in situ and laboratory testing to derive the above parameters are provided in Sections 5.4.4 and 5.4.6.

8.2.4 Drainage Effects

Fully undrained soil behavior is defined as the condition whereby applied stresses and stress changes are supported by both the soil skeleton and the pore fluid and do not cause a change in volume (ISO 19901-4). The soil-structure response may be fully undrained, fully drained, or partially drained in sand. The drainage response is a function of the rate and/or frequency of loading, foundation geometry, and soil permeability. Hence, due consideration of the drainage response of the subsoil considering the foundation size should be undertaken.

For determination of the drainage that could occur under a footing, see the method described by the Norwegian Geotechnical Institute (1986). For a laterally loaded pile, Osman and Randolph (2012) provide analytical solutions for drainage of sand. The decay of pore-pressure with time within the relevant volume close to the pile shaft can be estimated as a function of the soil coefficient of consolidation and pile diameter.

> **Guidance Note:**
> An application of the consolidation solution considering two different pile diameters (1-m and 8-m) is shown in Peralta et al. (2017). The paper illustrates that some sands might behave fully drained for a 1-m diameter pile but may behave partially to fully undrained if the pile diameter is increased to 8 m.

8.2.5 Special Considerations for Micaceous Soils, Carbonate Soils, and Glauconitic Soils

Micaceous, carbonate, and glauconitic soils may be encountered in U.S. offshore sites. Chalk and rock, such as sandstones and limestones, may also be present.

8.2.5.1 Micaceous Soils

Mica minerals vary in composition and properties depending on their geologic formation and climatic conditions. Key features are its unique platy structure, high elasticity, and hexagonal sheet-like arrangement. Due to the platy structure and elastic properties of mica minerals, the presence of mica in soils can lead to adverse changes in the mechanical behavior of both cohesionless and cohesive soils. During compression or shearing, the mica particles tend to rotate in a somewhat parallel fashion due to the mica's platy shape, resulting in low strength resistance (Harris et al., 1984). Micaceous soils are generally characterized by high compressibility, poor compaction, and low shear strength. Lee et al. (2005) present and compare change in constrained modulus, peak and residual friction angles due to mica presence in sandy soils. Zhang et al. (2019) investigate the mechanical behavior of micaceous clays for varying mica content from 5% to 30%.

Where micaceous soils are identified, a carefully developed field and laboratory testing program may be warranted to establish the mica content, effects of mica on the stress-strain properties and cyclic strain behavior of the soil.

8.2.5.2 Carbonate Soils

Carbonate soils are soils with a significant proportion of calcium carbonate content. They are variably cemented and can range from lightly cemented with sometimes significant void spaces to extremely well cemented. Sands and silts containing more than 15% to 20% carbonate content are known to adversely affect foundation behavior. A carefully developed field and laboratory testing program would be warranted to establish the stress-strain properties and cyclic strain behavior of the soil.

Additional guidance on carbonate soils is provided in ISO 19901-4, Section 6.4.

8.2.5.3 Glauconitic Soils

Glauconite is a clay mineral (phyllosilicate), which forms part of the illite mica mineral group. It is thought to be formed by alteration of the faecal pellets of bottom-dwelling organisms, modification of illitic and biotitic clay particles, and precipitation from seawater. Glauconitic soils have a characteristic green to dark green color due to the iron content of the mineral. Pyritized glauconite particles are usually greenish black in color. The particles forming glauconite may be found as sand-sized pellets or finer, which often leads to it being identified in the field as glauconitic sand. The particles are generally sub-rounded to rounded with low to high sphericity and with a mineral hardness of 2 on Mohs scale (compared to quartz sand with hardness of 7 on Mohs scale). As such, glauconite particles can easily be crushed. When remolded, the soil no longer behaves as granular and becomes "clayey". Glauconite may be difficult to identify through visual observation alone, and may be difficult to differentiate from other minerals, e.g., biotite. However, classification and identification of glauconite can be performed through use of microscopic inspection of soil samples. Chen and Thompson (1995) and Yoon (1991) discuss and present behavior of glauconitic soils including comparisons of density, soil friction angle, undrained shear strength and maximum shear modulus.

Where glauconitic soils are identified, the crushability, compressibility, and mineral composition of glauconitic soils should be considered in the foundation design of offshore wind structures. A carefully developed field and laboratory testing program would be warranted to establish the stress-strain properties and cyclic strain behavior of the soil.

8.3 Foundation Design

8.3.1 Introduction

There are several types of offshore foundations to consider for offshore wind turbine support structures. These are divided into fixed wind turbine structures and floating wind turbine structures. Many of the variations of fixed structure foundations are applicable to floating structures but are subject to different loading conditions.

> **Guidance Note:**
> General guidance on a range of foundation types is available in the book Offshore Geotechnical Engineering (Randolph and Gourvenec,2011). Considerations for the design of suction piles is available in Jostad and Anderson (2021)

Foundation design considerations are dependent on the embedment ratio, or the length to diameter ratio (L/D), of the foundation that supports an offshore wind structure. The L/D ratio impacts the distribution of applied vertical, lateral and moment loads over the depth of the foundation, and therefore the natural frequency of the structure, the overall stability, and the serviceability. The L/D ratio by extension strongly influences the depth of soil investigation required, the type of geotechnical laboratory tests used to characterize the soil properties, and the type of installation method and vessel required.

In order of increasing L/D, the various foundation options comprise:

- Shallow foundations, comprising gravity base structures (GBS) or gravity base anchors (GBA), which generally have an L/D ratio less than 0.5 and may include short skirts around its perimeter to aid installation and increase capacity. For fixed structures, GBS foundations rely on their bearing area and on-bottom weight to provide stability against sliding, bearing, and overturning. For floating structures, GBA foundations predominantly rely on the on-bottom weight to provide uplift capacity. They are installed through a ballasting procedure onto an intact or prepared seabed, sometimes comprising a rock filter layer.
- Suction caissons or buckets, which generally have an L/D ratio between 0.5 and 2, are by definition skirted foundations. For fixed structures, suction caissons rely predominantly on their embedment (through skin friction, passive pressure , shear resistance), end bearing, on-bottom weight and (in some cases) short-term uplift capacity to provide sliding, bearing and overturning resistance. For floating structures, suction caissons are loaded based on the mooring configuration, i.e., catenary, semi-taut, or taut moorings. The capacity is derived from a combination of lateral and uplift resistance depending on the configuration. Suction caissons are installed through a combination of self-weight penetration and application of suction, or under-pressure, below the foundation base.
- Monopiles generally exhibit L/D values between 2 and 6, but historically have included values up to 10. Monopiles are generally only used for fixed structures, providing a combination of lateral, vertical, and overturning capacity due to its embedment (through skin friction) and on-bottom weight. They are installed mainly by driven or vibratory methods.
- Suction anchors, which are similar to suction caissons but with higher L/D ratios between 4 and 10. Suction anchors can be used for both fixed and floating structures. They derive their capacity from a combination of embedment (skin friction and passive pressure), end bearing (in downward compression) and temporary uplift (in tension) and are also installed through a combination of self-weight penetration and underbase suction.
- Jacket piles, which are generally much longer than monopiles and suction anchors with L/D values greater than 30. Jacket piles rely predominantly on skin friction, both in bearing and

uplift, but can generate significant end bearing if installed into a competent soil or rock layer. Jacket piles are generally installed by driving.

The following sections describe the key design aspects of each foundation type and provide recommendations for more quantitative guidance in the various codes in use. Note that some foundations for anchoring solutions, like drag embedment anchors, plate anchors, and fluke anchors, do not fall into a category with a specific L/D range. These have a separate section discussing design aspects of these more unique solutions for providing uplift capacity for floating structures. Furthermore, temporary shallow foundations such as mudmats for piled jackets, are addressed separately.

A variation of the shallow foundation is the spudcan, which provides temporary support to mobile drill rigs and installation vessels. Discussion on spudcans in this document is limited to penetration assessment as provided in Section 8.4.4. Installation of spudcans is generally the governing design scenario since the seabed needs to support full vertical load of the structure and resist catastrophic spudcan bearing failure, or punch through, during installation. The embedment ratio of spudcans can vary since the penetration (or L) is a function of the vertical load and the soil resistance to spudcan penetration. However, individually a spudcan behaves as a shallow foundation.

8.3.2 Gravity Base Structures

Guidance provided herein is intended for designing gravity base foundations. Gravity base structures (GBS) are considered those that resist applied loads predominantly by their own self-weight.

8.3.2.1 Bearing Capacity Methods and Recommendations

Recommendations for estimating the bearing capacity for shallow foundations are described in Clause 7.1 to 7.6 and A.7 of API RP 2GEO as well as 7.1 to 7.7 of DNVGL-ST-0126 and A.7.1 to A.7.7 of ISO 19901-4:2016. The methods for evaluating bearing capacity outlined in the guidelines include the following:

- Classical bearing capacity approach;
- Vertical-Horizontal (VH) loading failure envelopes; and
- Vertical-Horizontal-Moment (VHM) loading failure envelopes (yield surface).

Guidance Note:
The classical bearing capacity approach tends to be overconservative for loading conditions of combined inclined and eccentric loading and complex soil layering conditions whereas the VHM failure envelope is considered to be more accurate. VHM envelopes can be developed using experimental, analytical, or numerical methods. Closed-form expressions have been developed for specific conditions, but it may be more appropriate to develop site-specific envelopes depending on the complexity of the site and loading conditions.

Guidance Note:
When the existing soils lack sufficient strength to satisfy bearing capacity requirements, soil improvement, soil replacement or foundation skirts may be used to achieve additional stability. Seabed preparation in the form of imported rockfill may be required in areas of significant topographic variation, or areas of rock outcrop.

8.3.2.2 Mixed Soil Types and Layered Subsoil (silt, chalk, rock, etc.)

Site conditions that have mixed soil types or layered subsoils may warrant special in-situ or laboratory testing as well as numerical modelling to determine the interaction between the structure and the various strata. Examples for which these considerations may be required for GBS design include the following:

- punch-through failures with stronger soil layers overlaying weaker soil layers;
- sliding failures with shallow soil layers overlaying rock or thin shallow weaker soil layers;
- loose granular soil layers subject to potential liquefaction (see Section 6.1.6.5); and

- complex effective stress distributions resulting from layered materials with different drainage conditions.

8.3.2.3 Horizontal/Sliding Capacity and Use of Skirts

Recommendations for estimating the sliding capacity for shallow foundations are described in Clause 7.15 of API RP 2GEO and Clause 7.4.2 of ISO 19901-4. The evaluation against sliding failure should consider the following conditions:

- subsurface layering where conditions such as a weaker soil-soil interface may provide the preferential shearing surface in failure rather than at the soil-structure interface.
- the potential for gapping to occur below the foundation base under undrained sliding conditions, which would act to reduce the effective foundation area and factor of safety against sliding.

Guidance Note:
It should be noted that for drained loading conditions, decreasing the vertical load can have the effect of increasing the factor of safety against bearing capacity failure while decreasing the factor of safety against sliding and overturning. Foundation skirts can be used to increase the sliding resistance, effectively decreasing horizontal displacements as well. When analyzing the stability of gravity base foundations with skirts, the difference in soil height between the outside and inside of the skirt should be accounted for in the embedment design. Underbase grouting may be required to distribute load to the soil. Skirt compartments should be designed to ensure the intended soil-soil failure plane at the skirt tip is developed before shearing along a potentially weaker failure plane, such as the interface between the foundation base and the surface soil.

8.3.2.4 Pore Pressure Build Up and Mitigation Methods

Excess pore pressures generated in the soil during installation of a GBS will cause an initial reduction in undrained shear strength. As excess pore pressures dissipate under the load of the new foundation, undrained shear strength will recover and can eventually exceed the original in situ strength, offset by the effects of cyclic loading (see Section 8.2.2.3) over the lifetime of the structure. The potential for foundation instability and bearing capacity failure due to pore pressure build up can be mitigated with some of the following methods:

- Staged loading of ballast during installation
- Active drainage under the foundation during installation and/or for a prolonged period until satisfactory drainage/consolidation has taken place
- Passive drainage under the foundation over the lifetime of the structure

Guidance Note:
The dissipation of excess pore pressures can be slower for GBS foundations than other foundation types because the foundation footprint typically results in longer drainage paths. Installation of drainage blanket material along the base of the structure can improve drainage response in the soil.

8.3.2.5 Settlement and Displacement Analysis

Guidance for evaluating settlements and displacements of gravity base foundations can be found in:

- Section 7.5 of DNVGL-ST-0126
- DNVGL-RP-C212 (Edition September 2021), Section 5
- Section 7.8 and A.7.8 of API RP 2GEO 1st Edition
- Section 7.8 and A.7.8 of ISO 19901-4.

Immediate settlements of gravity base foundations should be determined according to the theory of elasticity where the subsurface material is expected to compress elastically upon loading. Long-term displacement should be determined for both primary consolidation and secondary compression (creep) as appropriate in cohesive materials.

Guidance Note:
Immediate settlement of a prepared rockfill layer can include shakedown of the rockfill during installation and storm loading. Model testing may be required to estimate the magnitude of shakedown for a given rockfill gradation, placement method and nature of applied loads.

Differential settlements should be considered for factors caused by moment, torsion, and eccentric loading, lateral variability in soil conditions below the foundation, and cyclic degradation. For evaluating eccentric loading conditions, the reduction in the effective area of foundations can be accounted for using the effective area method for simple, or simplified, foundation geometries. The effective area of complex foundation geometries can also be modelled without simplification using numerical modelling.

Guidance Note:
Issues caused by differential settlements include excessive inclination in the structure or the re-distribution of stresses below the foundation resulting in reduced bearing capacity. The installation of embedded foundations can result in differential settlements from the onset if there is difficulty obtaining the design embedment depth across the entire foundation due to inaccurate estimates of penetration resistance. Controlled, eccentric loading during installation may be warranted to reduce the potential for penetration-induced differential settlements.

8.3.2.6 **Effect of Gapping and Mitigation Measures**

Design considerations for gapping are provided in Section 7.5.5 of DNVGL-ST-0126. Gapping may occur when overturning moments result in tensile loads on part of the foundation and can result in scour and the detrimental re-distribution in the contact stresses between the foundation and the seafloor, potentially causing differential settlement, foundation instability, and/or changes in the rotational stiffness of the structure.

Gapping can be mitigated with the following measures:

- Foundation skirts to prevent uplift and erosion;
- Void filling with sub-base grouting to re-distribute contact stresses; and
- Increasing foundation ballast with concrete or iron ore to re-distribute contact stresses.

If the foundation design intentionally provides for gapping to occur (i.e., tensile loads are expected to develop on part of the foundation), the effects of gapping on differential settlement, bearing capacity, and rotational stiffness should be considered. Closed-form solutions for SLS and ULS limit state evaluations are less appropriate and not recommended when gapping is expected.

8.3.3 **Suction Caissons**

Suction caissons, otherwise known as suction buckets or suction cans, generally have L/D ratios between skirted GBS foundations and monopiles, and gain their capacity from a combination of lateral, vertical (compressive and tensile) and rotational resistance from the soil. Suction caissons may be designed as support to a jacket structure or as a mono-caisson, with the governing failure mechanisms for these applications likely to be different. The combination of shaft resistance along the skirts and bearing capacity beneath the base plate (both compressive and tensile) provide resistance to axial loads. The combination of the weight of the soil inside the caisson, and the short-term tensile capacity of the soil below the base plate, provide resistance to overturning during cyclic loading. The passive resistance from the soil surrounding the caisson in combination with the soil-soil shear resistance at level of the caisson tip and the shear resistance from the skirts provide horizontal capacity.

Note that this section does not cover suction caissons (or suction piles or anchors) designed to resist sustained uplift loading for mooring applications. For suction piles or anchors, refer to Section 8.3.8.5.

Guidance for design of suction caisson foundations is provided in:

- Offshore Wind Accelerator, Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines, February 2019, Issue. 1.0

Codes generally do not provide guidance on suction caissons to the same degree as shallow foundations and pile foundations, but the following standards have limited guidance for suction caissons:

- DNVGL-ST-0126 (Edition July 2018), Section 7.7;
- DNVGL-RP-C212 (Edition September 2021), Section 5.

For earthquake loading effects, refer to guidance in Section 8.3.9.

8.3.4 Monopiles

Monopile foundations are characterized by having a lower length (L) to diameter (D) ratio resulting in a rigid pile behavior compared to the flexible pile behavior of jacket piles (Section 8.3.5).

Guidance Note:
L/D ratios for monopiles used for OWTs typically range between 2 to 6.

The most common type of application for offshore monopiles is acting as support structure for OWTs, which results in the dominating loading being a combination of lateral loading and overturning moment loading (Section 8.3.4.1). However, monopiles may also be subjected to very high axial loading, e.g., when used as support structure for offshore substations or other types of platforms (Section 8.3.4.2).

8.3.4.1 Laterally Loaded Monopiles

Laterally loaded monopiles should be designed according to the following standards and guidelines: DNVGL-ST-0126 or API RP 2GEO/ISO 19901-4.

Guidance on topics considered particularly important for design of laterally loaded monopiles is given in the following subsections.

The geotechnical design shall ensure sufficient lateral pile capacity to withstand the external loads acting on the monopile to meet the design requirements for the limit states stated in Section 8.1.4.

The pile response can be modelled using the Winkler approach or other advanced models, e.g., 3D numeric modelling analysis.

Model tests data for monopiles can be found in several literature; Gilbert et al. (2015) provides results from model tests performed at the University of Texas, Austin.

8.3.4.1.1 API p-y Curves

API p-y curves (originally adopted by API RP 2GEO/ISO 19901-4 and included in DNVGL-ST-0126) have been applied for design of laterally loaded monopile foundations installed for OWTs. However, these may not predict the correct pile response due to the much larger pile diameters, lower L/D ratios and corresponding response mechanisms to loading typically governing today's offshore monopiles in comparison to the piles investigated for developing the API p-y curves.

If API p-y curves are used for geotechnical design, the guidance stated in DNVGL-ST-0126 and DNVGL-RP-C212 (2021) should be followed. This guidance includes various corrections (initial stiffness, diameter effects, etc.) and highlights the need for detailed validation of the API p-y curves, e.g., by use of 3D FEA or other advanced methods.

8.3.4.1.2 PISA Soil Reaction Curves

The PISA (Pile Soil Analysis) framework, which is specifically developed to capture the response of rigid monopiles, utilizes results from advanced 3D numeric modelling analyses to provide more accurate 1D monopile soil response predictions compared to the API p-y method. The PISA method has been validated against a large number of field tests.

The framework uses results from 3D numeric modelling analyses to calibrate site and/or unit-specific soil reaction curves, which are coded into more robust 1D numeric models, thus providing a computationally efficient method while maintaining a high level of accuracy for detailed geotechnical design.

The framework is based on four different soil reaction components. These are distributed lateral load $(p - v)$ and distributed moment $(m - \psi)$ springs down the pile shaft, along with base shear $(H_B - v_B)$ and base moment $(M_B - \psi_B)$ springs located at the pile tip.

As described by Byrne (2020), two alternative procedures are suggested for implementing the PISA methodology, namely the 'rule-based method' and the 'numerical based method'. Care should be taken in applying either method in situations for which they were not intended.

> **Guidance Note:**
> The outcomes of the PISA research program are provided in a series of nine Open Access papers in the journal Geotechniques (Géotechnique | Vol 70, No 11 (icevirtuallibrary.com)).

8.3.4.1.3 Other Methods for Establishing Soil Reaction Curves

Other soil reaction curves can be used if they can be proved appropriate and safe for the foundation solution and design in question.

8.3.4.2 Axially Loaded Monopiles

Axially loaded monopiles should be designed according to the following standards and guidelines:

- DNVGL-ST-0126
- ISO 19901-4
- API RP 2GEO

Guidance on areas that are considered particularly important for axially loaded monopiles is given in the following subsections.

8.3.4.2.1 Axial Pile Response

The geotechnical design shall ensure sufficient axial pile capacity to withstand the external loads acting on the monopile to meet the design requirements for the limit states in Section 8.1.4.

The axial response of monopiles can be determined by the API method or any of the CPT-based methods (NGI, ICP and UWA) as detailed in DNVGL-RP-C212 (2021). Other methods are allowed if they can be proved appropriate and safe for the foundation solution and design in question.

8.3.4.2.2 Other Important Aspects

Several aspects presented for laterally loaded monopiles also apply to axially loaded monopiles and should be taken into consideration, e.g. effects of cyclic loading (Section 8.3.4.5), earthquake loading and liquefaction (Section 8.3.9) and scour effects and scour protection (Section 8.3.4.6).

8.3.4.3 Monopile Foundation Initial Stiffness

Spectral fatigue analysis for evaluating structural performance under FLS and the NFA typically require linearization of the soil-pile response under relatively small displacements or strains. Guidance for evaluating appropriate elastic stiffness for FLS and NFA can be found in F.2.4 of DNVGL-ST-0126.

8.3.4.4 Effects of Cyclic Loading

Offshore monopiles are subject to cyclic loading from various loads such as wind and waves. Such effects may result in cyclic degradation (or strengthening in some cases) of the soil properties and will subsequently affect the monopile response. Cyclic effects shall therefore be taken into consideration in the geotechnical design.

No standardized method on how to design for cyclic loading exists. Recommendations given in DNVGL-RP-C212 (2021) can be used in the absence of more advanced analyses. Further details are provided in Section 8.2.2.3.

8.3.4.5 Scour Effects and Scour Protection

Scour development (due to erosion from waves and current) can occur around offshore piles and shall be considered in the geotechnical monopile design as it can have a negative impact on the lateral pile capacity. The recommendations provided in DNVGL-ST-0126 and API RP 2GEO can be used to determine the effects of scour in absence of more detailed analyses. Scour protection should be designed and installed if deemed necessary. Additional details are also provided in ACP OCRP-1, Section 7.7.3.

8.3.5 Jacket Piles

Guidance provided herein is intended for designing piles supporting jacket structures or other slender piles. For large diameter, small L/D ratio monopiles, please refer to Section 8.3.4 in this document.

Jacket piles or slender piles should be designed according to the following standards and guidelines:

- ISO 19901-4: 2016, Clause 8 and A.8
- API RP 2GEO 1st Edition (Addendum 1, October 2014), Clause 8 and C.8
- DNVGL-ST-0126 (Edition 2018), Section 7.6 and Appendix F

8.3.5.1 Axial Compression and Tension Pile Capacity Methods and Recommendations

Recommendations for pile axial capacity and resistance-displacement curves (t-z and Q-z curves) are outlined in Clauses 8.1 to 8.4 and A.8.1 to A.8.4 of ISO 19901-4 or Clauses 8.1 to 8.4 and C.8.1 to C.8.4 of API RP 2GEO as well as F.1 of DNVGL-ST-0126. For pile axial capacity in sands, ISO and API also recommend four CPT-based methods in the appendices in addition to the main text method.

Other methods are allowed if they can be proved appropriate and safe for the foundation solution and design in question.

8.3.5.2 Lateral Pile Capacity Methods and Recommendations

Recommendations for pile lateral capacity and resistance-displacement curves (p-y curves) are outlined in Clause 8.5 of ISO 19901-4 or API RP 2GEO as well as F.2 of DNVGL-ST-0126.

The pile response can be modelled using the Winkler approach. The p-y curves produced using the approaches described in these standards and guidelines may underestimate the actual lateral soil-pile stiffness and ultimate resistance. Recognized alternative p-y criteria that target best-estimate resistance-displacement response may be used to capture the actual pile and structural response and potential failure mechanism. Jeanjean et al. (2017), Zhang et al. (2017), and Zakeri et al. (2015) present alternative p-y curves in clays that may better capture the lateral response under monotonic, cyclic, and fatigue loading conditions, respectively. Alternative p-y curves in sands under fatigue loadings can also be found in Zakeri et al. (2015).

The lateral pile response can also be modelled using other advanced models, e.g., 3D FEA.

8.3.5.3 Pile Resistance under Cyclic Loading Conditions

The nonlinear t-z, Q-z and p-y curves recommended in ISO, API, and DNVGL standards and guidelines are intended to be used for evaluation of pile capacity under static loading conditions or the ultimate limit state (ULS). Cyclic loading conditions such as storm waves and earthquakes can have two potentially counteractive effects on the pile capacity:

- Cyclic degradation causing a decrease in resistance and/or an accumulation of deformation
- Rapidly applied loads causing an increase in resistance and/or stiffness of the pile foundation

Guidance regarding cyclic axial behavior of piles can be found in Clause 8.3.2 and A.8.3.2 of ISO 19901-4 or Clause 8.3.2 and C.8.3.2 of API RP 2GEO. Additionally cyclic interaction diagrams may be used such as those described in ICP (2005). These methods can be used in monotonic or push-over type evaluations, but alternative approaches may be required in dynamic analyses involving load-reversals.

The monotonic p-y curves for clay in Jeanjean et al. (2017) may be used to analyze piles under seismic loads. They have been shown to provide a satisfactory match between measured and calculated bending moments in fixed-structure piles when they are combined with appropriate unload-reload behavior and parallel dashpots to model radiation damping (Litton et al. 2014). For other soil types or for axial responses, load- or deformation-multipliers may be considered for the monotonic relationships for static loading. A range of multipliers should be developed taking into consideration the potential impacts of cyclic loading and rate of loading effects.

Piles installed in soils that can liquefy under design earthquake shaking levels require careful evaluation and may require coupled effective stress soil-structure interaction analyses. Guidelines for simplified modelling of liquefaction effects on piles are available in Brandenberg et al. (2007) and Boulanger and Brandenberg (2004).

8.3.5.4 Pile Foundation Initial Stiffness

Spectral fatigue analysis for evaluating structural performance under fatigue limit state (FLS) and the natural frequency analysis (NFA) typically require linearization of the soil-pile response (t-z and Q-z curves) under relatively small displacements or strains. Guidance for evaluating appropriate elastic stiffness for FLS and NFA can be found in F.2.4 of DNVGL-ST-0126.

8.3.5.5 Pile Group Behavior

Generally, pile group effects should be evaluated if the center-to-center spacing of adjacent piles is less than eight pile diameters. Recommendations for evaluating pile group effects can be found in Clause 8.6 and A of ISO 19901-4 or Clause 8.6 and C.8.6 of API RP 2GEO.

8.3.5.6 Effects of Scour on Piles

Scour or seabed erosion due to wave and current reduces the overburden stress and lateral soil support. Two common types of scour can occur around piles:

- General scour (overall seabed erosion)
- Local scour (steep sided scour pits around piles)

Recommendations and guidelines considering scour effects on p-y curves can be found in A.8.5 of ISO 19901-4 or C.8.5 of API RP 2GEO. For scour effects on pile axial capacity in sands, A8.1.4.2.7 of ISO 19901-4 and C.8.1.4.4.1.7 of API RP 2GEO provide recommendations to account for its effects for CPT-based methods. Additional guidance is provided in Section 8.5.2.

8.3.6 Drilled and Grouted Piles

8.3.6.1 General

A drilled-and-grouted pile is a type of pile which is typically installed by drilling a hole below the seafloor, removing the drilling tool, lowering a steel pipe into the open hole, and grouting the annulus between the steel pipe and the ground. A two-stage variation comprises two concentrically placed piles in which the outer pile is driven to a given penetration and the inner or ‘insert’ pile is lowered into a drilled hole performed beyond the toe depth of the outer pile. Another variation comprises the use of a temporary casing lowered as the drilling progresses, when the drilling reaches target penetration, the drill is removed, the pile lowered inside the casing and the casing extracted as grouting between the pile and ground progresses.

Other variations on these methods exist, such as pressure grouting through perforated piles and pre-grouting of open holes, prior to inserting piles.

Drilled-and-grouted piles can be installed in rock and soils which will hold an open hole with or without drilling mud. Their design is primarily based on FHWA-IF-99-015 (2000), API RP 2A-WSD(2014), DNVGL-ST-0126 (2016) and CIRIA R181 (1999).

Drilled and grouted piles may be considered advantageous over driven piles in projects with hard soils and rocks as the installation process gives a predictable end pile depth and installation duration. Cycling drive/drill processes are avoided. It may also be considered where environmental restrictions, such as those associated with the presence of marine mammals, prohibit or restrict the use of large, powerful impact hammers, provided that the ground conditions are suitable.

8.3.6.2 Load Transfer

A load-factor approach can be adopted to satisfy SLS requirements implicitly through ULS calculations; however, an attempt to perform more realistic calculations of behavior under SLS should be made. It is recognized though that the load transfer mechanism of drilled-and-grouted piles is complex and there are not yet sufficiently established numerical and analytical procedures for design applications.

Drilled-and-grouted piles transmit their load to the ground through a combination of side shear and base loading. The relative stiffness of the ground and interface properties will control the nature of the load transfer. The design shall satisfy ULS and SLS conditions using the examination of ultimate capacity as a starting point for serviceability calculations. As for driven piles, the components of pile capacity that come from shaft resistance and from base resistance can be examined separately.

For serviceability calculations, stiffness values may have to be assumed and so it is paramount that the ground investigation produces realistic results in this regard. Consideration should be given to the potential changes of the stiffness and strength properties modified during the installation process, and special circumstances such as stiffness values for rock suspected to be considerably anisotropic. Consideration should also be given to the anticipated degree of grout continuity around the pile, which will be a function of the installation process.

8.3.6.3 Axial Compression and Tension Pile Capacity Methods and Recommendations

The axial compression and tension pile capacity methods are provided in the design requirements and guidelines given in FHWA-IF-99-015 (2010), FHWA NHI-10-016 (2010), API RP 2A-WSD (2014) and CIRIA R181 (1999). Other methods may be used if they can be proven appropriate and safe for the foundation solution and design in question.

The ultimate axial load capacity of drilled-and-grouted piles is a function of several factors, often interconnected, that include the pile and pile socket geometry, pile socket roughness and cleanliness along the shaft and base, mechanical properties of the ground, continuity of grout annulus, shear strength at the pile-grout-ground interfaces, normal stresses acting on the pile shaft, and rock mass properties where applicable.

8.3.6.4 Shaft Capacity

The shaft capacity of drilled-and-grouted piles can be defined following the approaches from the FHWA-IF-99-015 (2010), FHWA NHI-10-016 (2010), DNV-ST-0126 and API RP 2A-WSD (2014) or API RP 2A-LRFD (2019), accounting for recommendations found in the CIRIA R181 (1999) guidelines.

Specific shaft capacity factors to the drilled-and-grouted design case should be applied carefully to ensure sufficient conservatism. These factors need to be refined based on specific ground conditions, the interface between grout and rock and grout and pile, and the chosen installation methods. If drilling mud is used, its effect on the shaft capacity should be considered. Relevant recommendations can be found in multiple references cited in the CIRIA R181 (1999) guidelines.

It is a common practice to apply a limiting skin friction for shaft capacity calculations. The limiting value is expected to be a function of the grouting method, interface roughness, rock/soil strength, initial normal stresses on the interface and the rock/soil stiffness during interface dilation. Therefore, the limiting value shall be confirmed once the drilling equipment, grouting equipment and parameters and rock parameters are all finalized. Existing literature references can vary widely for rock and soils as presented in guidelines such as FHWA-IF-99-015 (2010) and BS8081 (2015).

Ultimately, the unit skin friction should be taken as the lower of the strength at the interface of the rock and the grout or at the interface of the grout and the pile. A few methodologies have been suggested for calculating unit skin friction in rock based on either load tests or collective review of available databases including load tests reported by various authors. Comprehensive summaries of load test data have been presented by Horvath and Kenney (1983), Williams and Pells (1981), Rowe and Armitage (1987), Kulhawy and Phoon (1993) and Prakoso (2002). The databases used for recommending a design approach were based on load test data conducted predominantly in sedimentary rocks.

The above databases are predominantly limited to piles with small diameters and the associated recommendations for assessing unit shaft friction are independent of diameter. Theoretical frameworks relating shaft friction to dilation such as presented in Seidel and Collingwood, 2001 (Seidel and Collingwood 2001) imply a reduction of shaft friction for increasing diameter and this trend was also observed in pile load tests that systematically assessed diameter effects reported by Manceau et al., 2021 (Manceau et al. 2021). Therefore, the extrapolation of recommendations based on databases of typically small diameter piles may lead to an overestimation of shaft friction for large diameter offshore piles.

When selecting a design method to assess drilled and grouted pile capacity, the designer should prove that the design method is appropriate for the pile geometry, ground conditions, allowable displacement and loading regime considered on site. This can be done by a project specific pile load testing program. A comparison with historical pile load tests or alternative evidence-based approaches can also be considered although the relevance of the method should be proven.

The interface strength between the grout and the pile defined as the bond strength can be calculated in accordance with DNVGL-ST-0126 (2021). Therefore, the steel-grout bond capacity is often used as a limiting value for the design. The capacity of the steel-grout interface can also be upgraded with shear keys to minimize the size of the socket required to resist tension and ensure that this failure criteria does not govern the design of the socket lengths. Shear key design codes are available from the American Institute of Steel Construction (AISC) and/or the American Concrete Institute (ACI).

8.3.6.5 **Bearing Capacity**

The bearing capacity calculations of drilled-and-grouted piles can be based on API RP 2A-WSD(2014) codes and CIRIA R181 (1999) guidelines applying appropriate bearing factors to the specific ground conditions under consideration. The effective base area could comprise the full base area of the pile and drilled annulus, only if it can be demonstrated that the pile base will be fully grouted.

Sockets shall be built with significant roughness, otherwise restrictions on the use of end bearing in design for smooth sockets would apply. It is also noted that for offshore drilled-and-grouted piles in compression, the end bearing component is commonly neglected due to the possible presence of cuttings and/or drilling mud in the bottom of the hole and the significant displacement required to mobilize end bearing. If the end-bearing component is to be relied upon for design, the installation method should reliably ensure a clean base of drill hole and the design should be based on a load transfer analysis to assess the proportion of end bearing mobilized.

8.3.6.6 Effect of Cyclic Loading on Axial Capacity

The percentage reduction of axial capacity due to cyclic loading can be quantified based upon pile load tests or site-specific cyclic laboratory tests for drilled-and-grouted pile design. Laboratory tests should be designed to examine the behavior of the interface between the soil or rock and the grout if it is deemed to be the governing factor in the design of socket length. This assessment could also be supplemented with the use of the published interaction diagrams.

According to DNVGL-ST-0126 (2016) recommendations, a detailed calculation of the effects of cyclic loads induced by standard wind turbines may be omitted if it is shown that no tensile forces occur for axially loaded piles under the SLS load case LDD 10-2 (i.e., the load level only exceeded 1% of the time equivalent to 1750 hours in 20 years).

8.3.6.7 Resistance-Displacement Curves in Clay and Sand

The formulation of axial resistance-displacement curves (t-z and Q-z curves) in soils for drilled-and-grouted piles can be based according to DNVGL-RP-C212 (2021) and API RP2A-WSD (2014) recommendations, see 8.3.5.1. It is noted that, in some cases, the axial performance of the piles in the superficial deposits and in the rock layers along an installed casing is often set to 0.

8.3.6.8 Resistance-Displacement Curves in Rock

For drilled-and-grouted piles socketed into rock, the ultimate skin friction shear force that can be reached will depend on the maximum bond strength between grout and steel or grout and rock. The lower of these two interface shear forces that can be mobilized will limit the maximum t-z shear force used for design.

The mobilization of unit shaft friction depends on several factors including the rock mass stiffness and the roughness of the rock socket. It can be assumed that the t-z curves follow an elastic perfectly plastic response, and the ultimate resistance will be experienced after a certain displacement. The magnitude of this displacement is uncertain, and a sensitivity study is always recommended to examine the effect of the peak displacement on the axial response of the structure. Some reduction on the axial pile response would be expected as a result of selecting a higher peak displacement, but the natural frequency of the structure response may or may not be influenced significantly by this. Ultimately, it is paramount to predict the natural frequency of the offshore wind turbine-support structure-foundation system because both an under- or and over-prediction of the natural frequency may be unconservative.

For drilled-and-grouted piles in rock, the Q-z springs, if required, can be assessed following the methods provided in API RP2A-WSD (2014). Note that the rock socket length to diameter ratio for drilled-and-grouted piles considered for wind farms will generally be greater than 4. Therefore, the axial capacity of these rock sockets would be supported predominantly by side shear with negligible contribution from base resistance. The side shear resistance will be mobilized at relatively small displacements compared to the base resistance. Therefore, the Q-z resistance could be neglected for rock socketed piles with L/D ratios greater than 4. Ratio's lower than 4 may typically be expected for rock socketed monopiles, or full drill-and-grout monopiles which may also be considered.

The calculation of the pile settlements will be in accordance with DNV-RP-C212. The total axial pile settlement will include the immediate settlement and dynamically induced settlements which will be derived from the in-place analysis and the appropriate loading conditions.

8.3.6.9 Lateral Pile Behavior

There is little data on lateral behavior of drilled and grouted piles (particularly for monopile in rock) and consideration should be given to uncertainties in predicting lateral response when critical in design.

The determination of the lateral pile response of grouted piles in weak rock may be determined with reference to Reese (2011).

Reese's methodology proposes that the lateral resistance-deflection behavior (P-Y) is represented by a curve; the initial gradient of which is directly proportional to the initial Young's Modulus of the rock mass (Eir). Rock is likely to be highly anisotropic and so it is important that Eir is taken from tests which strain the rock laterally. Pressuremeter tests are a suitable means of providing such test data in-situ and the initial gradient of the pressuremeter test may be used.

The presence and influence of highly weathered, soil infilled or open joints within the rock mass should be considered, particularly if rock exists near the top of the pile where confining stresses are low. The presence of such features may permit the displacement of blocks of rock, and hence the pile, at relatively low lateral pressures. It is noted that, generally, the lateral performance of piles in rock is mainly influenced by the rock strength, the rock mass modulus, and the quality of the rock, among other factors. The quality of the rock mass may be assessed by Geological Strength Index (GSI) (Hoek, 1994), Rock Mass Rating (RMR) (Bieniawski 1989) or Q System (NGI, 2013) methods.

8.3.6.10 **Axial Skin Friction Considerations -Drilled Holes and Shear Keys**

According to the API, the diameter of the drilled hole should be at least 6 in. larger than the pile diameter. API RP 2A-WSD (2014) or API RP 2A-LRFD (2019), API also states that the selection of skin friction values should consider soil disturbance resulting from installation.

The API also recommends a check be made of the allowable bond stress between the pile steel and grout (API RP 2A-WSD (2014), Para. 10.4). The interface strength between the grout and the pile defined as the bond strength can be calculated in accordance with DNVGL-ST-0126 (2021)). The presence of shear keys will increase the strength of the pile-grout interface and move the failure plane to the soil-grout interface. Laboratory and/or pile load testing could be performed to refine the limiting factors and allow for a more optimized design.

8.3.6.11 **Socket Stability**

The borehole stability assessment depends upon the available geotechnical information and the proposed installation method including factors such as drilling equipment, socket cleaning, standing time, etc. The socket stability is a function of the drilling method/socket roughness, grouting method and pressures applied, unsupported time period, and where applicable rock mass characteristics including weathering, fracture infill, fracture orientation, roughness, and frequency.

The potential for 'block' collapse or drilling induced collapse from the socket wall should be assessed using the available information and detailed information regarding drilling and grouting methods. In rock, fracture characteristics from rock core descriptions or ideally from acoustic or optical tele-viewer data should be used in this assessment. Short-term and long-term stability issues, including creeping and secondary effects, could be assessed using Numeric Modelling Methods. Plastic deformation and rock displacements due to wedge sliding and global instability should be considered. For local stability checks, a wedges kinematic stability analysis could be performed based on the acquired acoustic or optical tele-viewer data at each specific location. The purpose is to identify potential discontinuities resulting in wedge instability. Stereographic techniques or a poles cluster approach could be also considered.

In any case and regardless of the ground conditions, the stability and in-situ condition of the socket should be assessed prior to the grouting process.

8.3.6.12 **Other Design Considerations**

Although used offshore in the past, the experience with drilled-and-grouted piles is still limited in comparison with other piled foundation solutions. There is still a need to study various construction techniques and develop specific guidelines for construction, and capacity and integrity testing of such piles.

Particular design issues for drilled-and-grouted include, but are not limited to, the difficulty in modelling the interaction load transfer mechanisms, mechanical properties of the ground being modified by the installation process which affects the pile performance, assurance of the continuity of the grout annulus, pile design being generally based on the ULS which gives no indication of performance under serviceability conditions, and an over-conservative pile design based on the weakest parameters of the strata, among others. The solutions to these issues should include an improved understanding of the behavior and mechanism along the different pile-grout-ground interface boundaries, the use of models which consider SLS pile performance, and the adoption of a flexible design procedure which allows modifications and optimizations in response to specific ground conditions.

Other design considerations include the implications of using a sacrificial casing to support the superficial deposits and extended to competent strata of soils or rock. In such cases, the unit skin friction along the casing length may be neglected for pile design if left in-situ or to reflect the disturbance due to the installation process if it is eventually removed.

8.3.7 Temporary Shallow Foundations

Temporary shallow foundations, such as piled jacket mudmats used for temporary support during installation, are subject to the same design principles as permanent shallow foundations such as GBSs. However, due to their short-term life in the field, simplified design approaches for assessment of stability and serviceability can be adopted. Further information can be found in API RP 2GEO Section 7 or ISO 19901-4:2016 Section 7.

8.3.8 Anchor Solutions for FOWTs

8.3.8.1 General

There are several types of anchor solutions for floating structures, comprising:

- Plate anchors (drag-embedded, direct-embedded, suction-embedded, fluke anchor)
- Anchor piles (driven, suction, jetted, drilled and grouted)
- Gravity anchors (clump anchor, free fall, or torpedo anchor)

General information on floating wind turbine structures and anchoring solutions can be found in DNVGL-ST-0119 Design of Floating Wind Turbine Structures, including an overview of the various anchoring options listed above. The requirements for foundation design given in DNVGL-ST-0126 apply to the geotechnical design of anchoring systems referenced in DNVGL-ST-0119. Some anchor types may only be suitable for single turbine anchoring, e.g., fluke anchors and plate anchors.

The global safety factors applicable for the ULS for the design of anchors for floating structures are distinguished into mobile, or temporary structures such as drill rigs, and permanent structures. All floating wind turbine structures should be considered permanent structures since they are not associated with temporary field activities. Global safety factors for specific anchor solutions are given in ISO 19901-7, defined as anchor capacity divided by the extreme value of the anchor force from dynamic analysis:

- Table 6 of ISO 19901-7 for the design of drag anchors;
- Table 7 of ISO 19901-7 for the design of anchor piles, comprising driven piles, suction piles, and gravity-embedded anchors (i.e., free-fall ‘torpedoes’);
- Table 8 of ISO 19901-7 for the design of gravity anchors and plate anchors.

For all other types of anchors, a similar level of reliability should be achieved as outlined in these tables.

Further details for most anchor types can be found in ISO 19901-4, supplemented by additional information for specific anchor types in:

- DNVGL-RP-E301: Design and installation of fluke anchors
- DNVGL-RP-E302: Design and installation of plate anchors in clay
- DNVGL-RP-E303: Geotechnical Design and Installation of Suction Anchors in Clay

Guidance note:

For certain mooring configurations, the phenomenon of chain trenching can occur. This acts to remove soil between the mooring-foundation connection (termed the pad eye) and the floating structure. The degree of chain trenching is also affected by the sea state and soil conditions but should be considered in regions where observations of chain trenching have occurred.

8.3.8.2 Gravity Base Anchors

The weight of a gravity base anchor is the main resisting force for the tension applied by the mooring line. The anchor may be designed with a short skirt to resist sliding. Such a foundation is suitable in situations when it may be necessary to re-locate the floating structure elsewhere as removal of the anchor is relatively easy.

Foundation design of gravity base anchor is similar to that of gravity base foundation, refer to Section 8.3.2.

8.3.8.3 Drag Anchors and Vertical Load Anchors

The holding capacity of the drag anchor is generated by the resistance of the soil in front of the anchor. The anchor consists of four main components: fluke, shank, shackle, and chain. The weight of the anchor, geometry and angle of penetration are critical factors in determining holding capacity.

Anchor capacity may be calculated by empirical methods, analytical methods using limit equilibrium or using numeric modelling.

The empirical method relates the depth of penetration to the weight of the anchor. Design curves for clay and sand are available in API RP 2SK and ISO 19901-7.

Vertical Load Anchors (VLA) are similar to drag anchors and usually installed in the same way, except that the VLA can resist both horizontal and vertical mooring forces. It is used primarily in taut leg mooring systems, where the mooring line arrives at an angle at the seabed.

8.3.8.4 Driven Anchor Piles

Driven anchor piles are treated similarly to jacket piles, refer to Section 8.3.5.

8.3.8.5 Suction Piles

The capacity of a suction pile, or suction anchor, may be checked by either limit equilibrium methods or the numeric modelling method. Suction piles are usually wider than driven piles and may be assumed as ‘rigid’ in comparison to the surrounding soil. The padeye that connects the mooring line to the suction pile is usually placed about 2/3 of the way down the pile from the seabed. This ensures that the pile rotates during extreme operation loads, thus no gap develops on the upper back side of the pile. An open gap may reduce the holding capacity and in extreme cases may undermine the tension suction resistance at the pile tip.

The loads at the padeye of the anchor are different in both magnitude and angle from the loads of the corresponding mooring line at the mudline. The foundation load at the padeye becomes smaller than the corresponding line load at the mudline, and the loading angle at the padeye will be greater than the loading angle at the mudline. The change in shape and load is due to soil-chain friction acting tangentially to the chain and bearing resistance acting normally to the chain. The soil resistance typically results in an inverse-catenary mooring line shape of the embedded chain.

8.3.8.6 Screw (helical) Piles

Screw piles, also referred to as helical piles, are a steel screw-in piling and ground anchoring system used for building deep foundations. The pile shaft transfers a structure's load into the pile. Helical steel plates are usually welded to the pile shaft. The number of helices, their thickness, diameters, and position on the pile shaft are determined by design load and geotechnical parameters. Further reference on geotechnical parameters can be found in Section 8.2.2.1.

8.3.8.7 Hybrid Anchors

Hybrid anchors generally constitute a pile rafted foundation in which a gravity foundation is constructed on the seabed with holes that allow for installing small drilled and grouted anchor piles. This design allows for use of a smaller diameter and shallower gravity base as the load is transferred deep into the ground. The upper gravity base may be of precast concrete. The foundation can then be considered as pile group with the gravity base represented as a pile cap. A hybrid anchor foundation system can then be designed as a pile rafted foundation with consideration given to pile group effect.

8.3.9 Considerations for Earthquake Loading

The recommendations for earthquake loading given in API RP 2EQ should be followed when designing foundations for seismically active regions. Additional guidance for offshore wind facilities is available in DNV-RP-0585 – Seismic design of wind power plants. Earthquake loads can affect the soil properties such as soil stiffness and strength parameters. The change in soil strength and stiffness properties should be investigated as relevant. Cyclic degradation can also occur due to soil structure interaction.

Possible liquefaction risks and impact on the foundation response shall also be considered for seismically active regions and taken into account in the geotechnical design. Foundations installed in, above, or through soils that can liquefy under design earthquake shaking levels require careful evaluation and may require coupled effective stress soil-structure interaction analyses. If simplified approaches suggest that soils will liquefy for the design earthquake these evaluations should be based on Performance-Based methods using coupled effective stress soil-structure interaction analyses to ensure that the accumulated deformations and associated forces are tolerable. Consideration also should be given to loads (for example downdrag) and deformations resulting from post-liquefaction settlement of the surrounding soils.

Earthquakes can also trigger more widespread instability (e.g., slope failures, lateral spreading, etc.) that may impact the foundation. Site-specific geohazard assessments should consider the potential for such earthquake triggered actions and define appropriate mitigation schemes. Section 6.1.6 provides further guidance on seismicity and earthquake effects.

Soil structure interaction would preferably be captured using time-domain dynamic analyses where appropriate constitutive models capture the variation of soil strength and stiffness resulting from the simultaneous action of ground motions and interaction with the structure. Where such analyses are not warranted or overly onerous, the monotonic soil deformation curves developed for foundation assessments can be considered as a baseline for dynamic analyses together with suitable unloading/reloading criteria and multipliers to incorporate cyclic degradation or strengthening effects. A range of multipliers should be developed taking into consideration the potential impacts of cyclic loading and rate of loading effects to simulate median, low estimate and high estimate strengths/stiffnesses. It should be recognized that for earthquake analysis cyclic degradation occurs both in the free-field due to wave propagation and due to soil-structure interaction.

Foundations should be designed to resist inertial loads from the superstructure (in combination with static loads) as well as kinematic loads resulting from deformation of the surrounding soils due to the earthquake. The consideration of kinematic loads is important when: a) the ground contains layers of sharply differing stiffness and when design shaking levels are moderate to high; or b) when a liquefiable layer is present; or c) in cases of soil-free-field displacement (example lateral spreading).

Guidance Note:
Coupled or de-coupled approaches may be adopted for evaluating the performance of foundations. In coupled approaches, the inertial response of the structure, foundation and soil are all included in a single dynamic model, whereas in decoupled approaches the effect of the foundations on the earthquake ground motions are first captured in a kinematic analysis and then used as inputs to a structural model with a simplified model of the soil support. In all analyses, appropriate consideration should be given to the potential impacts of cyclic loading and rate of loading effects to simulate median, low estimate and high estimate strengths/stiffnesses or ground motions.

Guidance Note:
Liquefaction-induced deformations of shallow foundations include: (a) Shear-induced deformations and (b) Volumetric settlements. Shear-induced deformations (horizontal and vertical) accumulate from the development of permanent shear strains as a result of partial bearing capacity loss and SSI-(Soil-Structure-Interaction)-Induced ratcheting. The former can be induced either by soil softening near the foundation and/or by liquefaction in the free field. SSI-Induced ratcheting results from out-of-phase structure movement relative to the subsoil. It occurs with or without soil liquefaction, however, the magnitude of the associated deformations depends on the degradation of the foundation subsoil. Shear-induced settlements mostly occur co-seismically, but post-shaking failures can occur due to loss of bearing capacity from factors like void redistribution, soil mixing and strain localization below impermeable layers. Volumetric settlements occur as a result of drainage and re-consolidation following excess pore pressure dissipation. They mainly occur after the end of shaking, however they can in-part occur concurrently with the earthquake, especially if the soils are highly permeable. Chaloulos et al. (2019) describe methodologies to assess co- and post-seismic shear-induced and volumetric settlements, while Bray and Macedo (2017) provide simplified methods for estimating liquefaction-induced settlement of shallow foundations.

8.4 Foundation Installation

Guidance on the transport and installation of offshore wind farm components is provided ACP-OCRP-1, Section 7 with installation of foundations discussed in ACP-OCRP-1, Section 7.7. Additional guidance on specific issues is provided below.

8.4.1 Gravity Base Foundation Scour Protection

In conditions where scour is a concern for gravity base foundations, foundation skirts can be used both for scour protection as well as increased foundation stability. When foundation skirts are used, the penetration resistance of the skirt during installation should be accounted for in the installation procedure. Scour can also be mitigated by the placement of scour-resistant materials around the outer edges of the foundation. In some cases, the scour protection design can alter the loads against the foundation and should be considered.

8.4.2 Skirted Foundations, Suction Piles and Suction Caissons

8.4.2.1 General

Skirted foundations, suction piles, caissons, and bucket foundations all rely on sufficient penetration of the pile or skirt length to provide the required sliding, bearing, overturning, and/or uplift capacity. Penetration of the foundation into the seabed generally comprises a portion of embedment through the self-weight of the foundation and the remainder through application of underpressure, or suction, below the base. The maximum pressure is limited by the cavitation, pump capacity and buckling capacity of the suction bucket or skirted foundation.

Unit skin friction and unit end bearing can be determined from analytical solutions using soil properties measured in geotechnical laboratory tests. Alternatively, they may be estimated from geotechnical laboratory tests and/or in-situ data.

Detailed guidance on skirted foundations and suction pile / anchor / caisson installation is provided in:

- ISO 19901-4 Section 7.6.2, Appendix A.7.6.2 and Appendix A.11.5.2
- DNVGL RP C212 Section 7.3
- DNVGL RP N103 Section 6.3
- DNVGL ST-0119 Section 9.4
- Offshore Wind Accelerator, Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines, February 2019, Issue. 1.0

Some key considerations for suction-assisted installation are provided in the following sections.

8.4.2.2 **Effect of Suction**

Application of underpressure below the foundation base can impact penetration resistance and achievable depth of the foundation by:

- Increasing the tendency for plug heave in clay and sand
- Reducing the penetration resistance in sand
- Increasing the potential for foundation buckling in the case of a square or rectangular skirted caisson

The maximum underpressure is limited by cavitation, which is important in shallow water as well as the pump and buckling capacity of the foundation.

8.4.2.3 **Effect of Soil Variability and Ground Conditions**

The level of soil layering and geological complexity within the depth of penetration can impact installation by:

- Increasing the potential for contact of hard layers or boulders that stop further penetration. Contingency planning should provide mitigation measures in the case of premature refusal.
- Limiting the ability of a seal to develop around the foundation perimeter, limiting the ability to maintain suction pressure.
- Causing rotation of the foundation during penetration, resulting in gapping and limiting the ability of a seal to develop.

Increased control during the levelling process is recommended in variable ground conditions, in particular for multiple-foundation configurations and large skirted foundations with more than one suction port being operated.

8.4.2.4 **Effect of Foundation Geometry**

The design of the foundation can impact the penetration resistance and required suction pressures as follows:

- Friction break at the skirt or pile tip to reduce skin friction along the inside of the pile.
- Internal horizontal stiffeners that create additional end bearing resistance to penetration, potentially offset by reduced internal skin friction.
- Internal vertical stiffening and dowels that create additional skin friction resistance.
- Wedge-shaped (concrete) skirts that increase the penetration resistance through passive pressure.

8.4.2.5 **Monitoring Guidance**

Real time monitoring is required to allow timely response to issues that arise during suction installation, and may assist in checking how the observed resistance compares with the predicted resistance .

A comprehensive list of data to be monitored is provided in ISO 19901-4 Section A.11.5.2.4.

8.4.2.6 **Foundation Landing and Retrieval**

The installation planning process should include an assessment of water pressures below the base of the foundation in relation to the lowering rate and base plate venting area. It should be ensured that the pressure from the water cushion below the base does not exceed the bearing capacity of the soil, as this could create a blow out around the skirt, caisson or pile perimeter and limit the ability to create a seal before applying under pressure.

Detailed guidance can be found in ISO 19901-4 Section A.7.6.2.1, DNVGL-RP-N103 Section 6.4, and DNVGL RP C212 Section 7.3.7.

Other considerations include:

- Landing on seabed to ensure that foundation failure does not take place during landing and damage does not occur to acceleration sensitive equipment.
- Foundations without skirts also need to ensure sufficient area is available to avoid excessive sliding or "skating" of the foundation on landing.
- Foundation installation dynamics should be considered to capture the effect of vessel heave, crane wire tension, seabed currents on the water pressure and vertical reaction force subjected to the soil and foundation.

8.4.3 Leg or Spudcan Penetration Assessment

Location specific leg penetration assessments are required for each planned jack-up vessel installation for wind farm construction, O&M and decommissioning operations. The purpose of the leg penetration assessment is to identify and quantify possible installation geohazards and determine the depth to which the spudcan (jack-up leg footing) will penetrate the seafloor during installation preloading or pre-driving.

The maximum design predrive is imposed on each spudcan during installation to ensure that the jack-up has adequate capacity to withstand the design operational and environmental loads which could be expected during its elevated operation on location. For jack-up vessels employed in the renewable energy industry, with limited predrive capabilities, the critical foundation loading condition frequently occurs during crane operations as opposed to environmental storm conditions. Hence precautions should be taken to make sure that foundation instability doesn’t occur during weather-restricted operations, (RUK 2013).

Geotechnical analyses for leg penetration assessments should follow the design procedures and guidelines given in ISO 19905-1. Further information is provided in the Society of Naval Architects and Marine Engineers (SNAME) Technical and Research Bulletin 5-07: Guideline for the Site-Specific Assessment (SSA) of Offshore Wind Farm Jack-ups (2024).. Additional recommendations with regards site investigation and soil parameter selection, together with a review of spudcan penetration methodologies, are provided in InSafe JIP (InSafe JIP 2011), ISO 19901-8, and ISO 19901-10.

To assess the geotechnical risk at any given location it is necessary to have sufficient data to an appropriate depth to allow an assessment of the predicted leg penetration to be made with confidence. The geophysical and geotechnical site investigation requirements for a spudcan foundation assessment are provided in ISO 19905-1, 19901-8 and InSafe JIP guidelines which are provided based on the geological conditions, understanding of near-surface conditions and whether jack-up vessels have been previously installed at the proposed installation location.

Leg penetration analyses rely on a detailed understanding of specific geotechnical and geophysical conditions at each installation location, the spudcan geometry and the loading regime (stillwater load, predrive and maximum operational loads). The foundation failure mechanisms of punch-through, sliding and general shearing shall be considered in the analyses. Punch-through can be considered the greatest geotechnical risk to installation, a 2004 HSE study (HSE 2004) attributed over half of the incidents investigated to punch-through/rapid leg penetration, however sliding and seabed scour are also key stability risks which should be considered in the analyses and reporting. The effect of spudcan-foundation interaction should also be considered during assessment of foundation performance and leg penetration assessments, particularly when deep footing penetrations are predicted.

Existing guidance for leg penetration analysis is largely based on conventional bearing capacity evaluations derived for silica sands and clays of terrigenous origin. For complex multi-layered soils or where likelihood of soil plug forming ahead of advancing spudcan is anticipated, a modified approach may be required as discussed in the InSafeJIP guidelines. Additional consideration should be given for intermediate soils such as silts or any unconventional soils encountered which may exhibit compressible or collapsible behavior under loading e.g., micaceous, glauconitic and carbonate sediments that may be found in US offshore sites.

The recommendations for earthquake loading given in ISO 19905-1:2016 and ISO 19901-2:2022 may be followed when evaluating jack-up vessels operating in seismically active regions. Guidance on consideration of soil cyclic degradation effects and assessment of shallow foundations supported on potentially liquefiable soils is provided in Section 8.3.9. For jack-up operations in seismically active areas with a duration less than the RCS special survey period, site-specific adjustments to the design ground motions can be considered using a detailed seismic action procedure.

While ISO 19905-1:2016 indicates that a "jackup shall be assessed as L1 using a 1,000 year return period event", it also indicates that "the exposure level applicable to a jack-up shall be determined by the jack-up owner prior to the assessment and, where applicable, shall be agreed by the regulator and operator and by the regulator and operator(s) of adjacent facilities" Similarly ISO 19901-2:2022 indicate that "other target probabilities [Pf] may be used in the detailed seismic action procedure if recommended or approved by local regulatory authorities."

Guidance Note:
For jack-up operations with a duration less than the RCS special survey period, a Pf equal to 1/250 (2% in five years) may be appropriate. Guidance is also provided in DNV-RP-0585 Section 2.7.

Although the leg penetration analyses are primarily a geotechnical design exercise, consideration should also be given to geohazards across the site (as discussed in Section 6) when reporting results of the analyses and considering the impact on installation. Items of particular concern for installation and post-installation include (but are not limited to):

- Punch through and/or leg runs during installation, operation, or subsequent storm loading events
- Leg length adequacy
- Spudcan-footprint interaction and spudcan-foundation interaction
- High seabed mobility and sand waves which can leave the spudcans exposed or vulnerable to scour; with scour being the risk particularly for the leg hung-up condition
- Scour-induced punch through
- Surface and buried boulders which could represent an installation hazard
- Sloping seabed and buried slopes which can cause instability issues
- Shallow gas
- Extraction issues / leg retraction analyses
- Earthquake events
- Debris / unexploded ordnance
- Seabed infrastructure (cables, scour protection etc.)

Guidance Note:
Wind Turbine installation jack-up units are subject to frequent moves across a site. To consider the geohazards and range of geotechnical conditions across a wind farm, the use of heat-maps for geohazards and anticipated penetrations may be beneficial.

It is recommended that leg penetration is digitally recorded throughout the installation, records should include the spudcan geometry, loads imposed, and penetration achieved together with details of any unexpected rig behavior or additional measures that were taken during installation and extraction, (for example spudcan jetting pumping records). Such records can be used for continual improvements with regards to safety and efficiency including lessons learned, recalibration of prediction models and developing field statistics for future operations.

8.4.4 Pile Drivability Methods

8.4.4.1 Impact Hammer

Determining an appropriate hammer requires calculating the likely blow counts and driving stresses for a given hammer-pile-soil configuration. It is necessary to start by calculating the soil resistance to driving (SRD). This can be done using well publicized empirical methods such as Toolan and Fox (1977) and Stevens et al. (1982). More recent methods use CPT data to calculate SRD. Reference can be made to Alm & Hamre (1998, 2001).

In-house SRD methods should be permitted provided the validity of the in-house method can be demonstrated to be applicable.

8.4.4.2 Vibratory Hammer

The vibratory hammer causes the pile to vibrate, in turn vibrating the surrounding soil which reduces the friction between the pile and surrounding soil. This reduction in soil resistance is the key to the working principle of the vibratory hammer. The maximum penetration speed and depth of the pile is then further determined by the maximum driving force of the hammer in combination with the weight of the pile and hammer set and can be limited and controlled by the crane lowering speed, hammer frequency and hydraulic working pressure.

For driveability calculation of vibratory driven piles, just as for the impact driving, the SRD needs to be calculated using CPT data.

The soil models applied in pile design and applied driveability studies are often semi-empirical. Vibratory driving predictions are in general less accurate than impact driving predictions because less data is available from executed projects to evaluate calculations. The limitation for the drivability by vibratory hammers is defined by penetration speed; any speed > 0 mm/s is considered as sufficient penetration progress.

8.4.4.3 Pile Integrity and Pile Tip Buckling

The occurrence of cemented layers and boulders may create the potential for pile tip buckling. An initially deformed pile that is driven through the soil and gradually collapses may be the most critical condition for pile buckling analysis. These failures occur due to lateral soil pressures progressively building-up around the pile circumference due to increasing wedging action of the deforming pile as it is penetrated. Whether such a failure mechanism develops is a function of the relative stiffness of the pile (inversely proportional to $(D/t)^3$) and the soil stiffness. It is also dependent on the magnitude of any initial imperfections. For further details on methodologies and recommendations, reference can be made to Aldridge et al. (2005) and Erbrich et al. (2011), among others.

Guidance notes:
There have been a number of known extrusion failures during pile driving, including some cases with very large diameter thin walled monopiles for wind farms. It is recommended that the potential for pile buckling should be considered and planned for accordingly.

Where gravels, cobbles and boulders are expected, and where pile driving refusal may occur, installation of the pile using the drill-drive-drill technique may be considered or other installation methods (e.g. drill and grout) may be considered provided the applicability of the method can be demonstrated.

8.4.5 Installation of Drilled-and-Grouted Piles

The installation of drilled and grouted piles is not subject to well established methodologies, specifications, and standards. Often, installation equipment and procedures are project specific and substantial items of equipment, such as pile templates, pile drills and grouting assemblies are fabricated for individual project characteristics. The design of the installation should consider the requirements of the design but, equally, the design of such piles should consider what is achievable in the installation process.

Offshore drilling methods for drilled and grouted piles include Subsea drill (SSD) and Pile Top Drill Rig (PTDR). With regards to SSD, the subsea drill could be mounted in a recoverable casing that would be supported by a Subsea Pile Template (SPT). The SPT should have a built-in levelling system and sits on load spreading skirts. The SPT should be equipped with an oscillator which can grip and turn the casing, providing controlled descent. For the PTDR system the machine drills inside the casing upon which it is mounted to achieve the rotation force required to drill the socket to target.

The drilling process requires a conductor pipe, or casing, to deliver flush and reaction force for the pile drill. The conductor pipe can be removable/temporary or sacrificial and left embedded into the seabed. In some cases, the pile itself can be used as the conductor and then driven into the drilled void or used as the upper part of the pile system with a smaller 'insert' pile installed and connected from the conductor pile tip to the base of the drilled void. The choice of arrangement is dependent upon the geological conditions, in particular the depth of overburden soils and the ability of those soils to self-support during installation. The conductor may be driven, pushed, or drilled (internal, possibly with controlled under-reaming) into the seabed and consideration should be given to lessening the disturbance of the overburden soils to preserve their geotechnical capacity. Detailed consideration should be given to the variability in rockhead and rock conditions between individual jacket piles if data is available to support such as assessment. If a method of drilling without a temporary casing is selected, then the clamp system from the drill tool will load directly on to the rock face to achieve the required push down force; open hole stability, hole collapse and rock breakout should be also checked against these loads.

The stability of the drilling conductor is particularly important to ensure drill hole verticality and the proper performance of the drill without snagging. Excessive deflections of the conductor could cause damage to the pile drill. Stability of the drill conductor is ensured with restraint mechanisms – grippers – at deck level on the vessel and at seabed in the piling frame.

The stability and nature of the drilled surface created by the installation method is critical in the delivery of the pile design capacity. The pile drill should be selected or designed according to a detailed understanding of the strength and fabric of the rock to ensure optimal progress drilling rates, a stable drill hole and a rock surface with sufficient roughness to generate the required capacity from the rock-grout bond. Reverse Circulation Drilling (RCD) techniques are the most commonplace in industry. The specific design of the bottom hole assembly may be specially modified with cutters, underreamers and the suchlike to achieve the desired rock socket roughness and profile.

The nature of the drilled rock surface may be specified with reference to Pells (1999) and Seidel & Collingwood (2001). The socket finish should be inspected prior to grouting, and contingency measures should be available to provide further cleaning or roughening of the socket and base with flushing. The base of the drilled socket should be clean and free from debris to ensure a good coupling between the pile tip and fresh rock, although significant pile displacement would likely be required to mobilize any substantial end bearing.

It is important that the drilled rock socket condition is checked prior to the installation of the pile and commencement of grouting operations. Techniques such as calipers, visual cameras, optical or acoustic tele-viewers can be used to complete an inspection. Given the sensitivity of the performance of the pile to the finished nature of the rock socket, inspection of each socket may be required. Depending upon the success of initial pile drilling operations and the variability of the site geology, the frequency of such inspections may be lessened.

The grout provides the structural connection between the pile and the rock mass and so should be considered carefully in the installation design. A grout system should be used which monitors grout intake, possible grout loss and which ensures the integrity of the grout layer around the pile. Mitigation measures should be anticipated to prevent excessive grout loss, such as the use of curing accelerants where the pile installation timings allow. The continuity and integrity of the grout annulus should be ensured, and the diameter of the rock socket designed to account for this. Pile centralizers, or other methods, should be considered to prevent the pinching-out of the grout annulus. Finally, various grout materials are available to the market, and each comes with specific mixing, pumping, operating temperature and curing instructions. These should be adhered to carefully to ensure that the grout manufacturers' specifications have been achieved, including but not limited to the type, grade, compositional ratios, grout mixing and curing times. A program of regular grout cube strength testing should be initiated throughout the offshore installation works to quality control the grout preparation and curing process.

8.4.6 Installation of Anchors for Floating Wind Turbines

Anchors fall into three categories regarding installation: self-weight (GBA and 'torpedo' anchors), driven or suction-assisted penetration, and drag-assisted. For suction-assisted pile installation, reference is directed to Section 8.4.2. Further details for other types of anchors can be found in ISO 19901-4:2016 Section A.11 and DNVGL-RP-E301: Design and installation of fluke anchors and DNVGL-RP-E302: Design and installation of plate anchors in clay.

8.5 Foundation Local Scour

8.5.1 Scour Assessment Methods

Seafloor variations can usually be characterized as a combination of the following processes:

- **Local Scour:** Scour around single structural elements such as a monopile or individual jacket legs. Local scour shall be considered at all foundation types.
- **Secondary Scour:** Scour occurring at the edges of scour protection (edge scour). Secondary scour shall be considered at all foundation types requiring scour protection.
- **Global Scour:** Scour occurring over a wide area caused by overall structure effects (group effects) or multiple structure interaction. Global scour shall be considered at GBS, jacket-pile and all foundation types spaced closer than 6 times the width of the foundation structure.
- **Morphologic Evolution:** Overall seabed movement due to the migration of sand waves, sand banks, ridges and shoals which would occur in the absence of a structure. Such movements can result in the lowering or rising of the seafloor. Morphologic evolution shall be considered at all structures. Scour due to morphologic evolution and seabed mobility is discussed in Section 6.1.4.

The extent of scour and the required scour protection at the wind turbine site may be determined based on:

- **Model Testing:** Physical or, less commonly, numerical model tests of the site-specific foundation type, environmental conditions, and seafloor characteristics. Physical model testing is commonly used to test scour formation and scour protection designs at novel foundation shapes (e.g., GBS) or for designs with a poor resistance to scour (e.g., suction buckets)
- **Empirical Equations:** Empirical calculations appropriate for the foundation type, environmental conditions, and seafloor characteristics. Local scour estimation and scour protection design is generally only carried out using empirical equations for common and simple foundation shapes (e.g., monopiles) situated in non-cohesive soils. These are most commonly empirical equations based on physical model test programs and occasionally validated with field records. Caution should be taken when applying empirical equations outside of the range of foundation geometries, environmental conditions or seafloor characteristics used in their derivation.

8.5.2 General Guidance

Monopile foundations can be subject to large scour depths in current-dominated conditions (e.g., 1 to 2 times the pile diameter), but only modest scour depths in wave dominated conditions, particularly when wave lengths are short relative to the pile diameter. Scour protection is generally, though not always, installed at monopile foundations. Scour development and scour protection design for monopile foundations is well represented in literature and does not necessarily require project-specific model testing.

Scour at jacket piles depends largely on the spacing between the individual piles and the presence of mud-braces. If mud-braces are absent and pile spacing is sufficiently wide, each pile can be considered as an individual vertical pile. Otherwise, group effects and global scour should be considered. As the individual piles comprising jacket piles are relatively small, scour can be less of a concern than at other foundation types, although scour protection is commonly required. Due to the range of jacket pile designs, project-specific assessment via model testing can be required to establish scour development and appropriate scour protection at jacket pile foundations.

Sources of scour include pumping scour due to structural movements or the movement of water in/out of structures (e.g., on leg piles or rocking motion of GBS) and propeller induced scour in fairly shallow water (e.g., <20m).

GBS foundations can result in large hydraulic loads on the seafloor and considerable local and global scour. Local scour development is generally rapid once scour extends below the GBS base or skirt, and scour protection is almost always required at such foundations. Global scour may be present over distances several times the width of the structure. Due to the range of GBS foundation types, project-specific assessment via model testing is generally required to establish scour development and appropriate scour protection at GBS foundations.

For all other structure types, scour development and scour protection requirements should be evaluated on a project-specific basis.

Suction bucket foundations can be sensitive to scour formation and may require scour protection to ensure structural integrity. Project-specific assessment of scour development is generally required for suction-bucket based designs.

For foundations placed in cohesive sediment (such as silt or clay) most literature may not be applicable. As scour development is typically slow in these sediments it is recommended that scour development compliance and any required remediations are defined in the 'in service life' survey scope.

8.5.3 Special Considerations

8.5.3.1 Monopile

Scour development at monopile foundations in non-cohesive sediment can generally be adequately estimated from theoretical equations if the foundation dimensions, environmental conditions, and seafloor characteristics are within the test range of the data sets (field records or model tests) underlying the applied equations. Scour development at monopile foundations in cohesive sediment (such as silt or clay) cannot be adequately estimated from theoretical equations and should be determined on a site-specific basis, with consideration given to in-service life surveys and maintenance. An adequate factor of safety should be included to account for uncertainty associated with the environmental conditions and the seafloor characteristics.

8.5.3.2 Jacket Pile / Tripod

Local scour at jacket pile foundations in non-cohesive sediment may be estimated from theoretical equations if they have been developed for a similar arrangement of structural elements. For jacket-structures not well represented in literature, or any structure placed on cohesive sediment, scour development can be estimated from site-specific model scale tests, model-scale tests of comparable sites and structure shapes, or field records from comparable sites and structure shapes. An adequate factor of safety should be included to account for uncertainties related to the structure shape, environmental conditions, and the seafloor characteristics.

Jacket pile foundations have the potential to cause global scour of the surrounding seabed. Global scour, or the potential for global scour, at jacket-structures may be estimated from theoretical equations if they have been developed for a similar arrangement of structural elements. If a similar design is not represented in literature, site-specific model tests, model tests of comparable sites and structure shapes, or field records from comparable sites and structure shapes may be required to estimate global scour.

8.5.3.3 GBS

Local scour at GBS structures can be estimated from site-specific model tests, model tests of comparable sites and structure shapes, or field records from comparable sites and structure shapes. An adequate factor of safety should be included to account for uncertainties related to the structure shape, environmental conditions, and the seafloor characteristics.

GBS foundations have the potential to cause global scour of the surrounding seabed. The overall structure effect can be estimated from site-specific model tests, model tests of comparable sites and structure shapes, or field records from comparable sites and structure shapes.

8.5.3.4 All Other

For all other structure types, or any structure placed on cohesive sediment, scour development should be evaluated on a project-specific basis.

8.5.4 Recommendations to Mitigate Effects of Scour

Depending on environmental conditions and seafloor characteristics, the foundation may be designed to accommodate scour and therefore not require scour protection. The foundation then should be designed for both upper and lower seabed levels in combination with upper and lower scour depths. If this is not a feasible option, then scour protection should be installed to limit scour depths to an acceptable level.

When scour protection is adopted as a scour mitigation strategy, it should be designed to prevent or reduce local scour, prevent the progression of secondary scour damage and mitigate potential morphologic evolution of the seabed. Scour protection is most commonly comprised of two layers of rock installed at the foot of the foundation:

- **Filter Layer:** installed at the seabed level to prevent the loss of underlying sediment and mitigate failure due to scouring at the edges of the scour protection;
- **Armor Layer:** installed above the filter layer to secure it against movement due to hydrodynamic loading induced by waves and currents.

This arrangement of armor and filter layers is typically specified due to the competing aims of each layer: a filter layer requires a relatively small stone size to secure the seabed against erosion while the armor layer required a relatively large stone size to resist the hydrodynamic loading. Typically, a filter layer is installed prior to foundation installation, which is placed on or driven through the filter layer, followed by the placement of the armor layer, which is typically comprised of stones too large to drive a foundation pile through.

More recent scour protection designs at offshore wind foundations have been designed and installed with a single rock-grading that fulfils both the filter and armor requirements. While deterministic design criteria exist for armor-filter scour protection systems, there is a general lack of accepted design criteria available for single-layer systems.

Regardless of the design approach, scour protection should consider three requirements:

1) **External Stability:** The stability of the armor layer against the hydraulic loads of waves and currents. The rock grading should be heavy enough to remain stable under hydraulic conditions up to the design storm. Most typically, some degree of deformation is allowed to minimize the required rock size. Deformation should be limited such that in an armor-filter design the underlying filter layer does not become exposed or, in a single-layer design, the remaining thickness of scour protection fulfils internal stability requirements. If a filter layer will be left exposed on the seabed for a short period following foundation placement, its stability should be checked, although not necessarily for the design event.

2) **Internal Stability:** The prevention of material, either from within the scour protection or the underlying seabed, from escaping. The internal stability of the scour protection itself should be ensured, as well as stability across the armor-filter transition. Securing the underlying seabed against winnowing, or suction removal, is generally ensured through either the provision of adequate filter layer thickness and an adequately fine filter layer gradation or careful control of the gradation and thickness of a single layer scour protection. It can be more efficient to account for some amount of seabed loss in the foundation design, rather than attempting to design a perfectly sand-tight scour protection.

3) **Flexibility:** The ability of the scour protection to adapt to changes in the surrounding seabed. Secondary scour will form downstream (with respect to the dominant flow condition) of the scour protection causing degradation of the scour protection edge. Large scale morphologic change (e.g., due to the migration of large bedforms) is generally cited as a further process driving a requirement for additional scour protection volume as mitigation against degradation.

Care should be taken if a structure is placed in an area where large or rapid morphological change is anticipated. For example, foundations installed in sand wave fields will likely have to accommodate the lowering of the surrounding seabed over their design life.

8.5.5 Foundation Scour Monitoring

Observational methods are generally adopted in the design of scour protection for the offshore wind industry, provided it is possible to monitor the scour protection to be able to predict that a failure may occur; and to mitigate this failure before it affects the expected lifetime of the assets.

General seabed levels at the project site should be monitored at regular intervals and after severe storms. If the measured seabed levels are within the design levels and no damage to the scour protection has occurred, no further action is required. If the measured seabed levels are outside the design seabed levels or the scour protection has deformed outside of its design range, mitigating action should be taken.

Monitoring methods and procedures for in-service life surveys are described in Section 5.3.1.8.

Recommendations for a scour monitoring program and survey intervals are provided in the following:

A post-installation as-built out-survey should be performed closely following the installation of scour protection.

A further three surveys covering representative turbine positions should be performed within the first 8 years following commissioning of the wind farm:

- The first survey may be performed within the first year and a half after commissioning, ideally after the first and before the second storm season (winter) from time of commissioning
- The second survey may be performed within the first three and a half years after commissioning, ideally after the second and before the fourth storm season (winter) from time of commissioning
- The third survey may be performed between the 5th and the 8th year from commissioning

The first and second surveys are aimed at observing any unexpected damage to the scour protection, while the third survey is intended to observe longer term seabed changes and adaptation of the seabed to the presence of the scour protection.

The survey schedule for the remaining lifetime of the wind farm shall be determined after the first three surveys. This schedule should include, as a minimum, two further surveys over the remaining lifetime of the wind farm.

An event driven post storm bathymetric survey is recommended within a year from passage of a severe storm event. A 10-year return period storm in terms of wave conditions has often been considered appropriate as trigger for post storm survey.

8.6 Foundation Maintenance, Design Life Extension, and Decommissioning

Guidance on Operations and In-Service Inspections and Life Cycle planning are provided in ACP-OCRP-1, Sections 8 and 9, respectively. Further guidance on foundation issues is provided herein.

8.6.1 Background

After the offshore foundation installation, the foundation behavior and performance can be measured and checked against their foundation design criteria. This process may be used to investigate foundation life extension from a technical point of view.

8.6.2 Measurements and Surveys

Typical measurement and survey data used in foundation performance and life extension review are:

- Driving records (for driven pile foundations)

- Suction caisson installation records (for suction caisson foundations)
- Anchor installation and pre-tensioning records (for floating structure foundations)
- Bathymetric survey results
- Structure settlement measurements (total and differential)
- Structural frequency measurement
- Metocean data
- Corrosion.

General guidance on foundation installation and installation records is provided in ACP OCRP-1, Section 7.7-3.

The driving records can come in different forms, depending for instance if pile driving analyzer records were made. To be usable for back-analysis, the driving record data package in its most basic form should include (for every pile foundation):

- Project details, including the foundation location (e.g., WTG location), if more than one pile per foundation, the pile numbering details as per design of installation plan, the installation date and time (accuracy to the nearest minute)
- The pile details: diameter, wall thickness, length during driving (including stick-up), embedment length, batter angle
- Hammer details: brand and energy rating
- Installation details (e.g., use of follower)
- Stop and start of driving activity (minute accuracy)
- Measured pile self-weight penetration (after placing the hammer assembly on the pile)
- For every penetration depth interval below the self-weight penetration, with typically intervals of 0.25 m or one foot:
  - Pile tip penetration depth
  - Blow count over this interval
  - Hammer energy used over this interval.
- Measured pile tilt after installation (note: for jacket on pile, this can be derived from the jacket measured tilt).

For suction installed foundations, installation records are mandatory for back-analysis. This is to allow an understanding of potential differences between predicted (theoretical) behavior and observed behavior (e.g., from frequency measurements). The suggested suction installation record data package should include (for every foundation):

- Project details, including the foundation location (e.g., WTG location), if more than one suction caisson/bucket/pile per foundation, the numbering details as per design of installation plan, the installation date and time (accuracy to the nearest minute)
- The suction foundation details: diameter, wall thickness, theoretical embedment, design heave
- Installation details: minimum design suction tip embedment before pumping, maximum design allowable pumping rate versus suction tip embedment
- Pump details: brand, type, flow rate
- Stop and start of pumping activity (typically second accuracy)
- Measured suction foundation self-weight penetration
- For every penetration depth interval below the self-weight penetration, with typically intervals of 0.01 m or one inch:
  - Suction foundation tip penetration depth
  - Pump flow rate
  - Time (typically second accuracy).

- Measured suction foundation tilt after installation (note: for jacket on suction caisson/bucket, this can be derived from the jacket measured tilt)
- Measured soil heave within the suction foundation after installation
- Information if the gap between the soil and the top plate has been filled. If filled (e.g., with grout), type and volume of fill.
- Observation of ROV survey, with details of any observed piping of installation anomaly.
- If the suction foundation is used as an anchor, measured anchor pre-tensioning force at the pad eye. If not measured directly at the anchor pad eye, results of an assessment using measured pre-tensioning force along the tensioning chain.

For floating structure anchors (other than pile or suction caisson), the following dataset shall be reviewed:

- Project details, including the foundation location (e.g., WTG location), anchor numbering details as per design of installation plan, the installation date and time
- The anchor details: type, weight, design embedment
- Anchor embedment (post installation)
- Measured anchor pre-tensioning force at pad eye. If not measured directly at the anchor pad eye, results of an assessment using measured pre-tensioning force along the tensioning chain.

Bathymetric surveys are typically done punctually, on a recurring basis. For foundation analysis, they should be accurate enough to allow an understanding of the seafloor evolution in the vicinity of the foundation (e.g., local scour evolution or sand wave mobility or seafloor subsidence) and conservative assumptions should be considered to recognize the potential seafloor evolution between surveys (e.g., potential seabed mobility during storm which may not be captured by punctual surveys).

Continuous WTG frequency measurements is standard practice in the offshore wind industry and is critical to back-analyze WTG foundation behavior and their evolution over time.

WTG structure settlement measurements, ideally taken during bathymetric surveys, are useful to determine the amount of settlement that has occurred related to any remaining settlement that could occur through life extension. Differential settlement can provide information on spatial variability across the WTG foundation footprint but should be considered in the context of predominant loading directions.

Metocean data are typically gathered at different locations across the wind farm array. A review of these data gathered over time can allow for accurate fatigue assessment and design storm redefinition.

Corrosion measurements level of accuracy should be accounted for to define the foundation wall thickness to be used in foundation analysis (e.g., for foundation life extension). If possible, a history of corrosion measurements will help to understand the development of corrosion weak point in the foundation structure and assess the corrosion rate.

8.6.3 **Design Verification, Foundation Life Extension**

8.6.3.1 **Ground Conditions Review**

Particular attention will need to be paid to alteration of the seafloor which may have occurred after the initial design (e.g., adjacent spudcan footprint, grout or cutting spill) and scour.

8.6.3.2 **Fatigue Assessment**

The fatigue life of a structure is generally defined in terms of:

- Pre-installation fatigue (e.g., in the yard and during transportation),
- Installation fatigue life (e.g., driving fatigue for driven piles or monopiles),
- Fatigue during the structure lifetime

- Fatigue during decommissioning.

During early design (before foundation construction), foundation fatigue assessments consider a range of conservative assumptions to mitigate unknowns and uncertainties during the fatigue life phase above. Later during the structure life, some unknowns and uncertainties have been lifted and this can allow for a more accurate prediction of the structure fatigue. In particular:

- The installation records can be used to back-analyze the foundation installation fatigue more accurately
- At some point during the structure lifetime, the measured metocean and bathymetric data can be used to predict the fatigue structure more accurately at this point of time. This can be critical for foundation life extension or decommissioning assessment.

8.6.3.3 **Frequency Assessment**

Frequency measurement allows for indirectly monitoring the foundation behavior. It can also evolve over time (e.g., depending on seabed mobility at the base or after a significant storm event). A back-analysis of the frequency measurements, through a coupled analysis, may allow for a review of foundation design assumption (e.g., regarding soil characteristics of soil-structure springs). Such foundation design assumption reassessments should account for geotechnical variability and the long-term use of the structure. It should be recognized that the frequency back-analysis process may need to consider specific parameters that are different from design parameters (e.g., regarding corrosion allowance or behavior during a storm).

8.6.3.4 **Ultimate Limit State Assessment**

An ultimate limit state reassessment of the foundation may be required if during the structure lifetime some of the ultimate limit state design criteria have changed. This can be for instance an adjustment of the design storm wave or design earthquake. In such case, the design process described in the subsections under Section 8 relevant to the type of foundation considered should be completed.

8.6.3.5 **Serviceability Limit State Assessment**

A serviceability limit state reassessment of the foundation may be required if the structure has experienced significant rotation or settlement, or if the serviceability limit state design criteria have changed. This can be for example an adjustment of the allowable tilt of the structure, or allowable settlement, for example with respect to J-tube connection and interaction with the seabed and inter-array cable.

8.6.4 **Decommissioning**

At the end of the service life of an offshore foundation, decommissioning requirements may be imposed where the foundation will be partially or fully extracted from the seabed, subject to environmental restrictions. Some foundation types, e.g., piles, may be allowed to remain buried below the seabed but should be cut at some depth below mudline to avoid interference with fishing trawl operations, vessel anchoring and future cable installation. Suction piles, suction anchors and caissons are generally extracted from the seabed by reversing the installation process through application of overpressure below the foundation base. For plate, fluke, drag and other floating structure anchors, these may be able to be extracted through crane operations or left in the seabed with the mooring line cut below the seabed elevation.

Detailed guidance can be found in:

- API RP2GEO/ ISO 19901-4 Section 7.11, Appendix A.7.11, and Appendix A.7.12.2
- DNVGL RP C212 Section 7.3.8 and 7.3.9
- DNVGL RP N103 Section 6.4

General considerations for foundation decommissioning include:

- For skirted foundations, caissons and suction anchors, skin friction during extraction may be higher than during installation due to soil setup through thixotropy, ageing and consolidation effects.

- For foundations with an underbase filter, limited suction is generated during lifting operations and therefore the total extraction resistance is approximately equal to the on-bottom weight of the foundation.
- For foundations without an underbase filter, sustained tension may be required to allow dissipation of excess pore pressures that develop below the base. This may be seconds or minutes in sand but is generally not feasible in clay. In clay, the extraction resistance may include the full reverse end bearing from the soil plug. Lifting operations should account for this.
- For foundations with internal stiffening, designers should consider the potential increased end bearing resistance, which may be higher during extraction than during installation.
- If overpressure is used to extract the foundation, controlled extraction should be performed to limit the potential for gapping and subsequent piping to occur, which would also limit the effectiveness of underbase pressurization. However, intentional inclined extraction can help to limit the extraction resistance when overpressure is not required.
- Application of cyclic tension loading from the lift vessel crane or cyclic over and underpressure may be used to degrade the increased skin friction.
- Jetting of the soil around the perimeter of the foundation may also be used to reduce the skin friction resistance but should avoid creation of a gap or scour hole that could limit the effectiveness of underbase pressurization later in the extraction process.
- As with installation, checks should be performed to ensure the overpressure is less than the allowable bearing capacity of the soil plug as well as within the allowable pressure on the steel walls of the anchor or caisson.

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